ALKALI-AGGREGATE REACTIVITY (AAR) FACTS BOOK

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16. Abstract
This document provides detailed information on alkali-aggregate reactivity (AAR). It primarily discusses alkali-silica reaction (ASR), covering the chemistry, symptoms, test methods, prevention, specifications, diagnosis and prognosis, and mitigation. Alkali-carbonate reaction (ACR) is also addressed.

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Alkali-silica reactivity, alkali-aggregate reaction, reactive aggregates, ASR, concrete durability, symptoms, identification, prevention, mitigation, lithium

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

Table of Contents

1 – Introduction

1.1 DEFINITIONS

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 following definitions can be found on the website of the American Concrete Institute (ACI)[1]:

Alkali-aggregate reaction, 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.
Alkali-carbonate reaction, 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.
Alkali-silica reaction, 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.

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[1] www.concrete.org

ASR is far more widespread than ACR and is the main focus of this text. 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.

1.2 HISTORY OF AAR

Problems due to ASR were first identified in the State of California in the 1930s (see Figure 1.1) 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, such as the Parker Dam in Arizona (Meissner 1941), and in the 1940s a number of agencies initiated studies on ASR in the USA (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 USA, incidences having occurred in most, if not all, of the contiguous states, and numerous countries worldwide.

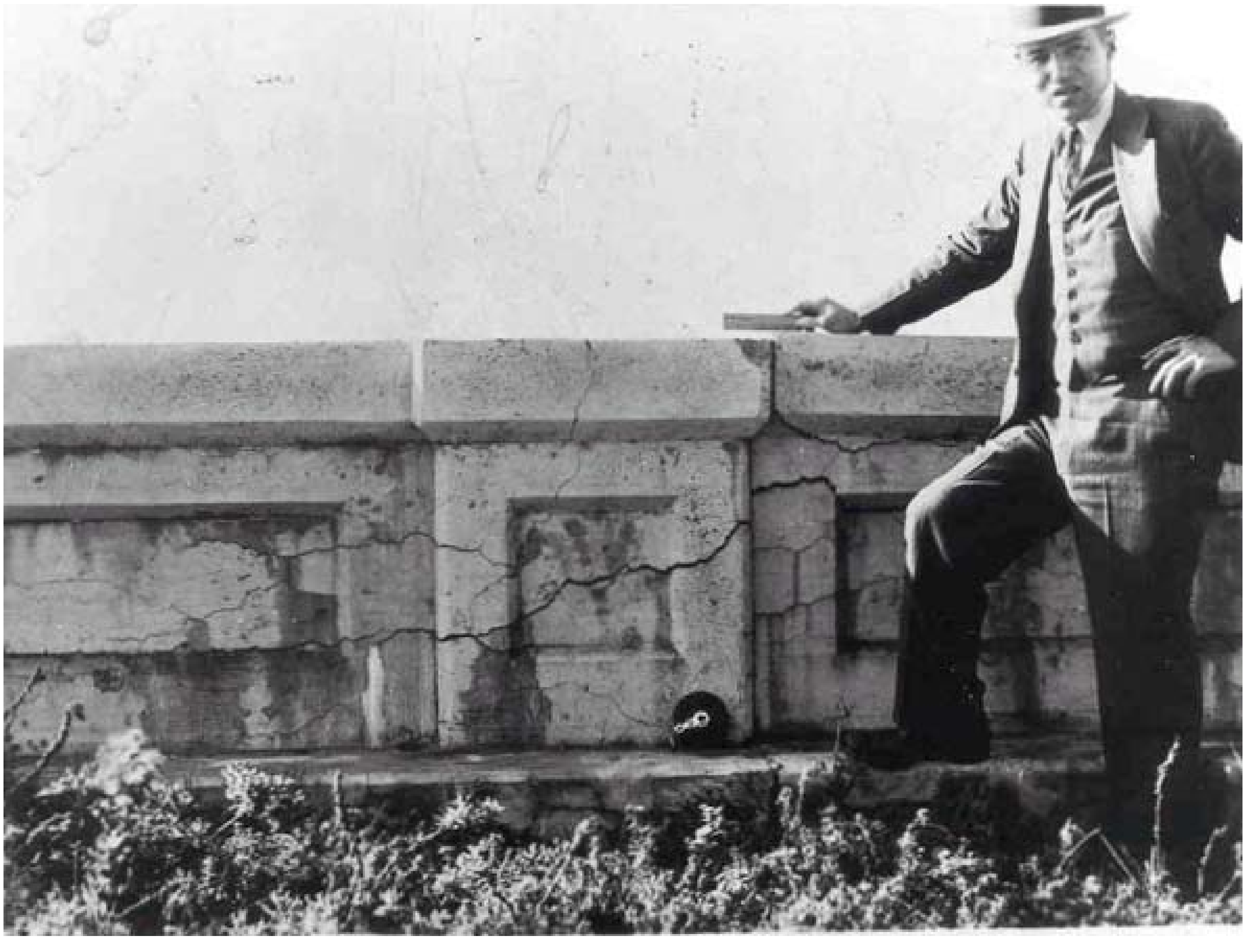

Figure 1.1. Thomas Stanton of the California State Division of Highways and a Bridge Parapet Wall that is Showing Signs of Damage due to Alkali-Silica Reaction

Alkali carbonate reaction (ACR) was first discovered by Swenson (1957a) as the cause of concrete deterioration in Canada at about the same time that ASR was first documented in the same country (Swenson 1957b). ACR was subsequently implicated in cases of degradation of concrete structures in the USA (Hadley 1961) and alleged cases of ACR have now occurred in Virginia, West Virginia, Kentucky, Missouri, Tennessee, Iowa, Illinois, Indiana, and New York, as well as England, Bahrain, Iraq, and China (Ozol 2006). However, unlike ASR, problems with ACR are still restricted to a few isolated locations worldwide. Consequently, there has been comparatively little research conducted on this topic.

A series of international conferences on alkali-aggregate reaction (ICAAR) in concrete began in 1974 (see Table 1.1). The first conference was held in Køge, Denmark in 1974 with 23 delegates presenting 13 papers and representing just 5 countries (Denmark, Germany, Iceland, U.K., USA). Interest in AAR grew rapidly from this time and in 1992 over 300 delegates representing 29 countries attended the 9th ICAAR in London, U.K., and 150 papers were published in the proceedings. Interest has remained at this level since that time with the most recent conference being held in Austin, Texas, USA, in 2012 (131 papers from 27 countries).

Table 1.1. International Conferences on Alkali-Aggregate Reaction in Concrete

#	Year	Host
1	1974	Køge, Denmark
2	1975	Reykjavik, Iceland
3	1976	London, U.K.
4	1978	West Lafayette, USA
5	1981	Cape Town, S. Africa
6	1983	Copenhagen, Denmark
7	1986	Ottawa, Canada
8	1989	Kyoto, Japan
9	1992	London, U.K.
10	1996	Melbourne, Australia
11	2000	Quebec City, Canada
12	2004	Beijing, China
13	2008	Trondheim, Norway
14	2012	Austin, U.S.A.

Alkali-silica reaction is now widely recognized as one of the more prevalent deterioration mechanisms affecting concrete worldwide[2].

1.3 AAR TODAY

Seventy years after ASR was first documented, much is now known about the chemistry of the reaction, the factors that contribute to the reaction and expansion, methods for testing aggregates, and strategies for preventing expansion. Proper application of the knowledge available today to new concrete construction should result in a very low risk of damage due to ASR occurring in the normal service life of the structure. A number of specifications or practices have been developed in recent years to assist the practitioner in the selection of materials and preventive measures for ensuring durable construction (with regards to AAR). This text reviews one such practice, AASHTO PP65, in detail (see chapter 6).

It is well established that the ASR results from a reaction between the alkali hydroxides provided (mainly) by the portland cement and certain types of reactive silica minerals present in some aggregates, and that limiting the availability of one (or both) of these is an effective

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[2] A comprehensive review of the history of alkali-aggregate reaction from 1940 to 1996 is given by Idorn (1997).

means of preventing deleterious expansion due to ASR. Thus, selecting "non-deleteriously-reactive" aggregates or using "low-alkali cement" have become common practices for preventing ASR, although it is now considered that controlling the alkali content of the concrete is more appropriate than merely limiting the alkali content of the cement.

The potential for using pozzolans to control damaging ASR was demonstrated by Stanton (1940) in his landmark paper that first revealed the phenomenon of alkali-silica reaction to the concrete community. The use of pozzolans for this purpose was first put into practice in the same decade when calcined clay was used to prevent ASR in the Davis Dam (Gilliland and Moran 1949), which was constructed between 1942 and 1950, the reaction having been implicated as the cause of cracking in the Parker Dam (Meissner 1941), which was completed shortly before construction began on the Davis Dam and is located 88 miles (141 km) upstream on the Colorado River. Ten years after Stanton's (1940) discovery of ASR the potential for using fly ash and slag for controlling expansion was first documented, and it is now widely accepted that supplementary cementing materials are an effective means for controlling ASR expansion provided they are used at a sufficient level of replacement (see chapter 5). The long-term field performance of fly ash in the role of ASR prevention was recently documented (Thomas et al. 2012) in the form of the excellent condition of the 50-year-old Nant-y-Moch Dam in Wales (Figure 1.2) and the 40-year-old Lower Notch Dam in Canada, both structures being built with the combination of fly ash and highly reactive aggregates.

Figure 1.2. The Nant-y-Moch Dam in 2011 – No Symptoms of ASR after 50 Years – Constructed with Reactive Aggregate and 25% Fly Ash

Test methods for correctly identifying reactive aggregates and evaluating the efficacy of preventive measures have constantly evolved since Stanton’s (1940) mortar-bar test, which was a precursor to the standard ASTM C 227 method. At the current time practitioners face the dilemma of selecting between tests that are reliable but have (often unacceptably) a long test duration or rapid tests that often have a poor correlation with field performance. The search is still on for rapid and reliable test methods. Testing methods for ASR and ACR are discussed in chapter 4.

There are few options available for mitigating ongoing AAR in existing structures. In other words, once concrete has alkali-aggregate reaction, it is very difficult to stop the reaction. In some cases, it may be possible reduce the availability of moisture and slow the reaction down. In other cases, methods have been developed to either physically confine the expansion or create space to allow for expansion and relieve stresses. Chapters 7 and 8 in this text discuss methodologies for evaluating existing concrete structures to (a) determine the extent of ASR and its impact on the concrete, (b) predict the future growth of the concrete due to ASR, and (c) select appropriate strategies for mitigating the effects of ASR.

There is comparatively little information on ACR, and consensus has yet to be reached on the exact mechanisms of expansion. Although it is agreed that alkali-carbonate reactive dolomitic limestones have a characteristic texture and composition and undergo a chemical reaction resulting in dedolomitization (dolomite → brucite + calcite), there is disagreement as to whether the accompanying expansion results from this reaction or from reaction of cryptocrystalline silica in the limestone (i.e., ACR expansion may be due to ASR). There does appear to be consensus that, regardless of the true mechanism of expansion, there are features of the alkali-carbonate reaction that set it apart from ASR with aggregates that are undisputedly alkali-silica reaction. These features include (a) the relatively short timeframe before damage is observed, (b) reaction (and expansion) at very low alkali levels, (c) the general ineffectiveness of pozzolans and slag in controlling expansion, and (d) the inability of certain tests to identify the reactive aggregates. ACR is discussed in chapter 9 and a case is made for treating these reactive rocks as a separate category of AAR, irrespective of whether expansion is due to ACR or ASR.

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2 – Fundamentals of Alkali-Silica Reaction

2.1 INTRODUCTION

As described in the introduction, alkali-silica reaction is a reaction between the alkali hydroxides in the pore solution of concrete (or mortar) and certain types of silica minerals present in some aggregates. The reaction product, an alkali-silica gel with varying amounts of calcium, is hygroscopic having a tendency to absorb water and swell. Under certain conditions the reaction causes expansion of the concrete and may eventually lead to cracking. A schematic showing the sequence of events is shown in Figure 2.1.

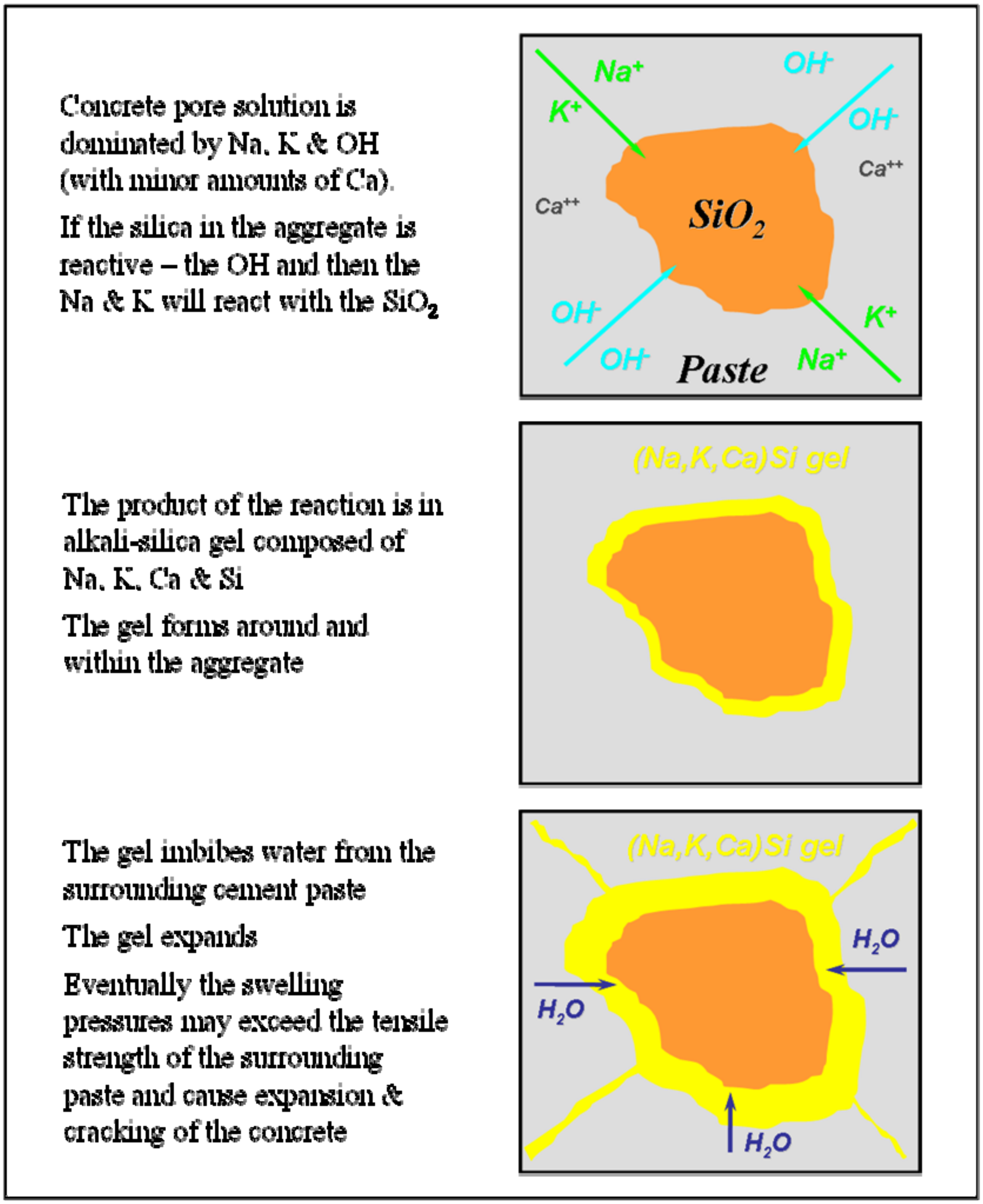
It is clear from this brief description of ASR that there are three requirements for damaging reaction to occur; these are:

- 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.

This chapter describes the chemistry of the reaction and the mechanisms of expansion, and discusses the role of various contributing factors including the composition and nature of the reactive aggregate, sources of alkali, and the impact of the exposure conditions.

Alkali-carbonate reaction (ACR) is discussed in chapter 9.



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

2.2 CHEMISTRY OF THE REACTION

Despite the term, alkali-silica reaction is initiated by a reaction between the hydroxyl ions in the pore solution and certain types of silica in the aggregate. Silica (SiO_2) is composed primarily of siloxane groups (≡Si-O-Si≡) but even crystalline silica is disordered at the surface and the surface oxygens are hydroxylated (even in pure water) forming silanol groups (≡Si-OH); see Figure 2.2. In the presence of a high concentration of hydroxyl ions (OH^-) silica tends towards dissolution first by neutralization of the silanol groups and then by attack on the siloxane groups; the reactions may be represented as follows[3]:

$$\equiv Si\text{-}OH + OH^- \rightarrow Si\text{-}O^- + H_2O \qquad \text{Eqn. 2.1}$$

$$\equiv Si\text{-}O\text{-}Si\equiv + 2OH^- \rightarrow 2Si\text{-}O^- + H_2O \qquad \text{Eqn. 2.2}$$

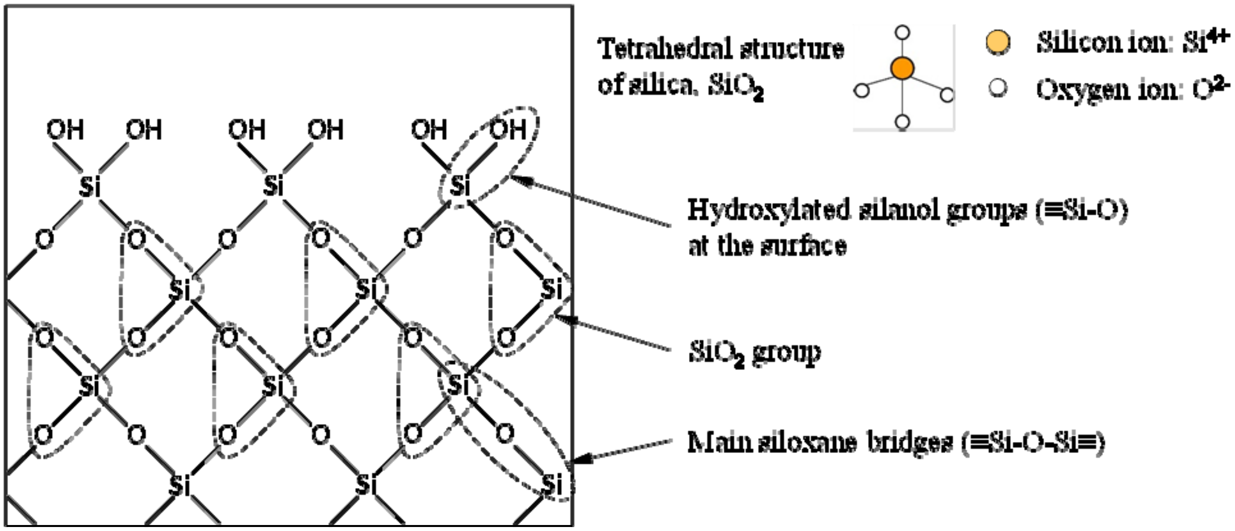


Figure 2.2. The Structure of Silica

The structure breaks down progressively as the siloxane bridges are broken (see Figure 2.3). The negatively charged Si-O- ions attract positively charged species such as sodium (Na^+) and potassium (K^+), which are abundant in concrete pore solution. The initial result is an alkali-silicate solution or gel depending on the moisture content. However, in the presence of calcium, the silica precipitates from solution as an alkali-silicate gel ($CaO\text{-}Na_2O/K_2O\text{-}SiO_2\text{-}H_2O$), primarily composed of sodium, potassium, and silica, with small amounts of calcium. The hydroxyl ion concentration (and the pH) decreases as silica dissolves. In a system composed solely of alkali hydroxide and silica, the silica will continue to dissolve until the concentration in solution reaches the silica-pH equilibrium curve (see Figure 2.4). The data show that final

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[3] The reactions may as also be written as follows (modified from Dent Glasser and Kataoka 1981a):

$$Si\text{-}OH + OH^- + Na^+ \rightarrow Si\text{-}O\text{-}Na + H_2O \qquad \text{Eqn. 2.1a}$$

$$Si\text{-}O\text{-}Si + 2OH^- + 2Na^+ \rightarrow 2(Si\text{-}O\text{-}Na) + H_2O \qquad \text{Eqn. 2.2a}$$

silica concentration depends on the initial SiO_2/Na_2O ratio as will be discussed later in the section on "pessimum behavior". The situation is more complex in concrete, probably due to the presence of abundant calcium, which reduces silica concentrations in solution and provides an additional source of hydroxyl ions. Consequently, equilibrium conditions are reached slowly in concrete (and mortar).

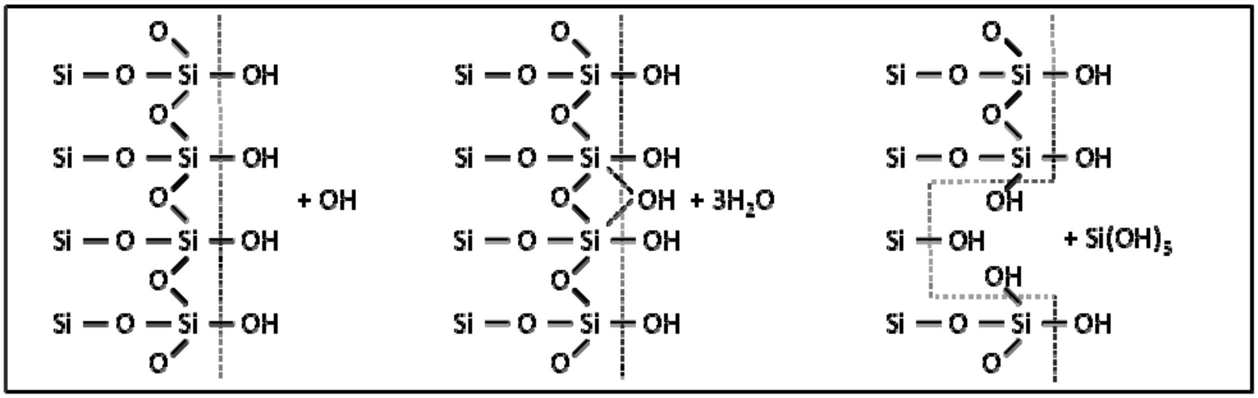


Figure 2.3. Mechanism of Dissolution of Silica due to Attack by Hydroxyl Ions. Dotted Line Represents the Interface between Silica and Water (Iler 1979; Urhan 1987)

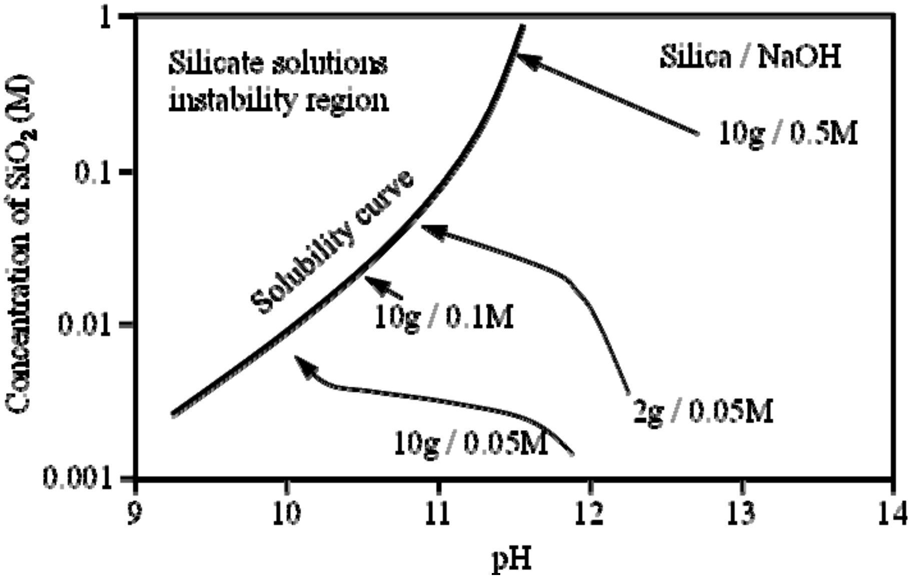


Figure 2.4. Equilibrium Solubility Curve (SiO_2-pH) and Change in Composition for Different Solutions of SiO_2-NaOH (from Dent Glasser and Kataoka 1981b)

The chemical composition and morphology of the reaction product (gel) vary widely as shown by Moranville-Regourd (1989), and other researchers have shown that the physical properties of gels, in terms of the capacity for unconfined swelling or for exerting forces when confined, also vary considerably.

2.3 MECHANISM OF EXPANSION

Figure 2.5 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.

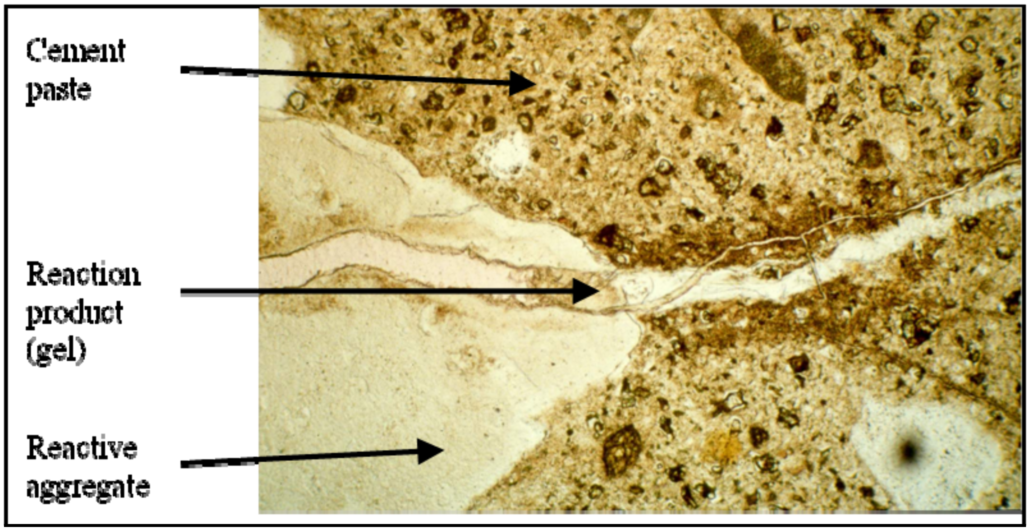


Figure 2.5. Photograph of a Thin Section of Concrete Undergoing ASR (field of view ~ 5 mm)

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 alkalis in the reaction product thereby regenerating alkalis for further reaction (Thomas 2001; Hansen 1944). Alkali recycling is discussed further in section 2.5.3 of this chapter.
- $Ca(OH)_2$ 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).
- Pozzolans are effective in controlling the expansion of concretes (and mortars) immersed in alkaline salt (including alkali hydroxide) solutions (Chatterji et al. 1987; Kawamura et al. 1988; Alasali and Malhotra 1991; Bleszynski and Thomas 1998) and this is attributed to the consumption of $Ca(OH)_2$ by the pozzolanic reaction which reduces the availability of calcium for the alkali-silica reaction. Bleszynski and Thomas (1998) showed considerable evidence of reaction in concrete containing 40% fly ash and a reactive flint sand after 4 years immersion in 1 NaOH at 80°C, but no expansion. Alkali-silica gel was observed to be migrating into and saturating the cement paste surrounding reactive flint particles with no signs of damage (see Figure 2.4). They

attributed this to the very low viscosity of the reaction product formed in the absence of calcium. By contrast, other workers (Tang et al. 1983; Wang and Gillott 1991) showed that the addition of CaO can induce expansion on concretes containing fly ash or silica fume.

- Expansion of mortars or concretes containing reactive aggregate can be prevented by removal of the $Ca(OH)_2$ prior to immersion of the sample in alkaline solutions. Chatterji (1979) removed the $Ca(OH)_2$ from concrete by leaching, and Thomas (2000) carbonated the $Ca(OH)_2$ by exposing mortar bars to an enriched-CO_2 environment.

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.

Dependence on $Ca(OH)_2$ for the promotion of damaging AAR is not a recent phenomenon. Conrow (1952) suggested that the expansion of concretes containing a siliceous sand-gravel may be related to the quantity of $Ca(OH)_2$ produced by the cement and that the beneficial effect of pozzolan is related to its ability to react with $Ca(OH)_2$. In a discussion of this paper, Mather stated an observation that concrete that has undergone ASR is characterized by "… materially reduced quantities of crystalline calcium hydroxide" and suggested that $Ca(OH)_2$ may be consumed by ASR and that "... the mere consumption of calcium hydroxide by reaction with a pozzolan is sufficient to explain the beneficial effects of pozzolans in preventing abnormal expansion."

2.4 REACTIVE SILICA

2.4.1 Types of Reactive Silica

Table 2.1 provides a list of deleteriously reactive rocks, minerals, and synthetic substances.

Table 2.1. Deleteriously Reactive Rocks, Minerals and Synthetic Substances
(from ACI Committee 201, 1991)

Reactive substance (mineral)	Chemical composition	Physical character
Opal	$SiO_2 \cdot nH_2O$	Amorphous
Chalcedony	SiO_2	Microcrystalline to cryptocrystalline; commonly fibrous
Certain forms of quartz	SiO_2	Microcrystalline to cryptocrystalline; crystalline, but intensely fractured, strained, and/or inclusion-filled
Cristobalite	SiO_2	Crystalline
Tridymite	SiO_2	Crystalline
Rhyolitic,dacitic, latitic, or andesite glass or cryptocrystalline devitrification products	Siliceous with lesser proportions of Al_2O_3, Fe_2O_3, alkaline earths and alkalis	Glass or cryptocrystalline material as the matrix of volcanic rocks or fragments in tuffs
Synthetic siliceous glass	Siliceous, with lesser proportions of alkalis, Al_2O_3, and/or other substances	Glass

The most important deleteriously alkali-reactive rocks (that is, rocks containing excessive amounts of one or more of the substances listed above) are as follows:

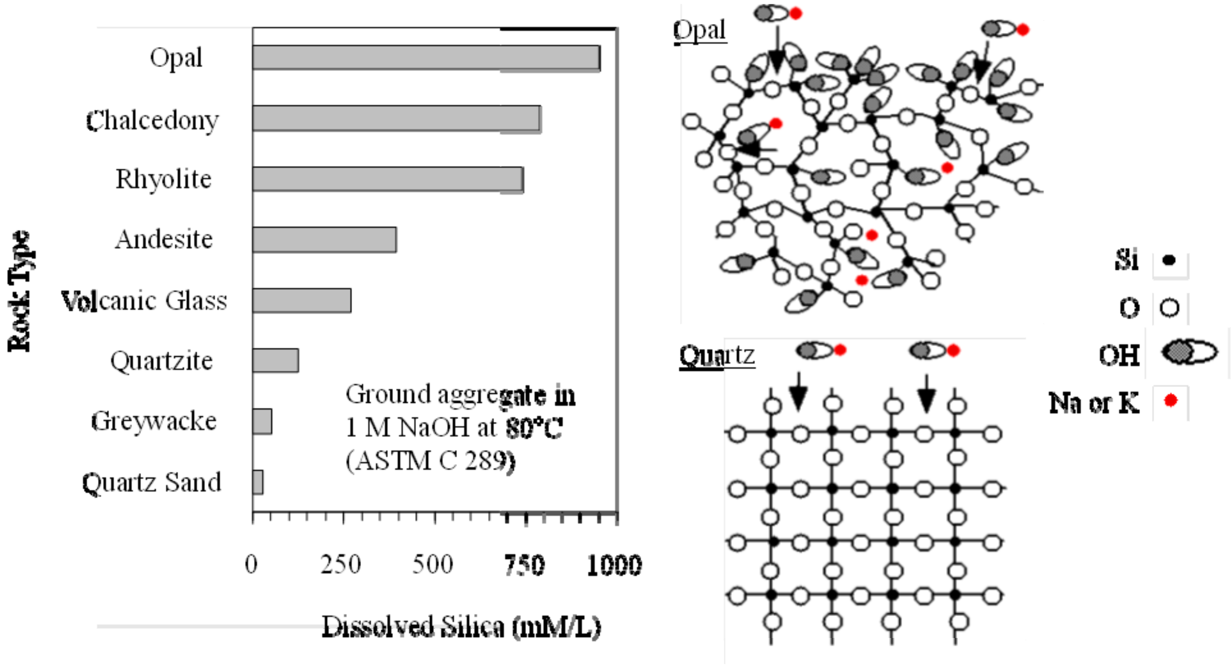
Opaline cherts
Chalcedonic cherts
Quartzose cherts
Siliceous limestones
Siliceous dolomites

Rhyolites and tuffs
Dacites and tuffs
Andesites and tuffs
Siliceous shales
Phylites

Opaline concretions
Fractured, strained, and limestone-filled quartz and quartzites

NOTE: A rock may be classified as, for example "siliceous limestone," and be innocuous if its siliceous constituents are other than those indicated above.

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 2.6 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[4].



(a) Soluble Silica in Aggregates

(b) Schematic Showing Crystalline Structure of Opal (top) and Quartz (bottom)

Figure 2.6. 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,

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[4] The potential for reaction exists even with well-crystallized silica (e.g. quartz), but the reaction is slow and may be considered to be negligible with regards to the design life of a concrete structure. However, temperature dramatically increases the rate of reaction and, in hydrothermal conditions, finely ground quartz is extremely reactive.

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 chapter 4.

Poorly crystalline or amorphous silica minerals such as opal, cristobalite, volcanic and artificial glasses react rapidly and may cause damaging reaction in a few years when present in amounts as little as 1%. Varieties of quartz such as cryptocrystalline, microcrystalline, or strained quartz react more slowly, take longer to produce damage, and are generally required to be present in greater quantities than poorly crystalline and amorphous forms of silica. However, it is difficult to classify aggregate reactivity based solely on mineralogy as aggregates may contain various types of reactive minerals in different quantities, and the extent to which reactive minerals cause damage in concrete depends on other factors such as particle size.

2.4.2 Effect of Aggregate Size

Stanton (1940) showed in his formative work on ASR that the particle size of the reactive aggregate could have a profound effect on the magnitude of expansion of mortar. Figure 2.7(a) shows the expansion of mortar bars containing a reactive siliceous magnesian limestone partitioned into various size ranges. As expected, the expansion at a given age was greater for mortars produced with the 180-600 micron fraction than for the larger fractions (0.6-2 mm and 2-6.7 mm), especially at earlier ages, and this is attributed to the greater surface area and increased accessibility of the reactive silica. What was surprising was that the mortar bars produced with reactive aggregate passing 180 micron did not exhibit any deleterious reaction. Stanton (1940) hypothesized that the absence of expansion when the aggregate is in a finely-divided form is due either to (a) the reaction being "dissipated throughout the mass in such a way as to cause no high expansive forces" or (b) the reaction being essentially complete before the mortar sets. Other workers have since shown similar data although the particle size to which the reactive aggregate has to be reduced to prevent expansion has not been established unequivocally. For example, Vivian (1951) using the same reactive aggregate as Stanton (1940) showed that expansion was delayed but not prevented for aggregate passing the 70-µm but retained 50-µm sieve, whereas no expansion was observed for aggregate passing the 50-µm sieve. Vivian (1951) also showed similar trends in behavior with opal as the reactive aggregate.

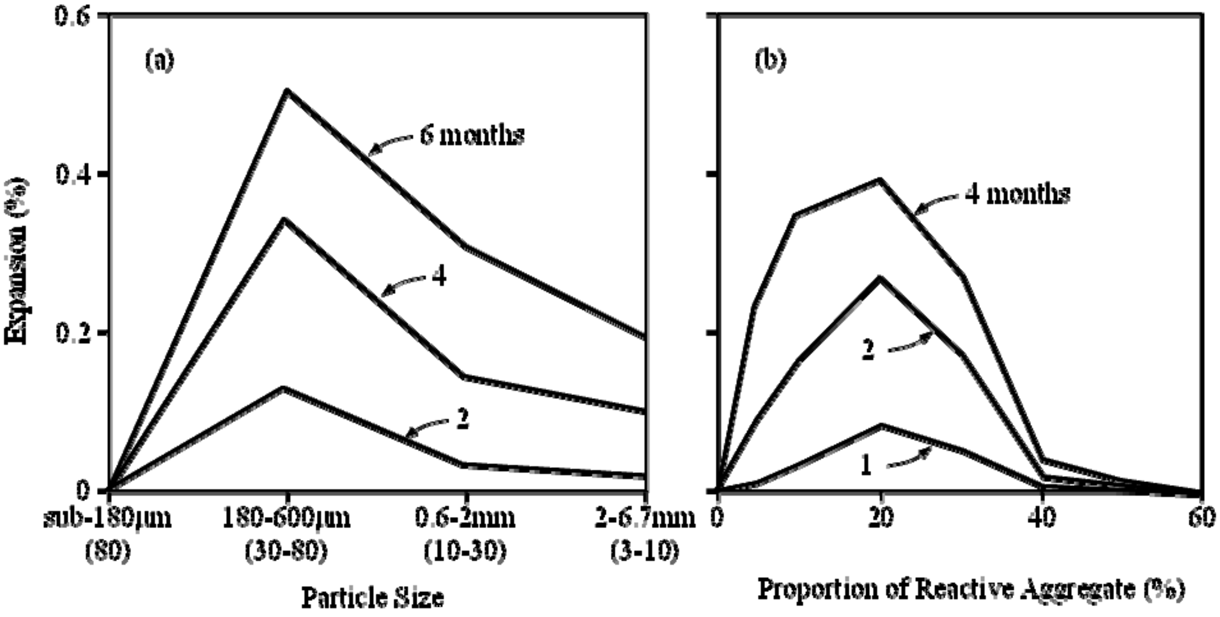

Figure 2.7. Effect of Aggregate Size (a) and Proportion (b) on Expansion due to ASR (Stanton 1940)

Numerous laboratory studies have shown that the expansion of mortar or concrete containing highly-reactive recycled glass as an aggregate can be eliminated provided the glass is crushed to a certain fineness (Carpenter and Cramer 1999; Shayan and Xu 2004; Jin et al. 2000). In fact glass crushed to high fineness may be effective in preventing damaging alkali-silica reaction when larger particles of the same glass are used as (reactive) aggregates (Shayan and Xu 2004), although such an approach may not be effective if the alkali content of the glass is high. Jin et al. (2000) suggested that the pessimum size, that is the size fraction that causes the greatest expansion when tested in mortar, reduces as the reactivity of the glass increases.

The strategy of using ground reactive aggregate as a preventive measure for ASR has been employed in Iceland where up to 9% ground rhyolite has been interground with portland cement clinker to produce a blended cement for controlling expansion with reactive rhyolitic aggregates (Asgeirsson and Gudmundsson 1979). The use of finely-divided, alkali-silica reactive materials (e.g., pozzolans) to control ASR is discussed in detail in chapter 5.

It should be noted that there is some evidence that reactive silica may cause damaging reaction even when it is finer than 30 µm. Diamond and Thaulow (1974) observed expansion of mortar bars containing a high-alkali cement and reactive opal in the size range 20–30 µm (the fraction was separated by sedimentation). However, to the authors' knowledge, this is the only reported case of damaging ASR with reactive silica particles in this size range (e.g., < 30 µm) and such behavior has not been confirmed by subsequent studies.

2.4.3 Pessimum Effect

Another phenomenon observed by Stanton (1940) in his landmark paper was that expansion did not necessarily increase with an increase in the reactive aggregate content. Figure 2.7(b) shows the expansion of mortar bars stored over water as a function of the proportion of reactive aggregate, in this case a siliceous magnesian limestone, in the mix. The maximum expansion occurred when 20% of the aggregate was comprised of the reactive sand, and the expansion decreases with greater amounts of reactive aggregate such that mixes containing 60% or more of reactive sand showed no significant expansion. This phenomenon is known as the pessimum effect, and such behavior is associated with some, but not all, highly reactive aggregates such as Beltane opal (an aggregate used in many earlier research studies on ASR).

Figure 2.8(a) shows pessimum behavior for six aggregates (five different types) containing opaline material, and Figure 2.8(b) shows the four regions of behavior described by Hobbs (1988), explained in Table 2.2.

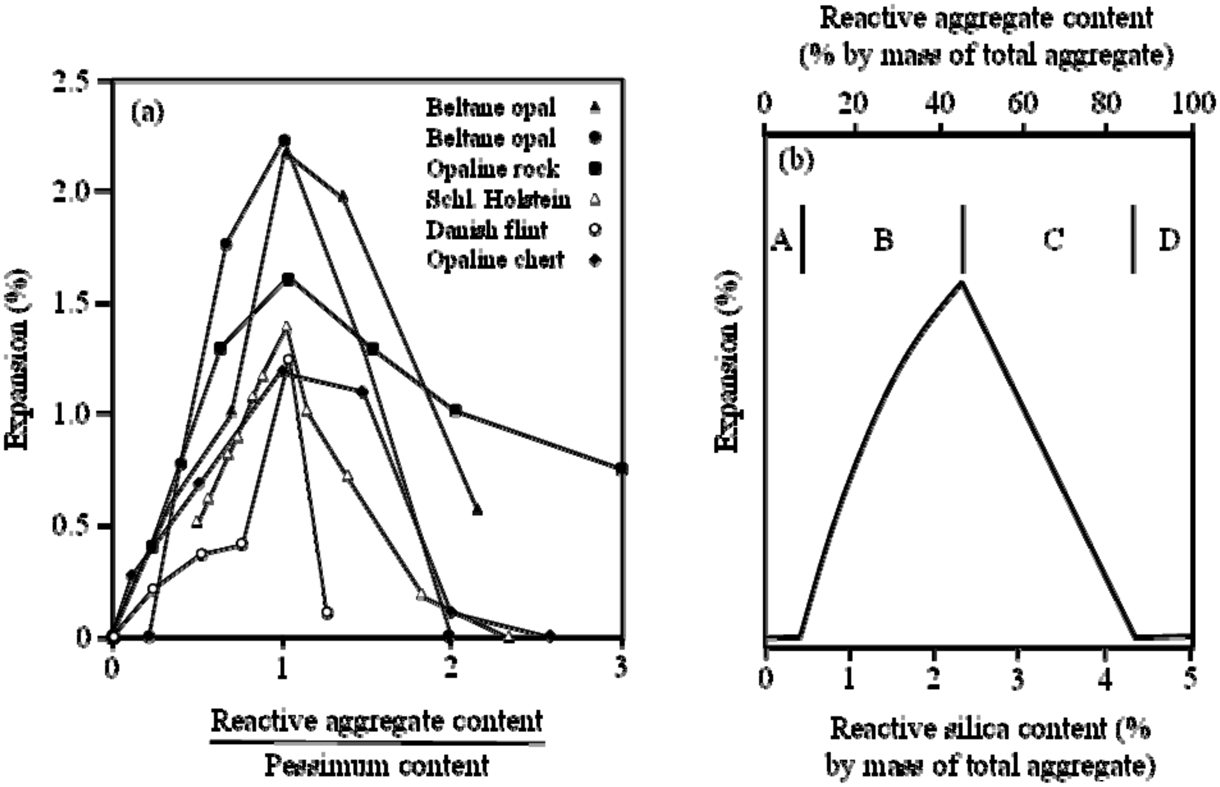

Figure 2.8. (a) Pessimum Behavior for Five Aggregates Containing Opaline Material and (b) Four Regions of Pessimum Behavior (from Hobbs 1988)

Table 2.2. Regions of Pessimum Behavior

Region	Effect	Explanation
A	Reaction but no cracking	Insufficient gel forms to cause damage.
B	Reaction, cracking – excess of alkalis	Expansion continues until all the reactive silica is depleted. Composition of gel is probably independent of the alkali/ reactive silica ratio.
C	Reaction, cracking – excess of reactive silica	Expansion continues until the alkali level is insufficient to sustain ASR. Alkali/silica ratio and swelling capacity of gel decreases with increasing silica content.
D	Reaction but no cracking	Reactive silica content very high and reaction so rapid that the gel forms before concrete has hardened sufficiently and there is no damage. Copious quantities of gel can be observed.

Hobbs' theory is based on two assumptions. Firstly, it is assumed that, as the reactive-silica-to-alkali ratio (SiO_2/Na_2Oe) in the system increases beyond the pessimum, the reaction product (gel) formed has a lower alkali-to-silica ratio (Na_2Oe/SiO_2) which reduces its affinity for water uptake and thus reduces its swelling capacity. Secondly, it is assumed that when the reactive silica content is very high, gel forms rapidly before the concrete has gained sufficient rigidity to suffer damage. The rapid gel formation consumes the alkalis and insufficient reaction occurs after the concrete has hardened to result in damage.

Dent Glasser and Kataoka (1981b) showed that quantity of silica dissolved depends on the ratio of silica to alkali (SiO_2/Na_2Oe). As discussed previously, in a system of reactive silica and alkali hydroxides, the concentration of silica in solution increases until the equilibrium solubility curve (SiO_2-pH) is reached as shown in Figure 2.3. If the silica content is low, the silica concentration increases until all of the silica is dissolved, and the solubility curve is not reached. If the silica content is very high, the concentration increases until the solubility curve is reached but then the excess silica in the mix removes hydroxyl ions (OH^-) from solution and reduces the pH. This leads to a reduction of the quantity of silica in solution. Consequently, there is an optimum silica to alkali ratio (SiO_2/Na_2Oe) where the maximum silica concentration is reached, and this occurs when the content of silica is such that it all dissolves when the solubility curve is reached. This is the pessimum reactive silica content with regards to expansion of mortar or concrete.

It is somewhat surprising that the pessimum effect is still observed when aggregates are tested using the accelerated mortar bar test (e.g., Shayan et al. 1988). In this test mortar bars are immersed in 1 M NaOH at 80°C (176°F). Despite this, expansion may not be observed with some aggregates in the test if the amount of reactive silica is above the pessimum level. It is difficult to explain such behavior on the basis of there being an excess of silica (that is, SiO_2/ Na_2Oe too high) since there is an inexhaustible supply of alkali.

2.5 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:

$$Na_2Oe = Na_2O + 0.658 \times K_2O \qquad \text{Eqn. 2.3}$$

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. The percentage of alkalis in portland cement is in the range of 0.2 to 1.3% Na_2Oe for most North American sources, but may be as high as 1.65% Na_2Oe or more worldwide.

2.5.1 Pore Solution Composition

Although the percentage of alkalis in portland cement is relatively low in comparison to other oxides, the alkalis are highly soluble and dominate the pore solution of the concrete. Figure 2.9 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^-. The pore solution is saturated with calcium (Ca^{2+}) but the concentration is very low, 0.6 to 2.5 mmol/L according to measurements made by Struble (1987), at the high pH resulting from the high concentration of alkali hydroxides.

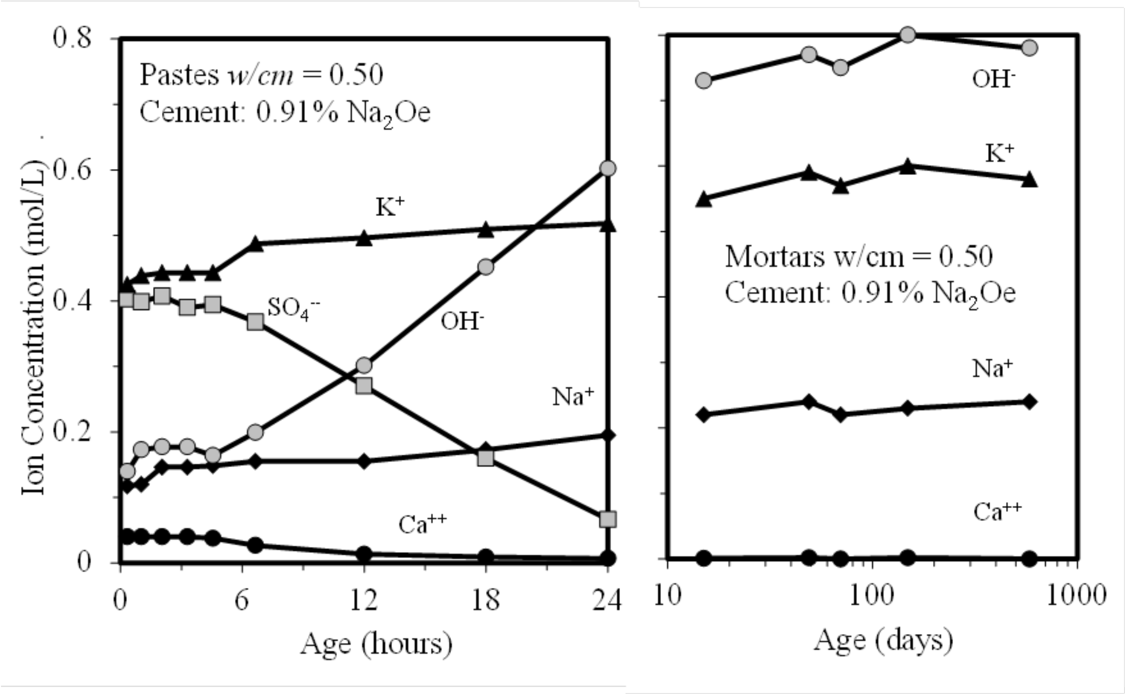

Figure 2.9. Pore Solution Composition in Portland Cement Pastes and Mortars (Diamond 1983)

Figure 2.10 shows that the alkali concentration of the pore solution is strongly influenced by the alkali content of the portland cement, the concentration being approximately 0.7 mol/L per 1% Na2Oe in the cement. Note in Figure 2.10 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 (calculated from the hydroxyl ion concentration assuming an activity coefficient of unity). The data in Figure 2.10 are for paste samples produced at a water-to-cement ratio of approximately w/c = 0.50.

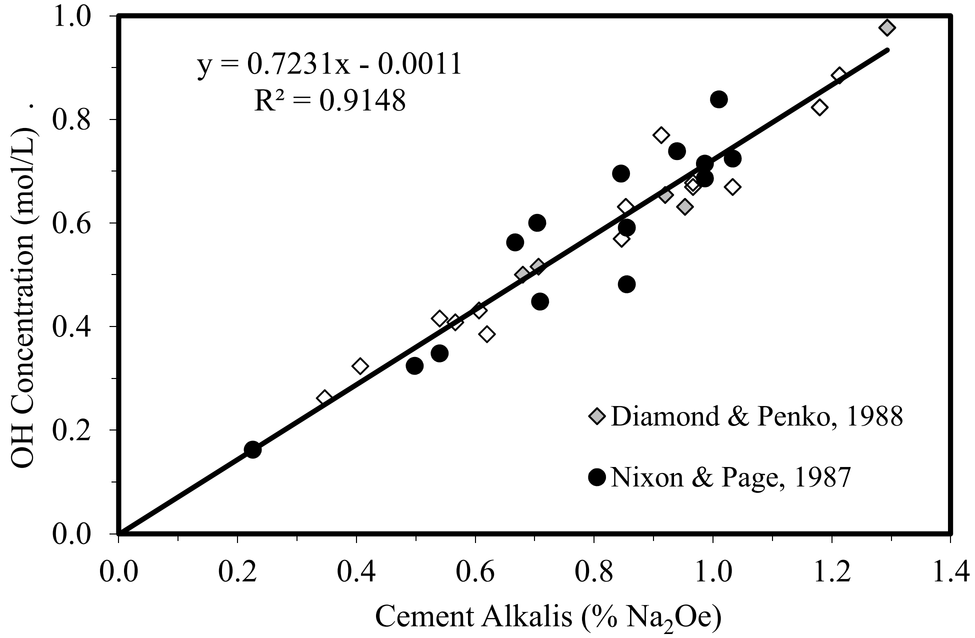

Figure 2.10. Relationship between the Alkalinity of the Pore Solution and the Alkali Content of the Portland Cement (Diamond and Penko 1992; Nixon and Page 1987)

A more general formula has been developed by consideration of the data in Figure 2.10 together with other published data; the expression is (Helmuth et al. 1993):

$[OH^-] = 0.339Na_2Oe\%/(w/c) + 0.022 \pm 0.06$ mol/L Eqn. 2.4

If reactive aggregate is present in the system the concentration of hydroxyl and alkali ions will decrease due to the alkali-silica reaction. Figure 2.11(a) shows the effect of reactive Beltane opal on the pore solution alkalinity of mortars (Diamond et al. 1981). After a brief induction period, alkalis are rapidly consumed by reaction with the opal until a steady-state is reached after approximately 1 month at 20°C; the hydroxide concentration is approximately 0.28M OH^- at this time. Tests at 40°C showed a rapid acceleration of the reaction, which is essentially complete in mortar containing Beltane opal after only 3 days (Diamond et al. 1981).

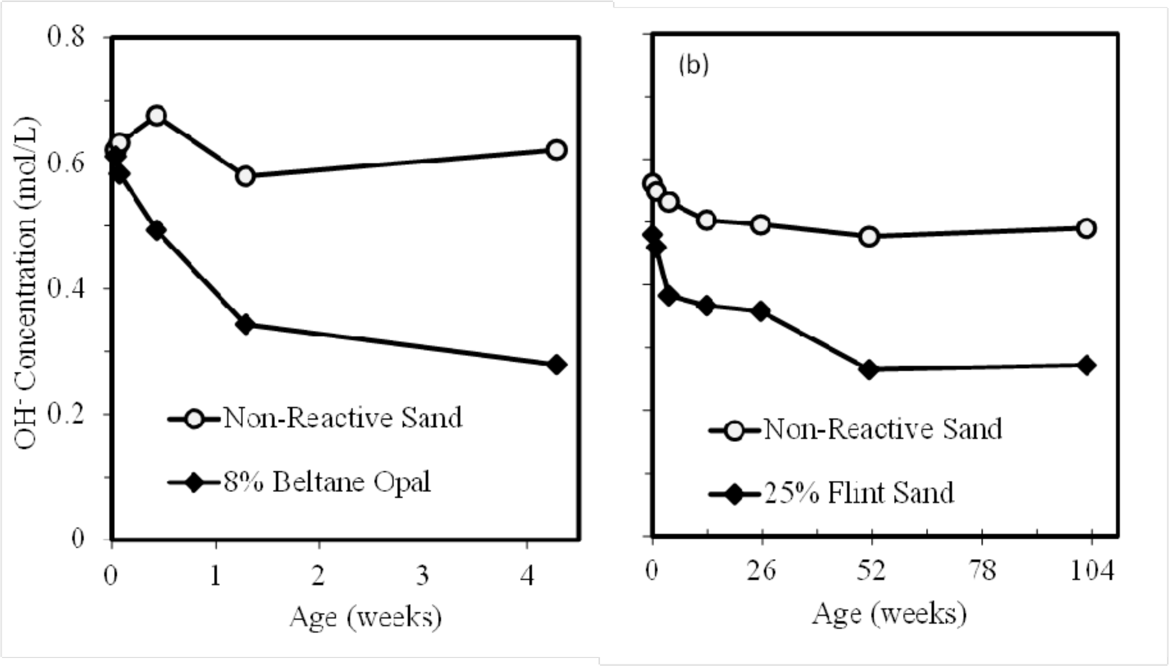

Figure 2.11. Effect of Reactive Aggregate on Pore Solution Composition in (a) Mortars (Diamond et al. 1981) and (b) Concrete (Thomas 1996)

Results from similar studies on concretes containing reactive flint sand (U.K.) are shown in Figure 2.11(b) (Thomas 1996). The pore solution of the control concrete showed some reduction in alkalinity with time, suggesting that there may be some interaction between the alkali hydroxides and the "inert" aggregate. However, this reduction is minimal compared to that observed in the concrete containing flint sand. The rate of consumption of alkalis due to the presence of flint was considerably slower compared with Beltane opal; steady-state conditions do not appear to be attained until after 1 year has elapsed. However, the hydroxyl ion concentration at this time (approximately 0.27M OH^-) is remarkably similar to the steady-state concentration observed with opal (approximately 0.28M OH^-). Diamond (1983) has postulated that the threshold alkali concentration required to sustain alkali-silica reaction is likely to be 0.25M OH^- or higher.

2.5.2 Threshold Alkali Contents

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_2Oe. Many specifications allow the use of potentially-reactive aggregates provided that the cement alkali content does not exceed 0.6% Na_2Oe. 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 2.12 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_2Oe). 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 2.12 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_2Oe. It should be noted that expansion has been found to occur in the field at lower alkali contents than that found necessary to cause expansion in concrete specimens stored over water 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 2.12 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_2Oe (MacDonald et al. 2012).

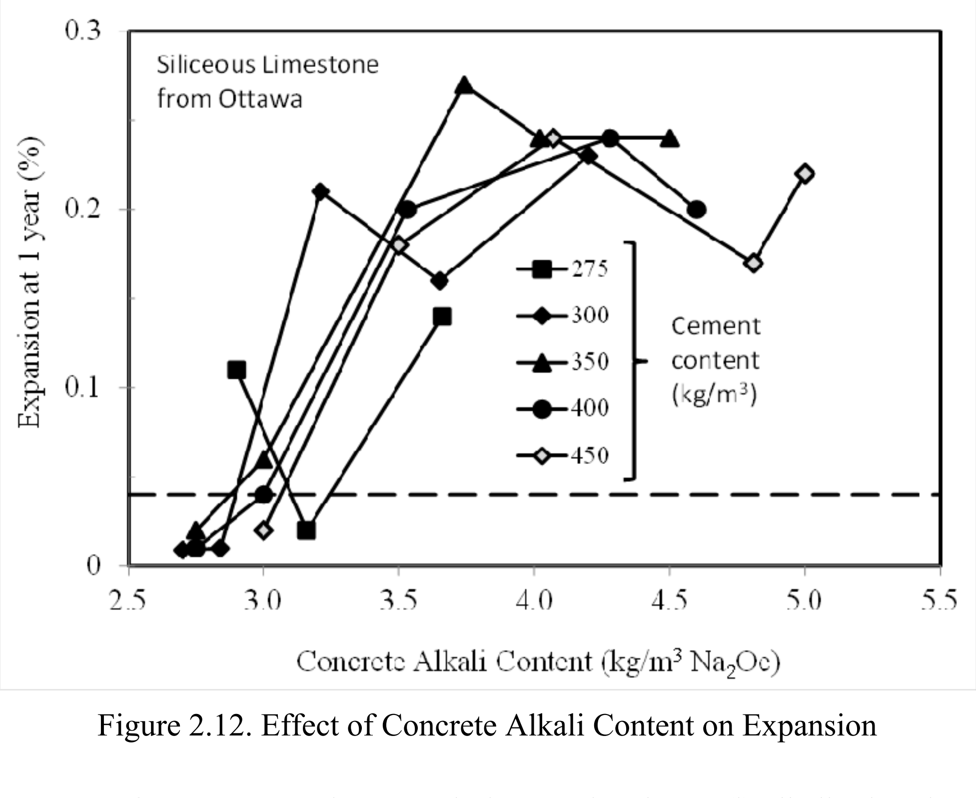

Figure 2.12. Effect of Concrete Alkali Content on Expansion

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.

2.5.3 Alkali Recycling

As discussed in previous sections of this chapter, alkali-silica reaction results in a reduction of the concentration of alkali ions (Na^+ and K^+) and hydroxyl ions (OH^-) in the concrete pore solution due to the formation of an alkali-silica gel ($CaO-Na_2O/K_2O-SiO_2-H_2O$) containing small amounts of calcium. There is evidence that a portion of the alkalis in the gel exchange for calcium with time, and alkalis are released back into the pore solution presumably fueling further reaction with the aggregate providing reactive silica is still available. This phenomenon was first proposed in Hansen in 1944 (Hansen 1944). Knudsen and Thaulow (1976) showed, using electron probe microanalysis of concrete, that gel formed within or close to aggregates was low in calcium and high in alkali, but that as the distance from the aggregate increased the calcium content of the gel increased, presumably due to close association with the calcium-rich cement paste.

Urhan (1987), summarizing observations from a number of studies, proposed that there is a progressive passage from low viscosity alkali-silica gel to C-S-H and that this is accompanied by a change in the physical and mechanical properties of the gel. Figure 2.13 illustrates changes in composition, physical and mechanical properties, and structure of the reaction product. Gel with a low calcium content may have a high swelling capacity, but the viscosity is very low. On the other hand, gel high in calcium is more rigid, but does not swell. Although not explicitly stated by Urhan (1987), at some point the composition of the gel is such that the gel has sufficient swelling capacity and viscosity to cause damage to the surrounding cement paste.

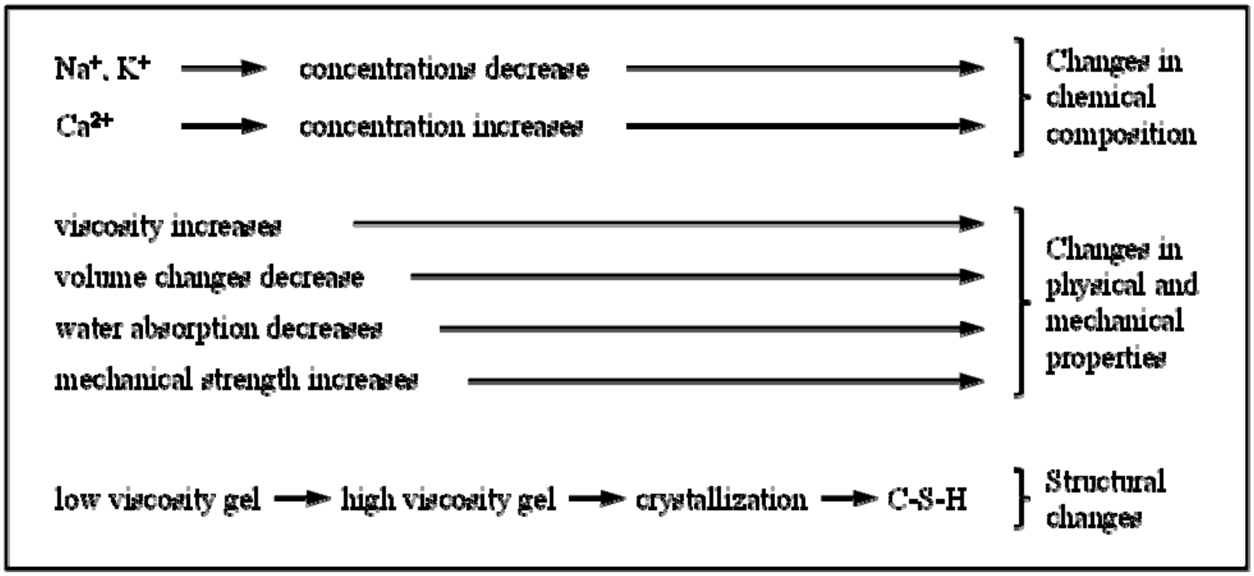

Figure 2.13. Changes in the Nature and Properties of the Alkali-Silica Reaction Product (Urhan 1987)

Thomas (2001) compared the composition of alkali-silica gels found in 7-year-old laboratory concrete and a 55-year-old concrete dam with the composition of calcium-silicate hydrate (C-S-H) in the concrete from the dam; the data expressed as alkali/silica and calcium/silica atomic ratios are presented in Figure 2.14. The composition covers a wide range, but it appears that there is a reasonable relationship between the alkali and calcium contents; i.e., as the calcium content decreases the potassium content decreases. This supports the concept of a cation exchange with the calcium replacing the alkali in the initially alkali-rich reaction product. This exchange occurs as the gel migrates away from the aggregate particle and comes into contact with the calcium-rich cement paste. The data in the figure indicate that the process continues slowly as the concrete ages and that the composition of the reaction product may ultimately approach something similar to C-S-H. This "final product" appears to retain very little alkali indicating that almost all of the alkali that originally participated in the reaction process may eventually be recycled to participate in further reaction.

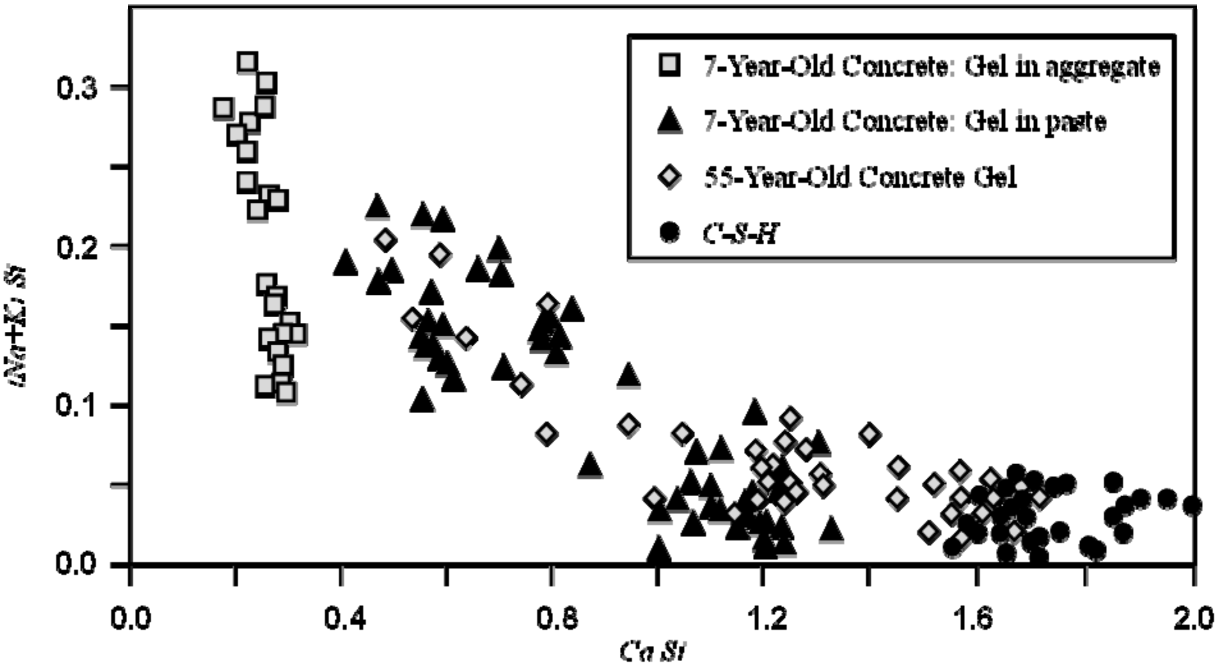

Figure 2.14. Composition of Alkali-Silica Gels in Concrete

In the same paper, Thomas (2001) compared the evolution of the pore solution extracted from concrete cubes containing a reactive flint aggregate with the expansion of concrete prisms from the same mixture. The data in Figure 2.15 show that the pore solution alkalinity drops steadily to an age of 12 weeks, after which it appears to remain stable with a hydroxyl ion concentration between 0.26 and 0.27 Mol/liter, whereas the expansion of the concrete continues long beyond the point at which the alkali concentration in the pore solution reaches a steady concentration.

This can be explained by alkali recycling, the alkalis being released from the gel being consumed by reaction, resulting in long-term expansion. Thomas (2001) suggested that alkali recycling might partially explain why in many large concrete dams the expansion continues for decades and does not appear to reach a maximum value as is the case for laboratory specimens (stored over water) where the expansion-time relationship is typically represented by an S-shaped curve reaching a maximum after a few years or less. In large structures, alkali recycling may continue to fuel ASR (until all the reactive silica is consumed), whereas in laboratory specimens leaching of the alkalis eventually reduces the alkali concentration to a level below that necessary to sustain ASR.

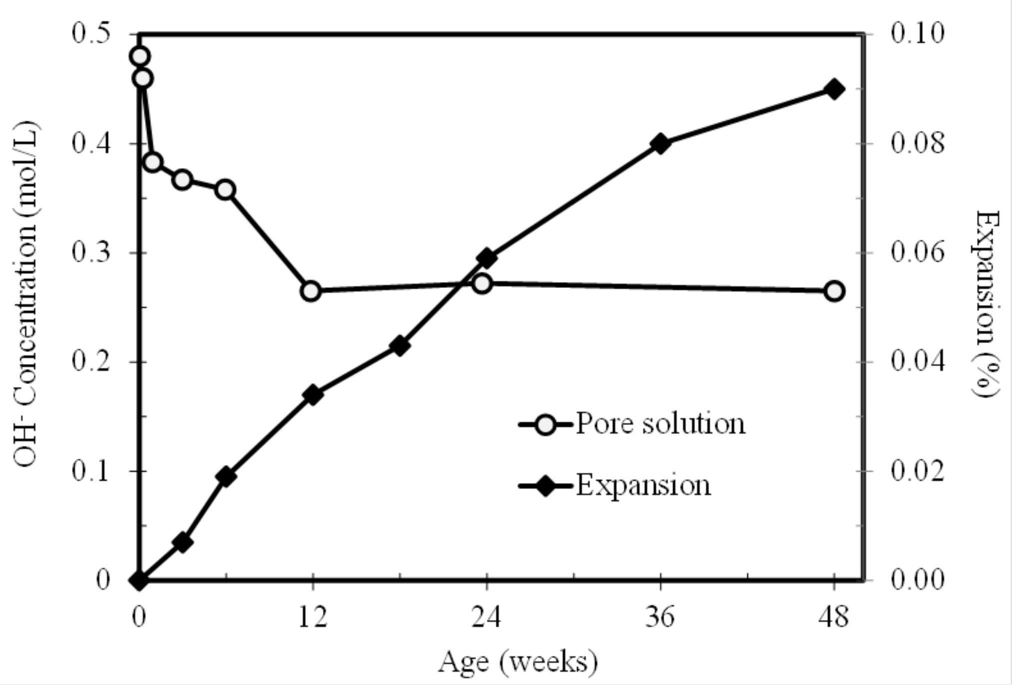

Figure 2.15. Evolution of the Pore Solution and Expansion in Concrete Containing Reactive Flint Sand (Thomas 2001)

2.6 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 2.16). 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.

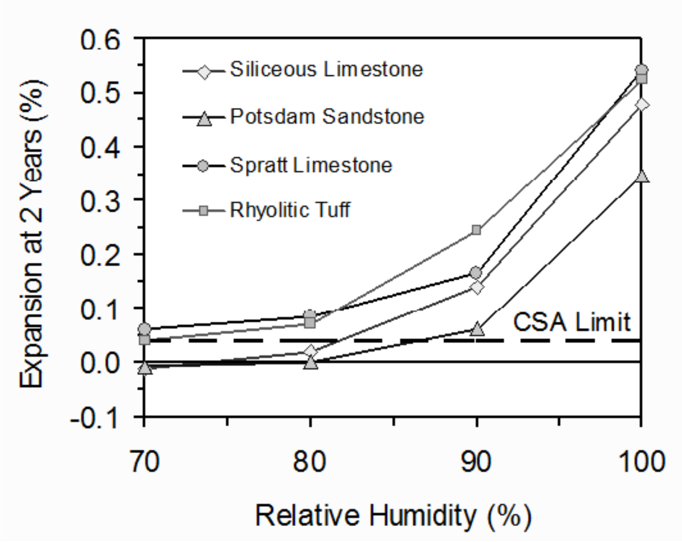

Figure 2.16. Effect of Relative Humidity on Expansion of Concrete Prisms (Pedneault 1996)

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3 – Symptoms of Alkali-Silica Reaction (ASR)

3.1 INTRODUCTION

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^+-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 (Figures 3.1A & 3.1B). However, drying shrinkage, freezing/thawing cycles, and sulfate attack can also result in a pattern of cracks showing a random orientation.

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). 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).

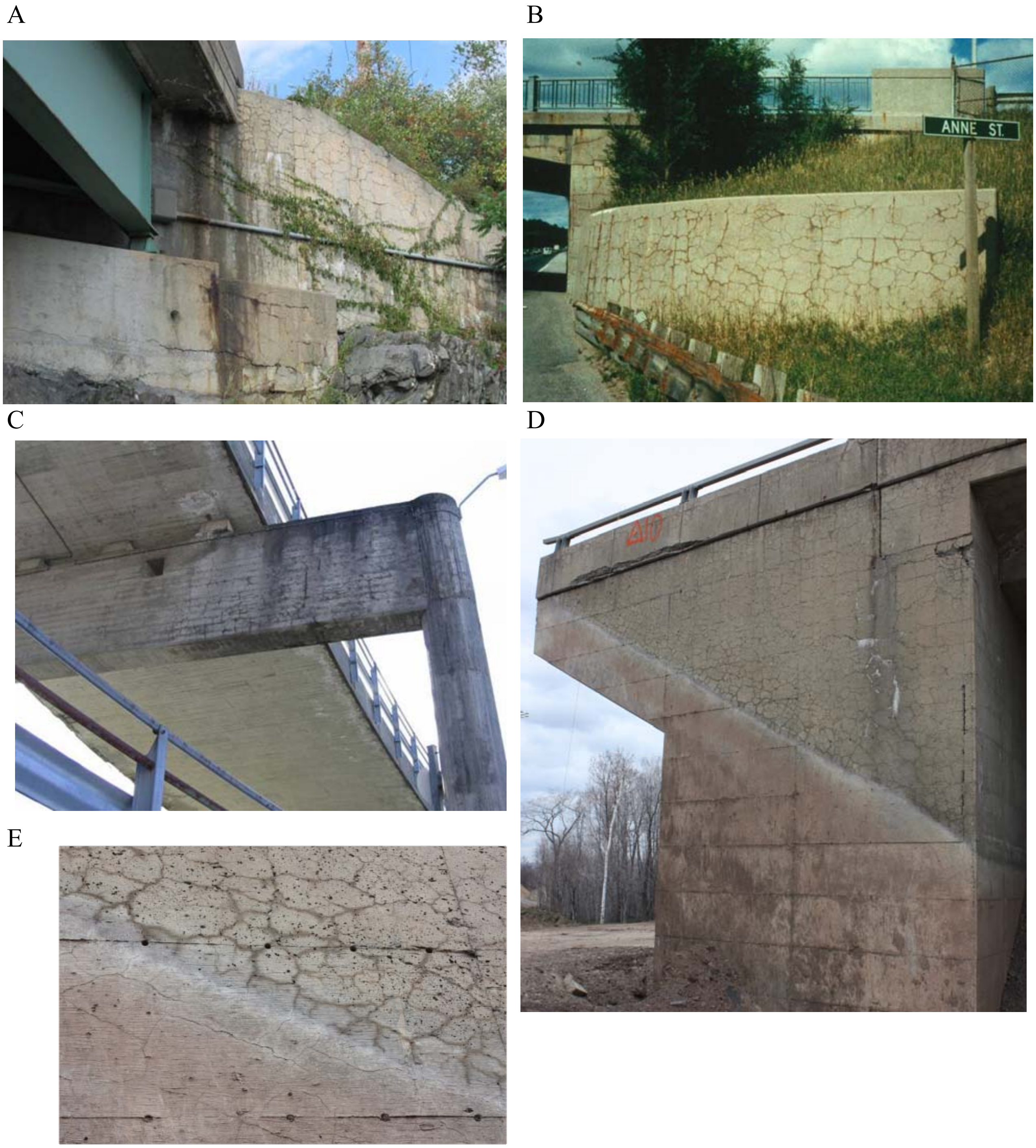

Figure 3.2. A&B. Map-cracking in wing walls (exposed elements) of bridge structures affected by ASR. C. Cracking in the column and the exposed portion of a beam affected by ASR. D&E. Cracking in the exposed portion (above ground) of the wing wall of a bridge structure affected by ASR; the wing wall has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

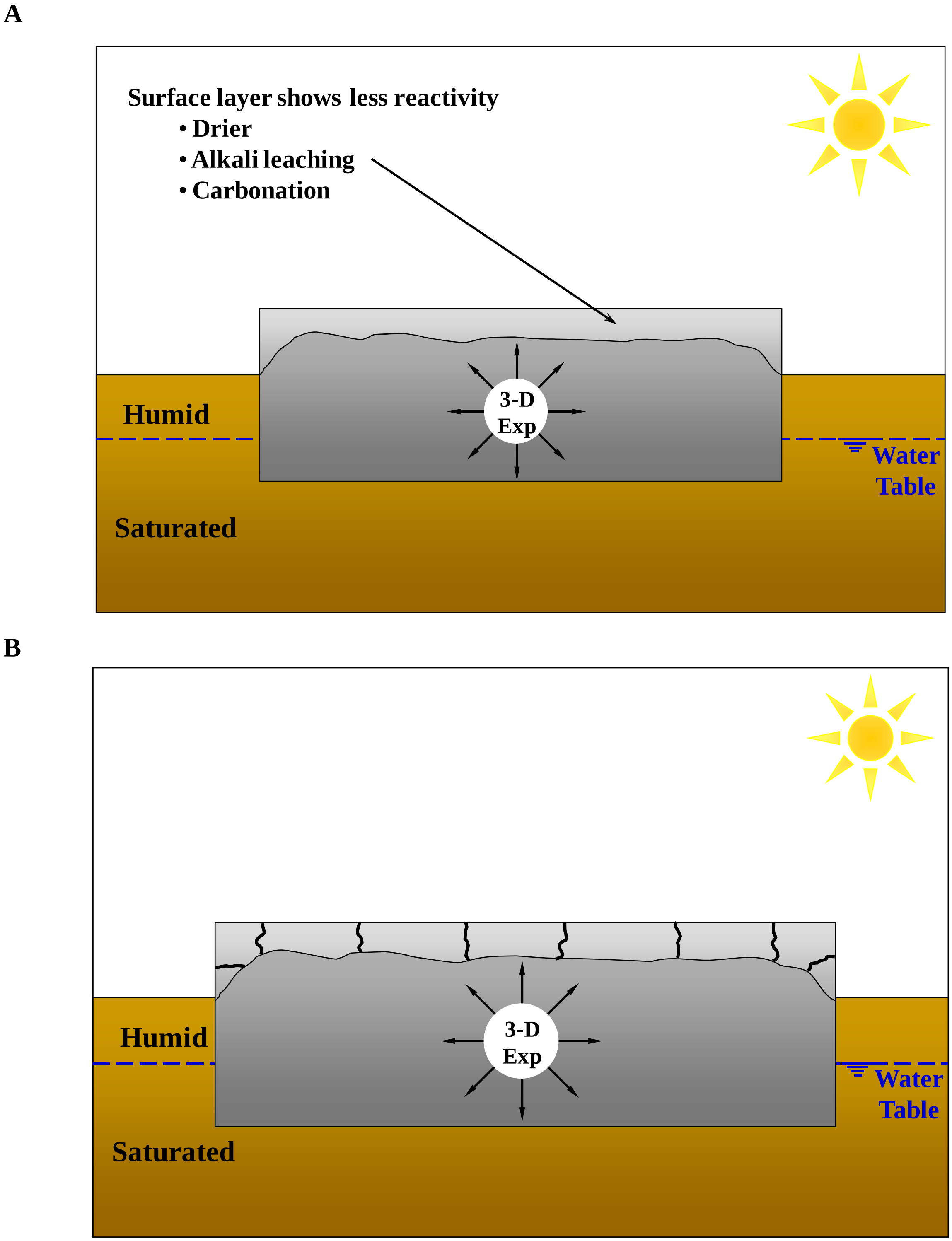

Figure 3.3. Concrete member undergoing ASR and experiencing cyclic exposure to sun, rain and wind show more severe surface cracking resulting from induced tension cracking in the “less expansive” (due to alkali leaching/dilution processes, variable humidity conditions, etc.) surface layer under the expansive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

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 3.4);
- deflection, closure of joints with associated squeezing/extrusion of sealing materials, and, ultimately, spalling of concrete at joints (Figure 3.5).

Figure 3.4. A. Relative movement of abutting sections of parapet wall in a bridge structure affected by ASR (Stark 1991). B-D. Expansion of concrete pavement due to ASR (overlaid with asphalt), which pushes against the adjacent building foundation causing shearing of concrete columns. E. Concrete sidewalk made with alkali-carbonate reactive limestone aggregate. Originally, the sidewalk and the curb were adjacent to each other; however, the expansion of the sidewalk created a gap that had to be filled with asphalt.

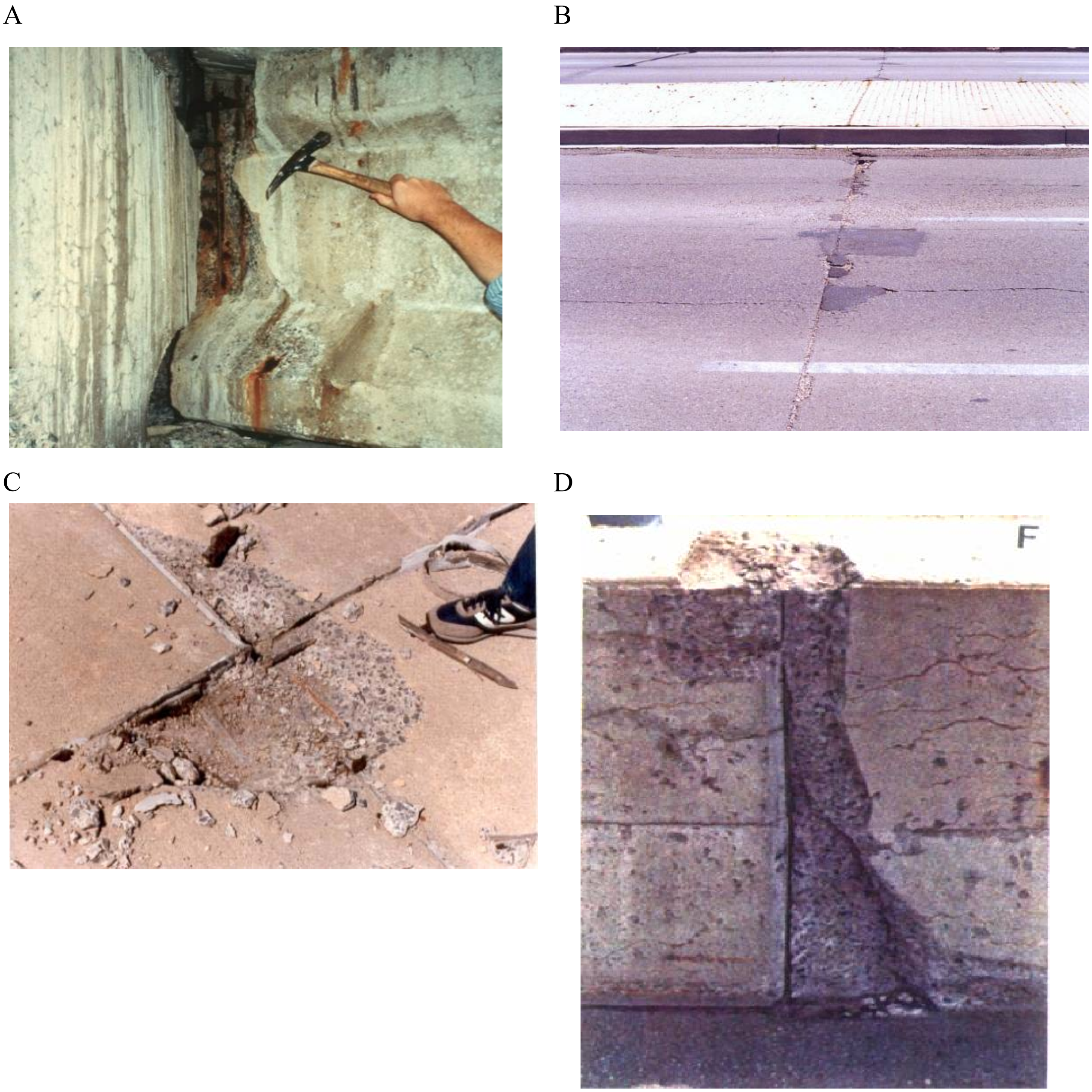

Figure 3.5. A. Expansion of bridge girder leading to loss of clearance between the girder and embankment and eventually crushing of the girder end with localized spalling. B&C. Expansion causing spalling at joints in concrete pavement sections incorporating highly-reactive aggregates. D. Expansion with 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 countries (Figure 3.6A). 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 3.6B).

Figure 3.6. 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 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).

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4 – Test Methods

4.1 INTRODUCTION

Ever since Stanton (1940) reported his discovery of alkali-aggregate reaction (AAR) in California, there has been keen interest in laboratory tests to (1) predict whether a given aggregate will cause expansion and cracking in concrete and (2) to evaluate preventive measures to allow for safe use of those aggregates found to be potentially reactive. This chapter 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. In addition to test methods aimed at assessing aggregate reactivity and preventive measures, this chapter also includes discussion on alkali release from aggregates and SCMs. There are no standardized tests that adequately address this important issue of alkali release, but for completeness, relevant information and recent trends are presented that should provide some useful insight to readers.

4.2 ASR TEST METHODS FOR EVALUATING AGGREGATE REACTIVTY

Stanton (1940) was not only the first to discover ASR in field structures, but he was also the first researcher to develop a test method to assess aggregate reactivity, and he used this technique to also evaluate the use of pozzolans to control ASR-induced expansion. The method developed by Stanton, which is essentially the same as the current ASTM C 227 test method, is still in use today by some researchers and practitioners, but a wide variety of test methods have been developed and implemented since the time of Stanton's pivotal research on ASR. Some of these test methods 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. This chapter 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. Discussion on each of these methods follows, with particular emphasis placed on the Accelerated Mortar Bar Test (AMBT) and Concrete Prism Test (CPT) as these two are the most commonly used ASR test methods and 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 developed under FHWA Project DTFH61-06-D-00035.

4.2.1 ASTM C 295: Standard Guide for Petrographic Examination of Aggregates for Concrete

A detailed petrographic examination, following ASTM C 295, provides useful information regarding the potential reactivity (with respect to both ASR and ACR) of a given fine or coarse aggregate. Within ASTM C 295 various techniques are employed that allow for the identification and quantification (through point count) of minerals that have been known to lead to expansion and cracking in concrete containing such aggregates. The specific minerals that are identified and quantified in ASTM C 295 are:

- Opal
- Chalcedony
- Cristobalite
- Tridymite
- Highly Strained Quartz
- Microcrystalline Quartz
- Volcanic Glass
- Synthetic siliceous Glass.

The minerals listed above are found in a range of aggregate types, as per ASTM C 295, including:

- Chert
- Gneiss
- Gneissic Granite
- Graywacke
- Phyllites
- Schist
- Vein Quartz
- Sandstone
- Quartzite

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., AASHTO T 303 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_20_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 – this long duration limits its use by many agencies and owners. • Cannot be used to determine the alkali threshold for a given aggregate due to leaching of alkalis from the prisms during the course of the test.

The identification and quantification of reactive phases within aggregates are accomplished following ASTM C 295 through the use of optical methods (using reflective and transmitted light) and may be aided with the complementary use of x-ray diffraction (XRD) and scanning electron microscopy (SEM). Most of the minerals that cause ASR can be identified and quantified, but some finely divided forms of quartz are not visible using a petrographic microscope. For example, some siliceous limestones contain less than 5 percent undetectable (using petrographic microscope) finely divided quartz particles, which can cause ASR-induced expansion in concrete (Fournier and Bérubé 1991). As such, caution should be taken in classifying a given aggregate as being non-reactive based solely on petrographic evaluation – information from other laboratory tests and/or field performance can be combined with such petrographic data/information to more accurately determine whether an aggregate may cause deleterious expansion in field structures.

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

ASTM C 289, often referred to as the "Quick Chemical Test," is a test that involves crushing an aggregate (passing the 300 µm sieve and retained on the 150 µm sieve), soaking it in 1 N NaOH solution for 24 hours at 80°C and measuring the amount of silica that goes into solution and the alkalinity at the completion of the test. The results of the test, specifically the amount of silica that went into solution and the reduction in the alkalinity of the soak solution, are plotted on a graph, contained in the Appendix to ASTM C 289. Based on how the data (amount of silica dissolved and reduction in alkalinity) plots within this graph, the aggregate is classified as innocuous, potentially deleterious, or deleterious.

Although ASTM C 289 is still used by some researchers and practitioners, experience has shown that it is generally not an accurate predictor of aggregate reactivity, with many aggregates showing discrepancies between the classification (innocuous, potentially deleterious, or deleterious) obtained from ASTM C 289 and the performance of such aggregates in the concrete prism test (ASTM C 1293) or in actual field structures (Hooton 1990; Grattan-Bellew 1989). The extreme severity of this test, specifically the high temperature and high alkalinity soak solution in which the aggregates are placed, coupled with the high surface area of the sample as a result of crushing, will cause some aggregates to be identified as reactive, even though such aggregates perform satisfactorily in ASTM C 1293 and in field structures (Bérubé and Fournier 1993). On the other hand, some reactive aggregates may lose most or all of their reactive phases during crushing and sieving and would thus be found to be innocuous per ASTM C 289, even though concrete containing such aggregates would exhibit significant expansion and cracking in the laboratory and field (Bérubé and Fournier 1992). Because of the above issues, ASTM C 289 is not recommended as a test method for evaluating aggregate reactivity, even though the test is still being used by some agencies and owners.

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

ASTM C 227 is essentially the method Stanton (1940) developed of storing mortar bars (25 mm x 25 mm x 285 mm) over water at 38°C to provide for a high-temperature, high-humidity atmosphere within a sealed container. Because of the small cross section of the mortar bars used in this test, coarse aggregates must be processed by crushing, sieving, and washing in order to produce particles small enough to be used in the mortar bars. The aggregate to cement ratio is specified to be 2.25 (by mass), and there is no specific requirement for the cement to be used in the test. The ASTM standard specification for concrete aggregates (ASTM C 33) classifies an aggregate as being reactive when a given aggregate expands by more than 0.05% at 3 months or 0.10% at 6 months.

There are several technical problems associated with this test method, some of which spurred the development of more accurate methods for evaluating aggregate reactivity. The most significant issue with this test is that due to the small specimen cross section, appreciable amounts of the alkalis within the mortar bars leach out and end up residing in the water in the bottom of the container or being absorbed by the wicking material that lines the side of the container (Swenson 1957). Slowly-reactive aggregates are particularly sensitive to the leaching issue as the alkalis may leach from the bars before the aggregates ever have the opportunity to react and expand. Some slowly-reactive aggregates that have been observed to pass this test but that will fail more accurate tests and/or that will exhibit expansion and cracking in field structures include greywackes, argillites, quartzites, and some gneisses (Grattan-Bellew 1978; Stark 1980). Another issue with this test is that only fine aggregates can be tested so coarse aggregates have to be crushed, graded, and washed, which may alter the textural nature of the aggregates or remove some reactive phases.

Given the technical issues associated with this test, specifically that a wide range of known, reactive aggregates will pass this test, it is not recommended as a test method for evaluating aggregate reactivity.

4.2.4 AASHTO T 303 (ASTM C 1260): Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)

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:
< 0.10% is considered innocuous
0.10 to 0.20% is considered potentially reactive
> 0.20% is considered reactive

CSA A23.2-25A expansion criteria:
> 0.15% (0.10% for limestones) is considered reactive
< 0.15% (0.10% for limestones) is considered non-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, sometimes with expansions as high as 0.25 percent at 14 days, even though they have performed well in concrete prism testing (ASTM C 1293) and in field applications (Bérubé and Fournier 1992). In fact, recent, unpublished work performed by the authors of this Facts Book included the evaluation of an aggregate from Oahu that when tested according to AASHTO T 303 expanded greater than 0.40 percent at 14 days but easily passed ASTM C 1293. Because AASHTO T 303 has been shown to fail a significant number of aggregates, especially natural sands and gravels, an aggregate should not be rejected based solely on the test results.

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). Arrieta (2012) reported that the primary reason the aggregates tested by Folliard et al. (2006) passed the AASHTO T 303 test but failed ASTM C 1293 was related to the pessimum effect. Specifically, the primary reactive mineral was found petrographically to be chert, which is an aggregate known to exhibit a pessimum effect. The concept of the pessimum effect is shown graphically in Figure 4.1 (adapted from Poole (1992) by Arrieta (2012)). According to Poole (1992), the pessimum behavior proposes that ASR will cease when either the alkalis in the pore solution or the reactive constituents within the aggregate particles are depleted. If the content of the reactive mineral (chert, in this case) is too low compared to the availability of alkalis, very little gel will form and hence expansion will be limited. On the contrary, if there is an overabundance of reactive minerals compared to the availability of alkalis, the alkalis will become depleted and only a small amount of gel with form, with minimal expansion observed. When an optimum ratio of reactive silica to available alkalis is achieved (50 percent for the illustration in Figure 4.1, and about 10-12 percent for most cherts), expansion will be at a maximum.

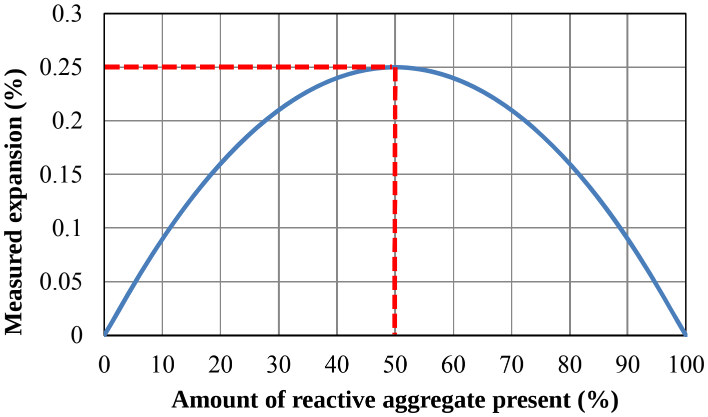

Figure 4.1. Graphical Representation of the 'Pessimum' Behavior Concept (adapted from Poole (1992) by Arrieta (2012))

For the aggregates studied by Folliard et al. (2006) and evaluated in more detail by Arrieta (2012), AASHTO T 303 was modified by replacing the pessimum aggregate with varying amounts of non-reactive sand. Figure 4.2 shows the typical response for an aggregate from Central Texas. Based on petrographic evaluations, it was found that the chert content in this pessimum aggregate was approximately 75% of the as-received coarse aggregate. Figure 4.2 takes the chert content into account, along with the level of replacement by the non-reactive aggregate, to plot the approximate chert content on the x-axis versus the expansion observed after 14 or 28 days storage in 1 N NaOH solution at 80°C. As shown in this figure, the pessimum proportion was found to be approximately 10 percent chert, which is consistent with past studies (Nixon et al. 1989; Hudec 1990). These results are encouraging in that they show that AASHTO T 303 can be used to not only identify pessimum aggregates but also to determine the actual pessimum proportion. Also, the results help to explain why these coarse aggregates failed the concrete prism test. The coarse aggregate content used in ASTM C 1293 is 70 percent of the overall aggregate content. Assuming that 75 percent of the coarse aggregate is comprised of flint, the total amount of flint in the mixtures was about 52 percent, whereas in the unmodified AASHTO T 303 test, the coarse aggregate content was 100 percent, hence resulting in a chert content of 75 percent. Referring to Figure 4.2, the higher the chert content (beyond the pessimum proportion of about 10 percent), the lower the expansion. Only when the chert content was reduced by replacing the reactive sand with a non-reactive sand did expansion exceed the 0.10 percent expansion limit typically specified for AASHTO T 303.

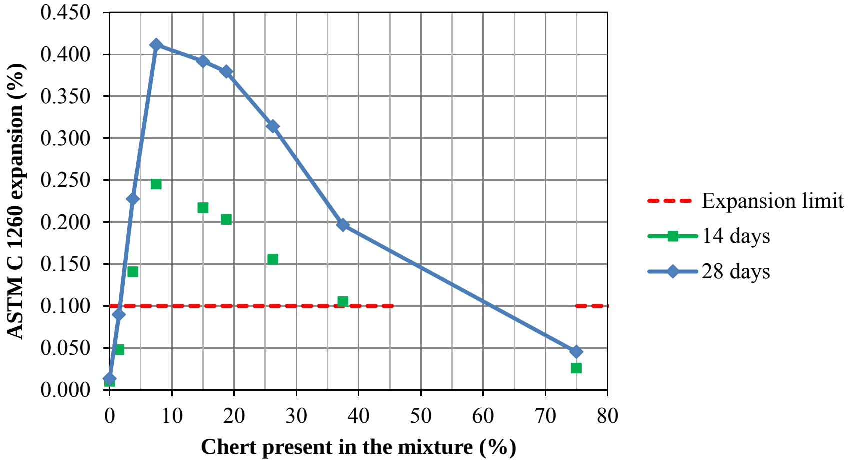

Figure 4.2. Pessimum Graph for Coarse Aggregate from Central Texas, Expressed as Chert Percent, when using AASHTO T 303 Testing Regime (Arrieta 2012)

In summary, AASHTO T 303 is a rapid test that in some cases yields results that agree with the results from ASTM C 1293 tests or field experience. However, as described herein, there are a wide range of aggregates that will give erroneous and misleading results in AASHTO T 303, when compared to the more accurate and realistic concrete prism test. As such, 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.

4.2.5 ASTM C 1293: Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction

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 chapter, 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 PP65.

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 shown in Figure 4.3. In this figure, the concentration of alkalis in the water at the bottom of the bucket (above which the prisms are stored) is shown as a function of time for a selected mixture following ASTM C 1293. Taking into account the volume of water at the bottom of the bucket and ignoring that some of the alkalis may be absorbed into the wicking material on the side of the bucket, Thomas et al. (2006) calculated the total amount of alkalis that leached from the prisms, arriving at the values shown above (20 percent after 90 days and 35 percent after one year). As such, higher alkali contents are needed to cause expansion in the CPT than what is actually needed in field concrete or exposure blocks, which also limits the use of the CPT to estimate the alkali threshold for a given aggregate, as discussed in more detail next.

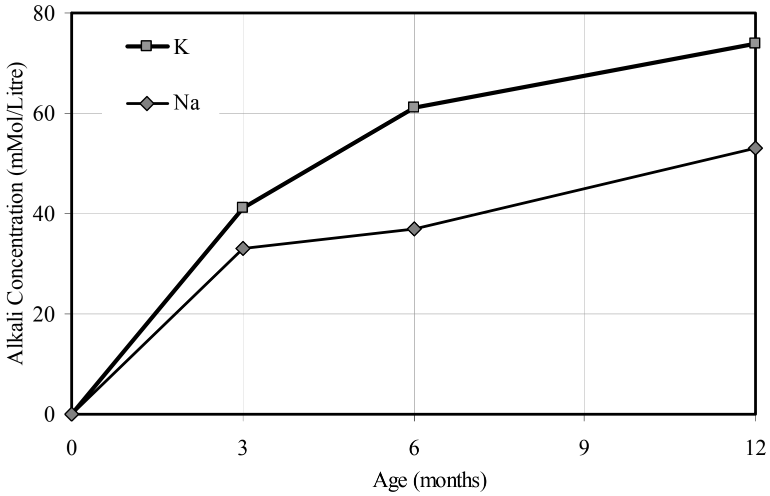

Figure 4.3. Concentration of Alkalis Measured in the Water at the Bottom of an ASTM C 1293 Container as a Function of Time (after Thomas et al. 2006)

The effects of alkali leaching from concrete prisms tested during the course of an ASTM C 1293 test were 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_20_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	**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.

Figure 4.4 shows the relationship between expansion after one year using the standard CPT at 38°C and the expansion after 13 weeks using the accelerated CPT at 60°C (Ideker et al. 2010) for a range of aggregates (note all mixtures are plain concrete). A test duration of 13 weeks is based on previous research by the same authors where it was observed that most aggregates have exhibited most of their expansion by this point (see Figure 4.5). In addition, other researchers have proposed that expansions at 13 weeks in the accelerated CPT correlate best with one-year expansions in the standard CPT when testing plain concrete (without SCMs or lithium-based admixtures).

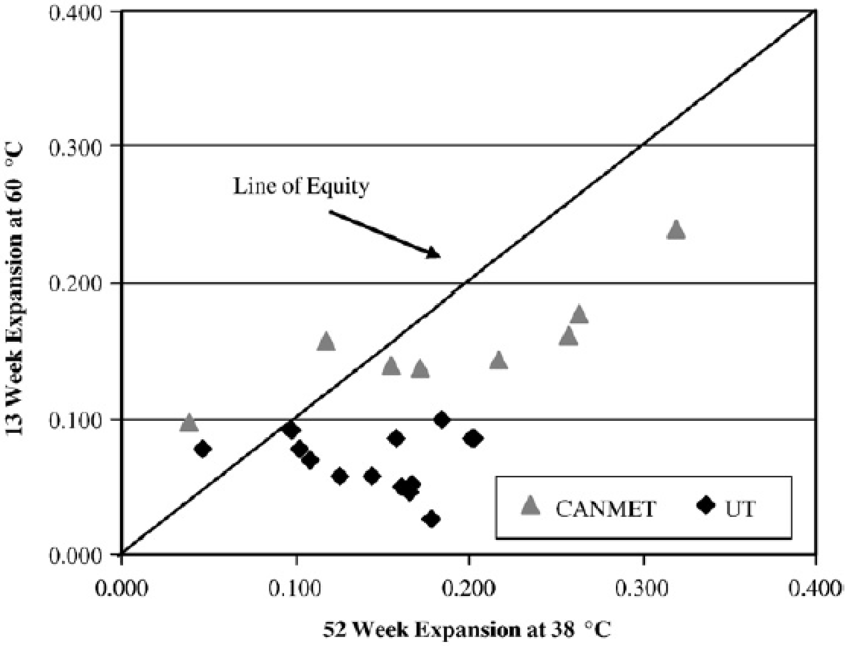

Figure 4.4. Relationship between Expansion after One Year Using the Standard CPT (38°C) and the Expansion after 13 Weeks Using the Accelerated CPT (60°C) (Ideker et al. 2010)

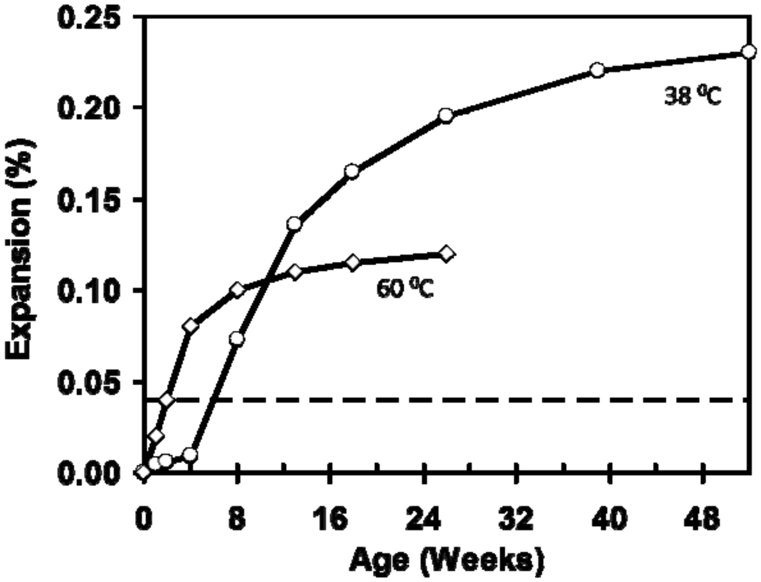

Figure 4.5. Expansion of Concrete Prisms Containing Spratt Limestone and Stored above Water at 38°C vs. 60°C (Folliard et al. 2006)

Referring to Figure 4.4, it is quite evident that most of the data points are well below the line of equity, which represents expansion values that are identical for both test methods/durations. As such, the general trend is that expansion values obtained at 60°C are significantly lower than those at 38°C for the aforementioned reasons. For some of the aggregates tested by Ideker et al. (2010), the results of testing at the two different temperatures yielded similar classifications when using a 0.04 percent expansion limit, meaning that some aggregates "passed" both tests or "failed" both tests. From a specification point of view, this may be acceptable in that both versions of the CPT would conclude that the aggregate is either reactive or non-reactive (and such requiring preventive measures). However, there were also aggregates that yielded conflicting results when they were tested at the two different temperatures, with the classification being "fail" at 38°C and "pass" at 60°C.

Because the two versions of the CPT can produce conflicting conclusions with regard to a 0.04 percent expansion criteria, caution should be taken when considering the accelerated CPT, by itself or in parallel to the standard CPT, for a given application or project.

4.3 ASR TEST METHODS FOR EVALUATING PREVENTIVE MEASURES

When considering the use of reactive aggregates, it is essential that test methods exist that can determine the safe level or dosage of a given product to prevent or substantially reduce ASR-induced expansion and cracking. This section will discuss and critique the most commonly used test methods that aim to achieve this goal.

Table 4.3 details the most common test methods used to evaluate preventive measures, such as SCMs and lithium-based admixtures. Specific details are listed in the table for each test, but additional 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.

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^{TM} 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
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
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.

4.3.1 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

This test method was developed by U.S. Bureau of Reclamation in the 1940s to evaluate SCMs (known as siliceous admixtures or pozzolans at the time) for the construction of the Davis Dam and other large projects (Gilliland and Moran 1949). The test is essentially the same as ASTM C 227, where small specimens (25x25 mm in cross section) are stored above water at 38°C, with one major difference – Pyrex (borosilicate) glass is used as a “model” reactive aggregate. The objective of the test is to have somewhat of an internal standard, Pyrex glass, that can be used to evaluate the relative efficacy of SCMs in reducing expansion. Specifically, the expansion after 14 days of storage above water at 38°C of mortar bars containing a high-alkali cement (0.95 to 1.05% Na_2O_e) and either 25% fly ash or 50% slag cement is compared to the expansion of control mortar bars (portland cement only). The percentage reduction in expansion is then calculated. Although the test specifies the use of either 25% fly ash or 50% slag cement, other SCMs and dosages may also be used. Various ASTM specifications allow for the use of ASTM C 441 to assess the efficacy of SCMs in controlling ASR-induced expansion, as summarized in Table 4.4.

Table 4.4. ASTM Materials Specifications and Expansion Limits for ASTM C 441

Specification	**Limit**
ASTM C 618: Fly Ash and Natural Pozzolans	14-day expansion of cement/fly ash blend ≤ expansion of control mix with low-alkali cement (≤ 0.60% Na_2Oe)
ASTM C 989: Ground Granulated Blast -Furnace Slag	If job cement and proportions of cement and slag are known: 14-day expansion ≤ 0.020% Otherwise: 14-day expansion of cement/slag blend ≤ 25% of the expansion of control mix with the high-alkali cement only (i.e., slag should reduce expansion by at least 75% compared with the control)
ASTM C 1240: Silica Fume	14-day expansion of blend of 90% cement and 10% silica fume ≤ 20% of the expansion of control mix with the high-alkali cement only (i.e., 10% silica fume should reduce expansion by at least 80% compared with the control)
ASTM C 595: Blended Hydraulic Cement	14-day expansion of blended cement ≤ 0.020% and 56-day expansion of blended cement ≤ 0.060%
ASTM C 1157: Hydraulic Cement (Option R)	14-day expansion of hydraulic cement ≤ 0.020% and 56-day expansion of hydraulic cement ≤ 0.060%

Although ASTM C 441 is referenced by all of the materials specifications shown in Table 4.4, this test method has numerous technical issues and has fallen out of favor with most agencies and owners. The test inherently suffers from the same major flaw described earlier for ASTM C 227, specifically leaching of alkalis from the mortar bars. But in addition to this, the use of Pyrex glass adds other complexities and problems that have led to high variability in test results. Although Pyrex glass was selected with the intention of it being a “model” or “standard” reactive aggregate, several issues with the glass have limited the widespread use of ASTM C 441 to evaluate SCMs, in favor of ASTM C 1567 (described next) and ASTM C 1293. Pyrex glass varies in its composition and particle size from source to source. Further, Pyrex glass, itself, contains relatively high alkali contents, which are released during the course of the test, and different sources of Pyrex glass will vary in alkali content. Hobbs (1989) showed that the amount of SCMs needed to reduce expansion triggered by Pyrex glass does not correlate well with the SCM content needed to suppress expansion from natural aggregates used in concrete construction. Because of the technical issues with ASTM C 441, the high level of variability of test results, and the lack of correlation between Pyrex glass and natural aggregates, this test is not recommended as a means of evaluating SCMs for use in controlling ASR in concrete.

4.3.2 ASTM C 1567: Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)

ASTM C 1567 is essentially identical to AASHTO T 303, with the one exception that it is aimed at evaluating the use of SCMs. The test utilizes the same test specimens and storage conditions as AASHTO T 303 and is, as such, 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 4.6, 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.

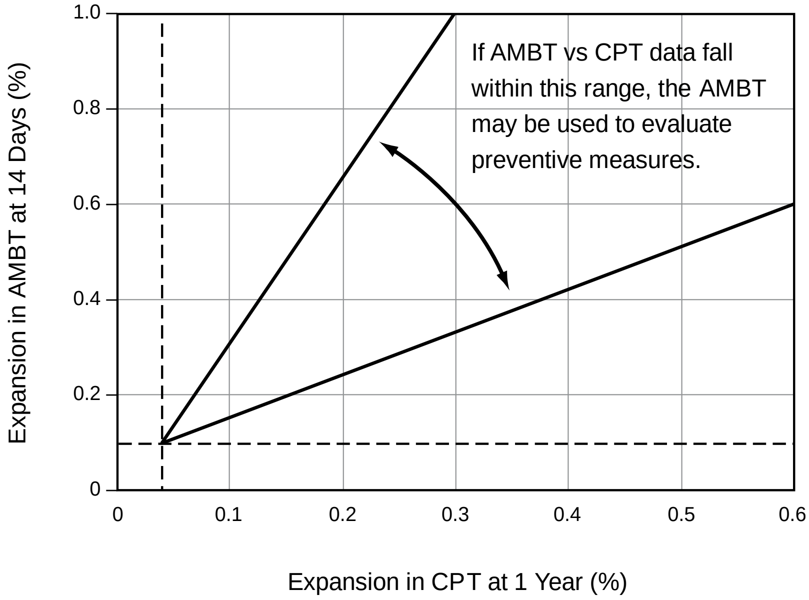

Figure 4.6. Comparison of AMBT and CPT Data for the Purpose of Determining Whether the AMBT is Suitable for Evaluating Preventive Measures with a Specific Aggregate (after AASHTO PP65-11)

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 4.7. This figure illustrates that similar SCM dosages are needed in each of these tests to reduce the expansions below the aforementioned limits.

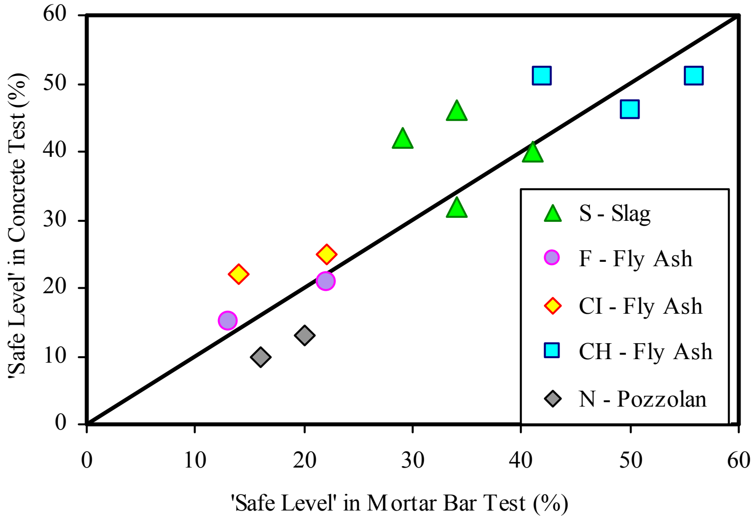

Figure 4.7. Comparison of SCM Dosage Needed to Reduce Expansions below Test Limits for ASTM C 1567 and ASTM C 1293 (after Thomas and Innis 1998)

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 4.8; 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).

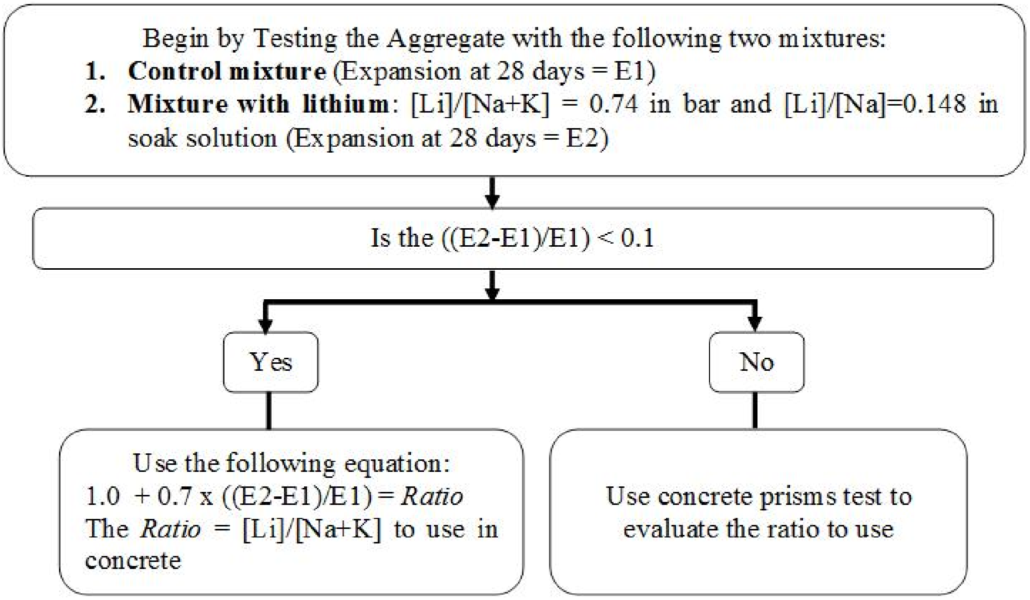

Figure 4.8. Approach to Using the Modified Accelerated Mortar Bar Test to Determine the Dosage of Lithium Nitrate (expressed as [Li]/ [Na+K]) to Adequately Suppress Expansion in Concrete (Tremblay et al. 2008)

4.3.3 ASTM C 1293: Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction

The concrete prism test was initially developed for evaluating aggregate reactivity, but the most recent version, ASTM C 1293, has been modified to allow for testing SCMs as means of preventing or reducing deleterious expansion. The procedures and testing regimes are identical to those previously described for evaluating aggregate reactivity (see section 4.2.5), except for two significant modifications: (1) test duration is two years, and (2) SCMs may be used as replacement for portland cement (replacement by mass).

In addition, when testing SCMs that significantly increase water demand, such as silica fume, ASTM C 1293 allows for the use of high-range water reducers (HRWR) meeting ASTM C 494. The dosage of HRWR may be adjusted to ensure adequate workability and sample preparation while still achieving the target w/cm of 0.42-0.45. Conversely, if SCMs are used that substantially reduce water demand, such as high-volume fly ash mixtures, a viscosity-modifying admixture (VMA) may be used to minimize segregation and bleeding, while still achieving the target w/cm of 0.42 to 0.45.

Testing SCMs using ASTM C 1293 carries with it the same inherent shortcomings encountered when testing aggregates in the concrete prism test. It is not possible to accurately assess the effects of cement alkalis (and to some extent alkalis from SCMs) due to leaching of alkalis from the prisms over a two-year period. Concerns raised over the long-term nature of ASTM C 1293 when evaluating aggregates, which takes one year, are elevated when testing SCMs, as two years are required. This extended, two-year testing period is necessary to evaluate preventive measures, such as SCMs and lithium-based admixtures.

However, because two years are need to test preventive measures using ASTM C 1293, it has led most agencies and owners in the United States to rely on ASTM C 1567 (accelerated mortar bar test) for evaluating and selecting preventive measures. As mentioned in the previous section, ASTM C 1567 is a suitable test for evaluating preventive measures, provided that the aggregate yields comparable results when tested using AASHTO T 303 and ASTM C 1293 (see Figure 4.6). For the aggregates meeting this criteria, AASHTO PP65-11 recommends the use of an expansion limit for ASTM C 1567 of 0.10 percent after 14 days as this correlates best with 0.04 percent expansion after two years testing the same SCM(s) following ASTM C 1293 (Thomas et al. 2007).

Some agencies have specified a 28-day expansion criteria when evaluating SCM/aggregate combinations, but increasing from a 14- to 28-day expansion limit in ASTM C 1567 results in SCM levels significantly higher (that is, by 1.5 times on average) than that required to prevent damage in concrete, based on concrete prism tests and/or performance of outdoor exposure blocks (Thomas et al. 2007). The perception that much higher levels of SCM are required to control ASR may present a barrier to using such materials when potentially reactive aggregates are used due to concerns related to slow setting and strength gain, and even deicer salt scaling resistance.

4.3.4 Other ASR Test Methods

There are several other ASR test methods that have been developed and used, besides the ones previously described in this chapter, since the discovery of ASR in the late 1930s. Some of these methods are described briefly herein for completeness, including accelerated tests aimed at evaluating aggregate reactivity and tests related to alkali-release from aggregates and SCMs. Although the latter tests, those focusing on the release of alkalis from aggregates and/or SCMs, are not direct tests for ASR (e.g., expansion tests), they are discussed here because of the importance of alkali release in field structures. Although no standardized methods exist for accurately predicting the release of alkalis from aggregates and/or SCMs, some relevant studies and published literature are cited that should provide useful information on this important topic.

Over the years, there has been interest in and research related to ultra-accelerated ASR test methods, where ASR is accelerated through the use of an autoclave, which can subject mortar specimens to high temperatures and pressures. Fournier et al. (1991) and Nishibayashi et al. (1996) conducted extensive work on autoclaved mortar tests – these tests generated results within a couple days (typically less than 24 hours) through the use of extremely high alkali loadings (2.0 to 3.5% Na_2O_e by mass of cement), high autoclave pressures (up to 1.03 MPa), and high temperatures (up to 130°C). Fournier et al. (1991) and Giannini (2012) reported that the autoclaved test was as good as or better than the accelerated mortar bar test (AASHTO T 303) at identifying reactive aggregates. There has only been limited work done on testing concrete prisms in an autoclave test (Nishibayashi et al. 1996). Overall the interest in autoclave testing lies in the rapid results, accomplished through high alkalinity, high temperature, and high pressure. This test is very aggressive and unrealistic compared to field exposure, and its only benefit would be that it generates results, for good or bad, in about a day instead of two weeks, as in AASHTO T 303.

The Concrete Microbar Test (formerly the Chinese Accelerated Mortar Bar Method), was introduced by Xu and co-workers to capture reactivity of alkali-carbonate rocks (Xu et al. 1998; 2000), but has since been used to identify aggregates susceptible to ASR (Lu et al. 2008; East 2007). This mortar test uses larger specimens (40 mm x 40 mm cross section) and thus allows for testing larger aggregate particles, preserving the textural and mineralogical characteristics of the original coarse aggregate. This test is also advantageous for aggregates in which reactive phases are removed due to crushing and processing required in other accelerated mortar tests, such as AASHTO T 303 and ASTM C 227.

Past laboratory and field evaluations have demonstrated that some aggregates may release significant amounts of alkalis into the pore solution of concrete, thus increasing the likelihood of ASR-induced expansion and cracking (Stark 1980; Thomas et al. 1992; Bérubé et al. 2002). There are no standardized tests for estimating the alkali release from aggregates. There is a general agreement that the total alkali content of an aggregate, as measured through bulk chemical analysis, is not indicative of the available alkalis that will be released from aggregates into the pore solution in concrete. Recent work by Bérubé et al. (2002) has shown promise in better estimating the alkali release from a given aggregate by measuring the release of potassium from the aggregate immersed in sodium hydroxide and measuring the release of sodium from the same aggregate immersed in potassium hydroxide. This technique maintains the type of alkaline environment in which the aggregate will reside and selectively promotes the release of the two critical alkalis present in aggregates that may have an adverse effect on ASR.

The release of alkalis from SCMs, in general, and fly ash, in particular, into the pore solution of concrete has been the focus of considerable research over the years. ASTM C 311 (Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use in Portland-

Cement Concrete) is a standard test that measures the total alkali content, as well as the "available" alkali content of fly ash. However, research has shown that ASTM C 311 does not accurately measure the available alkalis from within fly ash, mainly because the aqueous solution in which the fly ash is immersed (pH = 7) is vastly different from the highly-alkaline pore solution found in mature concrete, with pH values typically in excess of 13.2. To attempt to generate data that is more relevant to actual alkali release from fly ash to concrete pore solution, a method was developed by Shehata and Thomas (2006) in which mature paste specimens, containing subject fly ash, are immersed in various alkaline solutions that are intended to better mimic the pore solution in concrete (e.g., 0.25 OH^- mol/L) undergoing ASR. The leaching of alkalis from the fly ash paste specimens is measured at the end of a 90-day immersion period, thereby directly measuring the alkali contribution from the fly ash to the host solution. Although the test developed by Shehata and Thomas (2006) is not a standardized test, it does show promise in being able to better estimate the release of alkalis from fly ash to concrete pore solution under realistic conditions.

4.4 ACR TEST METHODS FOR EVALUATING AGGREGATE REACTIVITY

ACR, described in chapter 9, 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. The test methods recommended for evaluating aggregates potentially susceptible to ACR are presented in Table 4.5.

Table 4.5. Test Methods for Evaluating Alkali-Carbonate Reactivity of Aggregates

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. • Petrographer focuses specifically on calcareous dolomites or dolomitic limestones with clayey insoluble residues, some dolomites essentially free of clay, and some very fine-grained limestones free of clay and with minor insoluble residue (mostly quartz).
CSA A23.2-26A: Determination of Potential Alkali-Carbonate Reactivity of Quarried Carbonate Rocks by Chemical Composition	• Measures the CaO, MgO, and Al_2O_3 content of quarried carbonate rocks. • Chemical composition plotted (see Figure 4.10) and classified as nonexpansive or potentially expansive. Guidance, including additional testing, such as ASTM C 1105, is provided based on classification.
ASTM C 1293: Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction	• Test method developed originally for evaluating aggregates susceptible to ASR. • Test method will also trigger ACR if such susceptible aggregates are present in mixture – petrography is required to confirm ACR is the cause of part or all of the measured expansion.
ASTM C 1105: Standard Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction	• Similar test to ASTM C 1293, except lower cement alkali content is used. ASR will not occur at lower alkali loading, and as such, any observed expansion is assumed to be caused by ACR. • Expansion limits are 0.025 percent at 6 months or 0.030 percent at 1 year. Aggregates exceeding limit(s) are considered to be alkali-carbonate reactive and shall not be used in concrete.

A petrographic evaluation (ASTM C 295) is recommended when assessing the potential for an aggregate being susceptible to ACR. Aggregates prone to ACR are typically calcareous dolomites or dolomitic limestones with clayey insoluble residues. Figure 4.9 shows a unique petrographic feature of ACR – rhombs of dolomite floating in a fine-grained matrix of clay minerals, calcite and silica.

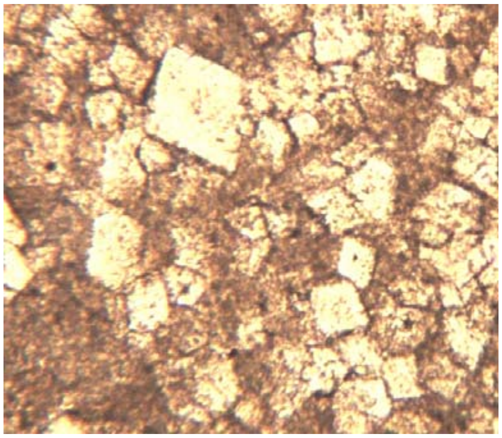

Figure 4.9. A Unique Petrographic Feature Typical of ACR Susceptible Aggregates: Rhombs of Dolomite Floating in a Fine-grained Matrix of Clay Minerals, Calcite and Silica (photo courtesy of C. Rogers)

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 4.10.

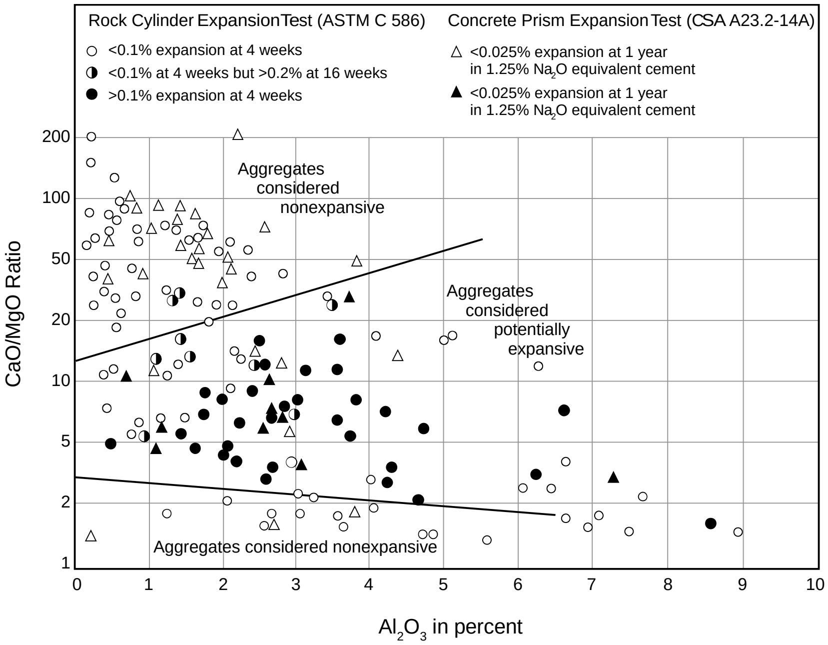

Figure 4.10. Using Chemical Composition as a Basis for Determining Potential Alkali-Carbonate Reactivity of Quarried Carbonates (from CSA A23.2-26A)

If the composition falls in one of the two ranges identified as "aggregates considered non-expansive" in Figure 4.10, the aggregate is not potentially alkali-carbonate reactive. However, the aggregate 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 4.10, 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 but should still be evaluated for potential ASR reactivity (see section 4.2).

4.5 SUMMARY

This chapter briefly described the various test methods that have been or are currently being used to evaluate aggregate reactivity (with regard to ASR and ACR) and to evaluate preventive measures against ASR, such as the use of SCMs or lithium-based admixtures. The test methods recommended in this chapter are consistent with those integrated into AASHTO PP 65-11, the first nationwide recommended practice aimed at preventing ASR in new concrete construction.

It is worth noting that the expansion limits and performance criteria recommended in this chapter are based extensively on the performance of large exposure blocks stored at outdoor exposure sites in the United States and Canada. As new data emerges from these sites and as blocks that had not previously expanded start to expand and crack, the expansion limits and performance criteria will evolve. It is hoped that future evolutions will increase the accuracy of AAR test methods and will also spur the development of improved test methods that better predict field performance.

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5 – Prevention of Alkali-Silica Reaction

5.1 INTRODUCTION

This chapter 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 chapter 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. Alkali-carbonate reaction is discussed in chapter 9.

5.2 PREVENTIVE MEASURES – OPTIONS

In chapter 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, from a consideration of the fundamental aspects of ASR discussed in chapter 2, 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 supplementary cementing materials (SCM)
4. Use lithium-based compounds

In fact, the use of SCM is a form of Option 2 as these materials consume a portion of the alkalis contributed by the portland cement 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. 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

Stanton's (1940) formative work on ASR indicated that expansive reaction is unlikely to occur when the alkali content of the cement is below 0.60% Na_2Oe. However, later research indicated that damaging ASR could occur both in the laboratory and in the field (e.g., Woolf 1952; Stark 1980; Blaikie et al. 1996) when low-alkali cements are used. Despite this, the 0.60% value has become the accepted maximum limit for cement to be used with reactive aggregates in the United States, and appears in ASTM C 150 Standard Specification for Portland cement as an optional limit when concrete contains deleteriously reactive aggregate. Appendix X1 of ASTM C 33 Standard Specification for Concrete Aggregates includes "low-alkali cement" (meeting the ASTM C 150 limit of 0.60% Na_2Oe) as a measure "known to prevent excessive expansion."

Limiting the alkali level of the cement takes no account of the cement content of the concrete which, together with the cement alkali content, governs the total alkali content of concrete, and is considered to be a more accurate index of the risk of expansion when a reactive aggregate is used in concrete. Figure 2.12 (in chapter 2) shows the relationship between alkali content and expansion for concretes produced with a range of cement contents and cements of varying alkali content (Lingård et al. 2012). The figure was produced using previously unpublished data from the Building Research Establishment (Blackwell, private communication). The relationship clearly shows that it is the product of cement content and cement alkali level (i.e., the alkali content of the concrete) that controls the alkali content rather than the cement alkali level alone.

Figure 5.1 shows expansion of concrete prisms plotted against the alkali content of the concrete for three different reactive aggregates (selected to demonstrate the range of behavior observed). It can be seen that the threshold alkali content required to initiate damaging expansion in the concrete prism test varies considerably between aggregates, with values ranging from approximately 3.0 kg/m^3 (5 lb/yd^3) Na_2Oe to more than 5.0 kg/m^3 (8.3 lb/yd^3) Na_2Oe for the aggregates shown in Figure 5.1.

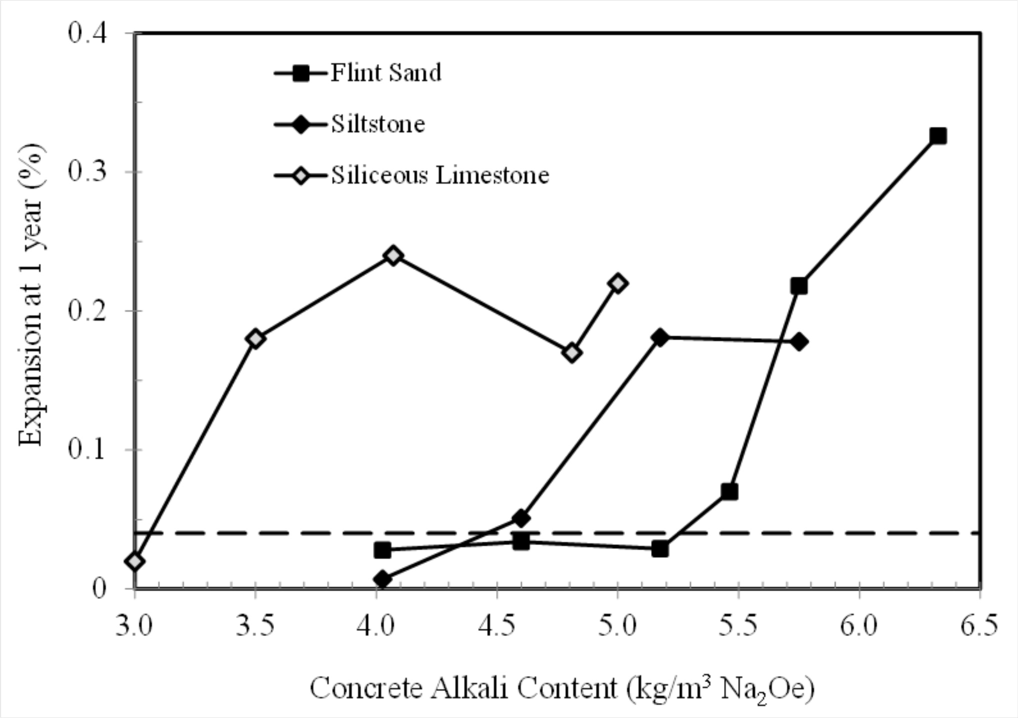

Figure 5.1. Effect of Alkali Content on Expansion of Concrete Prisms (Stored over Water at 38ºC, 100ºF) with Different Aggregates (data from Thomas et al. 1996 and Figure 2.12)

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_2Oe have been specified by various countries and agencies, with the allowable alkali content sometimes varying depending on aggregate reactivity.

Unfortunately, traditional concrete prism tests tend to underestimate the threshold alkali content for aggregates as a result of alkali being leached from the concrete during exposure. This phenomenon has been discussed in literature (Thomas et al. 2006; Lindgård et al. 2012) and a further example is shown in Figure 5.2. The figure shows the expansion of concrete blocks (0.38 x 0.38 x 0.71 m, 15 x 15 x 28 in.) containing reactive (Jobe) sand and various levels of alkali, and stored on an outdoor exposure site at the University of Texas in Austin, compared with the expansion of concrete prisms from the same mix stored over water (ASTM C 1293). It is evident that the concrete prism test will yield a higher threshold alkali content than may be observed for larger elements stored under field conditions.

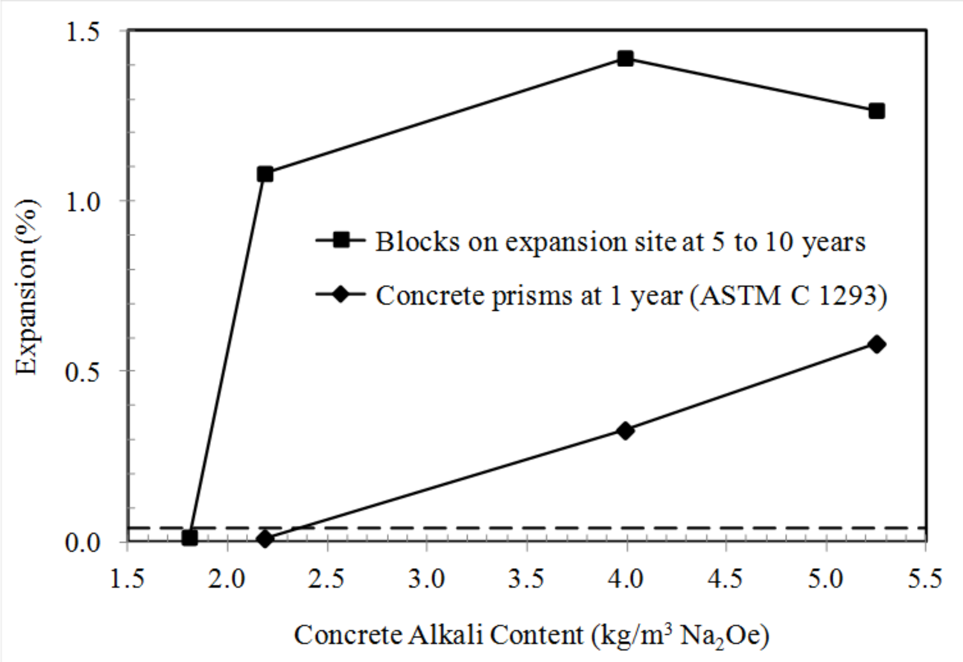

Figure 5.2. Effect of Alkali Content on the Expansion of Blocks Stored Outdoors and Prisms Stored over Water (unpublished data from the University of Texas at Austin)

In 2000, CSA A23.2-27A introduced a "sliding-scale" for the alkali limit used as a preventive measure, the value varying from 1.8 to 3.0 kg/m^3 (3 to 5 lb/yd^3) Na$_2$Oe 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 (W)	3.0	5.0
Moderate (X)	2.4	4.0
Strong (Y)	1.8	3.0
Exceptional (Z)	1.8 + SCM	3.0 + SCM

As the data available from laboratory tests at the time was considered unreliable for the purpose of establishing threshold alkali values, the maximum alkali content limits were based on published data from structures and field experience. Thomas (1996a) had previously reported damaging ASR in concrete dams in the U.K. and Canada where the estimated alkali content was in the range of 2.0 to 2.4 kg/m^3 (3.3 to 4.0 lb/yd^3) Na$_2$Oe. Rogers et al. (2000) reported results from a study of 8-year-old concrete blocks (0.6 x 0.6 x 2 m, 2 x 2 x 6.6 ft) stored on an exposure site in Ontario, Canada. Specimens produced with high-alkali cement showed very significant expansion and cracking after 8 years. Specimens with low-alkali cement (0.46%

Na_2Oe) and a calculated alkali content of 1.91 kg/m^3 (3.18 lb/yd^3) Na_2Oe showed significantly less expansion; however, the expansion did exceed 0.04% at 8 years and small cracks were evident. Subsequent investigations (Hooton et al. 2006; MacDonald et al. 2012) of these blocks at later ages (14 and 20 years) confirmed the presence of significant ASR-induced damage in the blocks produced with low-alkali cement and measured expansion of approximately 0.08% at 20 years (see Figure 5.3). Based on the information available, a maximum alkali limit of 1.8 kg/m^3 (3.0 lb/yd^3) Na_2Oe was selected for Prevention Level Y (e.g., concrete with highly reactive aggregate, exposed to moisture with a service life up to 75 years), and it was decided to require the same limit plus incorporate minimum levels of supplementary cementing material (SCM) for Level Z. This was supported by anecdotal evidence available to the committee preparing the guidelines, there being no known case of ASR in concrete structures with lower alkali contents. In the absence of any other data, the same alkali limits were adopted in AASHTO PP65-11; this is discussed further in chapter 6.

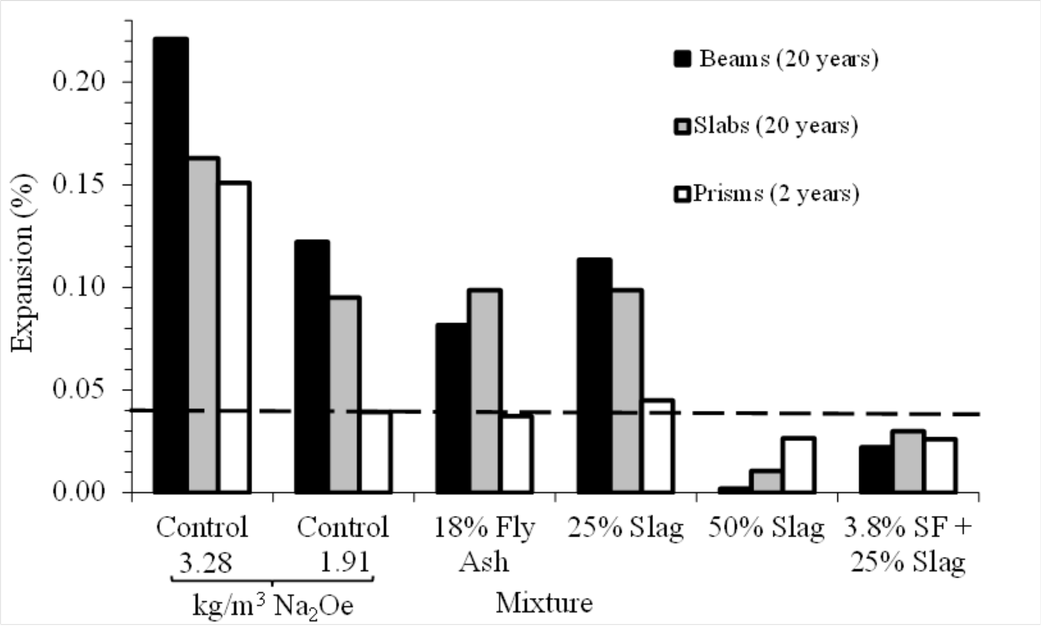

Figure 5.3. Expansion of Concrete Prisms in the Laboratory and Concrete Blocks and Slabs on the Kingston Exposure Site (data from MacDonald et al. 2012)

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_2Oe 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.

Alkali migration can occur very early in the life of concrete and has been implicated as the cause of popouts at the surface of slabs (Cong et al. 2004; Landgren and Hadley 2002). Alkalis may be carried upwards with the bleed water in freshly placed concrete, and this migration may be exacerbated during and after setting by the evaporation of water from the surface of the slab especially during hot, dry weather. The concentration of alkalis at the surface leads to ASR in the near surface of the concrete where the formation of gel and expansion of aggregates can lead to the fracture of a small conical-shaped mortar fragment (popout) overlying the reacting aggregate. Oftentimes the damage is restricted to the surface of the concrete where the alkalis are concentrated as there is insufficient alkali in the bulk concrete to cause damage. Some studies have reported surface concentrations that are as much as six times the alkali concentration in the bulk concrete (Nixon et al. 1979). This phenomenon can occur even when low-alkali cement or pozzolans are used (Cong et al. 2004).

A number of workers 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. A comprehensive review on alkali release from aggregates, including methods for determining the "releasable or available alkali" in aggregates, has been presented by Berube et al. (2002).

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 SCMs 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

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). In this paper, Stanton not only demonstrated that damaging reaction would only occur if there was a sufficient quantity of alkalis in the portland cement and reactive silica in the aggregate, but also that expansion was reduced when a pozzolanic cement was used. Ten years later, Stanton (1950) further demonstrated that partially replacing portland cement with a sufficient quantity of pozzolan (pumicite or calcined shale) eliminated deleterious expansion whereas replacement with similar quantities of ground quartz (Ottawa) sand did not, indicating that the beneficial action of the pozzolan extended beyond merely diluting the cement alkalis. In the early 1950s, various studies (Cox et al. 1950; Barona 1951; Buck et al. 1953) showed that other supplementary cementing materials (SCM), namely fly ash and slag, were also effective in reducing expansion.

Since these early studies there have been literally hundreds of studies and technical papers dealing with the effects of SCM on ASR, and it is now generally recognized that the use of a sufficient quantity of a suitable SCM is one of the more efficient preventive measures for controlling expansion when a deleteriously reactive aggregate is used in concrete (Thomas et al. 2008). Thomas (2011) reviewed selected published works dealing with (i) the mechanisms by which SCM controls ASR, (ii) the effect of SCM composition on its efficacy in this role, and (iii) test methods for determining the amount of SCM required to minimize the risk of damaging expansion to an acceptable level.

Thomas (2011) showed that 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, and
- The exposure conditions of the concrete; concrete exposed to external sources of alkali may require higher levels of SCM.

Figure 5.4 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.

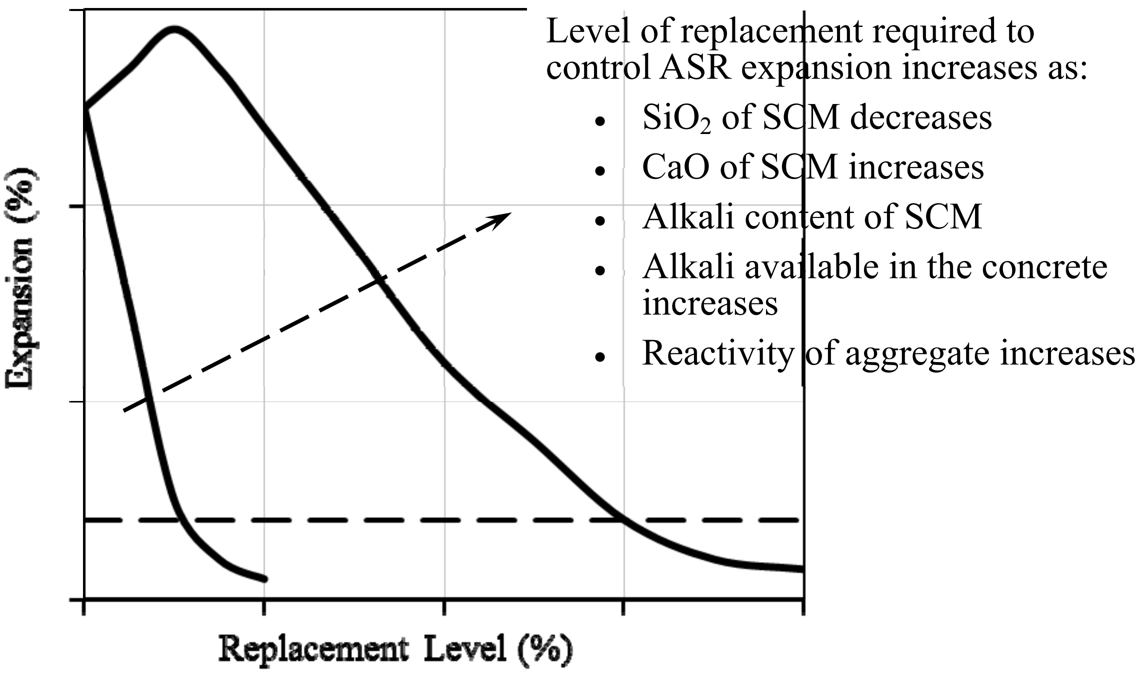

Figure 5.4. Expansion versus SCM Content - Conceptual Relationship (Thomas 2011)

It is generally considered that the principal mechanism by which SCMs control ASR expansion is by reaction with and consumption of the alkalis in the concrete pore solution which reduces the alkali available for reaction with the aggregates. However, the use of SCMs also results in a reduction in the availability of calcium (due to the dilution of and consumption of calcium hydroxide) and in a refinement in the pore structure, which leads to reduced ionic and moisture diffusivity; these effects may also be beneficial in terms of minimizing the risk and extent of ASR (Thomas 2011).

This section begins with a discussion on the effect of SCMs on the composition of the pore solution and the availability of alkalis, and then provides examples on the impact of SCMs on the expansion of concrete. The role of SCM composition, cement alkalis, and aggregate reactivity on the expansion of concrete containing SCMs is also discussed.

5.5.1 Effect of SCM on the Availability of Alkalis

Although all SCMs contain some level of alkali and some may contain significantly more alkali than the portland cement that they partially replace, the main mechanism by which SCMs reduce expansion due to ASR is by reducing the alkalis that are available to the concrete pore solution. Once the alkalis in the binder phase (portland cement + SCM) of concrete are "released" by hydration they may be present in one of three ways: dissolved within the pore solution, bound by the hydration products, or incorporated in alkali-silica gel. In the absence of reactive aggregate, alkalis will not be consumed by ASR, and the partition of the alkalis between the pore solution and the hydrates is largely a function of the composition of the binder.

Numerous workers have shown that SCMs have a significant impact on the concentration of alkalis in the pore solution (see Thomas 2011). Studies on the effect of fly ash and slag on the pore solution of pastes have been reviewed by Thomas (1996b), and studies involving silica fume have been reviewed by Thomas and Bleszynski (2001). These studies show that the incorporation of most SCMs leads to a reduction in the concentration of alkali hydroxides in the pore solution of pastes, mortar, and concretes, the amount of reduction increasing with higher SCM replacement levels. Figure 5.5 shows the evolution of the hydroxyl ion concentration of the pore solution extracted from sealed paste samples with w/cm = 0.50, and Figure 5.6 shows the OH^- concentration at 2 years as a function of the level of SCM (Shehata et al. 1999; Ramlochan et al. 2000; Bleszynski 2002; Shehata and Thomas 2002). Silica fume is the most efficacious SCM in this role, at least initially, followed by metakaolin, low-calcium fly ash, and slag. High-calcium or high-alkali fly ashes are less effective and have to be used at relatively high levels of replacement to produce a significant reduction in the pore solution alkalinity. It is interesting to note that in the case of the paste with 10% silica fume the OH^- concentration drops rapidly over the first 28 days but then starts to increase slowly with time beyond 3 months; similar behavior was observed in pastes containing 5% silica fume (Shehata and Thomas 2002). This behavior is not observed for pastes containing any of the other SCMs. As shown in Figure 5.5, the long-term increase in the OH^- concentration seems to be prevented in pastes containing 5% silica fume by the addition of either slag (25%) or fly ash (15%). It is conjectured that the presence of alumina in the SCM possibly contributes in some way to prevent the long-term release of alkalis back in to the pore solution. Hong and Glasser (2002) showed that introducing alumina into C-S-H, to form C-A-S-H, markedly increases its alkali-binding capacity, and they suggest that this partially explains the beneficial effects of aluminous SCM with regards to reducing pore solution alkalinity and the potential for ASR.

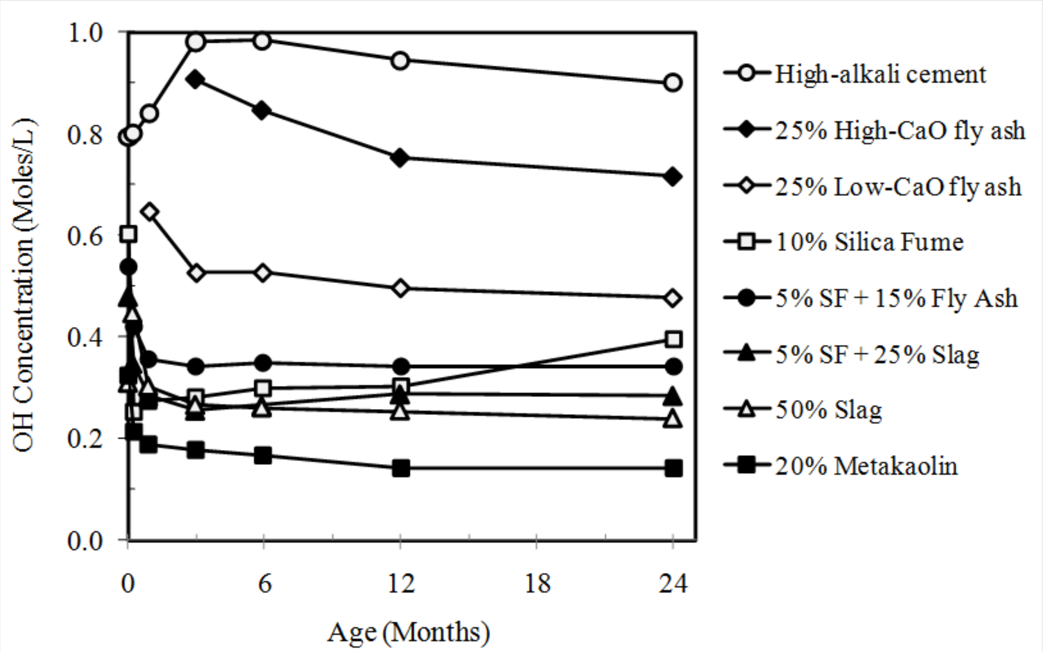

Figure 5.5. Evolution of the Pore Solution in Pastes Containing SCM (Shehata et al. 1999; Ramlochan and Thomas 2000; Bleszynski 2002; Shehata and Thomas 2002)

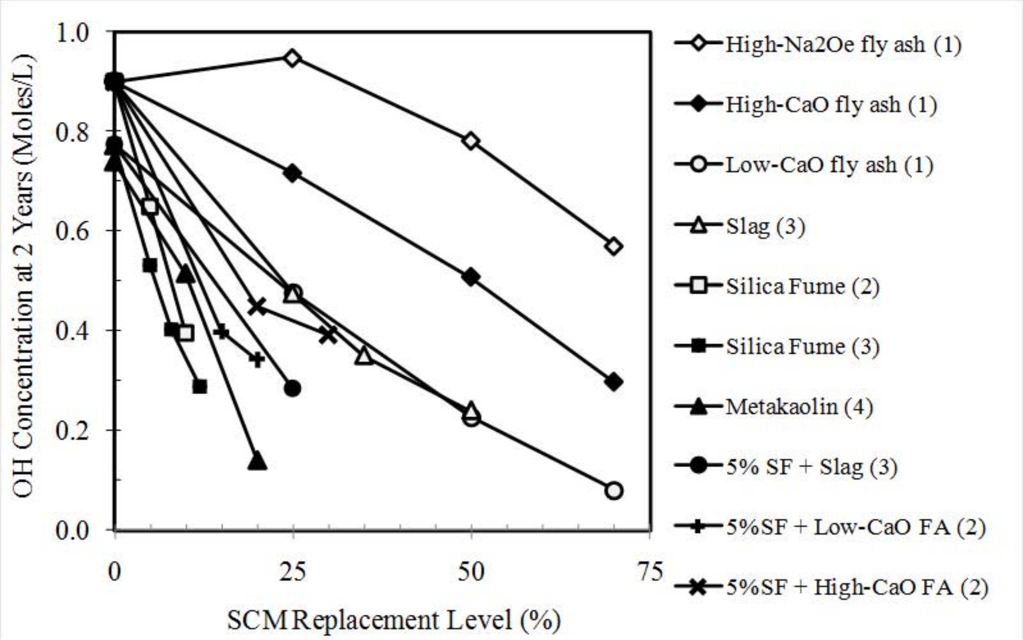

Figure 5.6. Effect of Type and Amount of SCM on Pore Solution Composition (Shehata et al. 1999; Ramlochan et al. 2000; Bleszynski 2002; Shehata and Thomas 2002)

Figure 5.7 shows an empirical relationship between the OH^- concentration of the pore solution extracted from 2-year-old sealed pastes with w/cm = 0.50 (Thomas and Shehata 2004; Thomas and Folliard 2007) and a "chemical index" derived from the chemical composition of the binder.

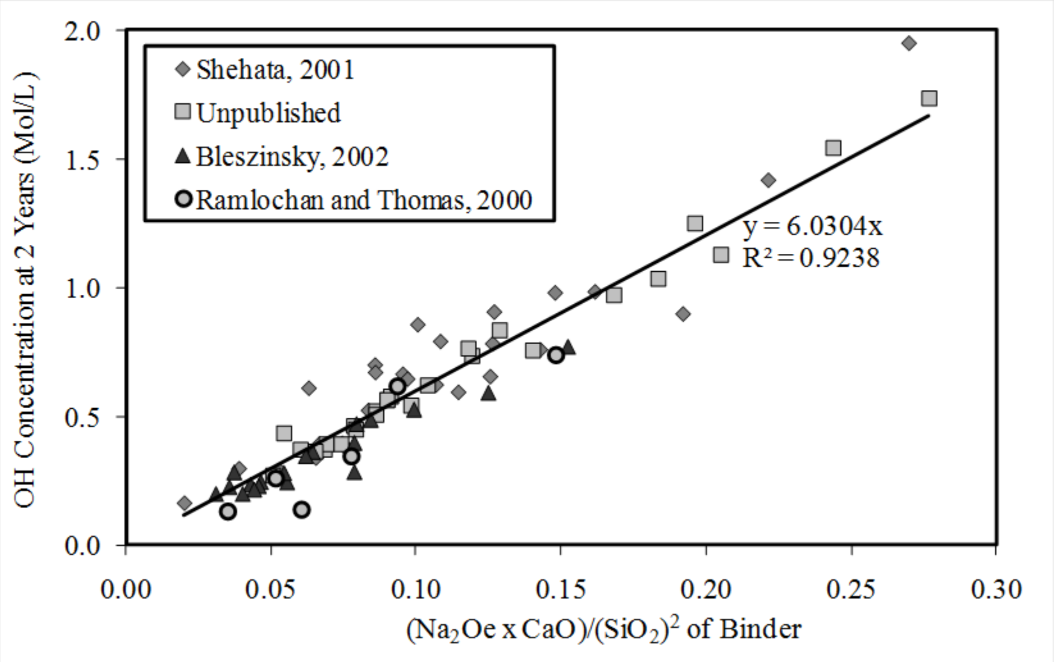

Figure 5.7. Relationship Between Pore Solution Composition and the Chemical Composition of the Binder (Thomas 2011)

A total of 79 different binders were tested including the following:

- 100% portland cement at a range of different alkali contents (0.36 to 1.09% Na_2Oe),
- Binary mixes with 25 to 70% fly ash using 18 different fly ashes with a range of chemical compositions (1.1 to 30.0% CaO, 1.4 to 9.7% Na_2Oe),
- Binary mixes with 25 to 50% slag, 5 to 10% silica fume and 10 to 20% metakaolin; each SCM came from a single source, and
- Ternary mixes containing silica fume blended with either slag or fly ash; both low-calcium and high-calcium fly ash were used.

The relationship was derived empirically to find the "chemical index" that was most reliably correlated to the OH^- concentration based on a least-square fit. The best-fit index was found to be the product of the equivalent alkalis and calcium divided by the square of the silica content of the binder $(Na_2Oe \times CaO)/(SiO_2)^2$. Although this is an empirical relationship it makes sense intuitively because the alkalinity of the pore solution can be expected to be a function of the alkalis in the binder and the ability of the hydrates to bind alkalis, which has been shown to be a function of the calcium-to-silica ratio of the binder (see discussion below). The alumina content of the binder was not found to be statistically significant based on the empirical analysis conducted using this dataset, despite the apparent benefit of alumina discussed above.

Analyzing the composition of the pore solution extracted from a paste sample only provides one point on the equilibrium curve between bound and free alkalis. If the alkali content of the pore solution in concrete decreases, due perhaps to leaching or reaction with reactive silica in the aggregate, a portion of the bound alkalis may be released to regain equilibrium. It is important to establish what portion of the alkalis in a binder are "available" to a solution at a pH that is just able to sustain the alkali-silica reaction, as it is these alkalis that are available to fuel the reaction. Shehata and Thomas (2006) studied the alkali release characteristics of pastes produced with high-alkali cement and combinations of silica fume and various fly ashes. Paste samples, 1 to 3 years of age, were immersed in solutions of alkali hydroxide at initial molar concentrations of 0, 0.10, 0.25, and 0.40 and the change in concentration was observed to determine how much alkali was leached from the binder. Figure 5.8 shows a selection of the data produced in this study. When mature paste samples were immersed in distilled water (pH = 7.0) almost all of the alkalis present in the binder (80 to 90%) were released regardless of composition. As the alkali concentration of the leaching solution increased, the amount of alkali released from the binder decreased and was strongly dependent on the composition of the binder. Pastes containing 100% portland cement, 5% silica fume, or 25% high-CaO fly ash still released a significant portion of the alkalis present in the binder (50 to 80%) even in the solution of the highest initial alkali concentration (0.40M). Much less alkali (~ 20%) was released from pastes containing 25% low-CaO fly ash or ternary blends containing silica fume with either low-CaO or high-CaO fly ash. A correlation of the data (Shehata and Thomas 2006) available for 24 different binders showed that the amount of alkali released to a solution with an initial alkali hydroxide concentration of 0.25M (assumed to be the concentration necessary to sustain alkali-silica reaction) was related to the chemical composition of the binder as represented by the parameter (Na_2Oe x CaO)/(SiO_2).

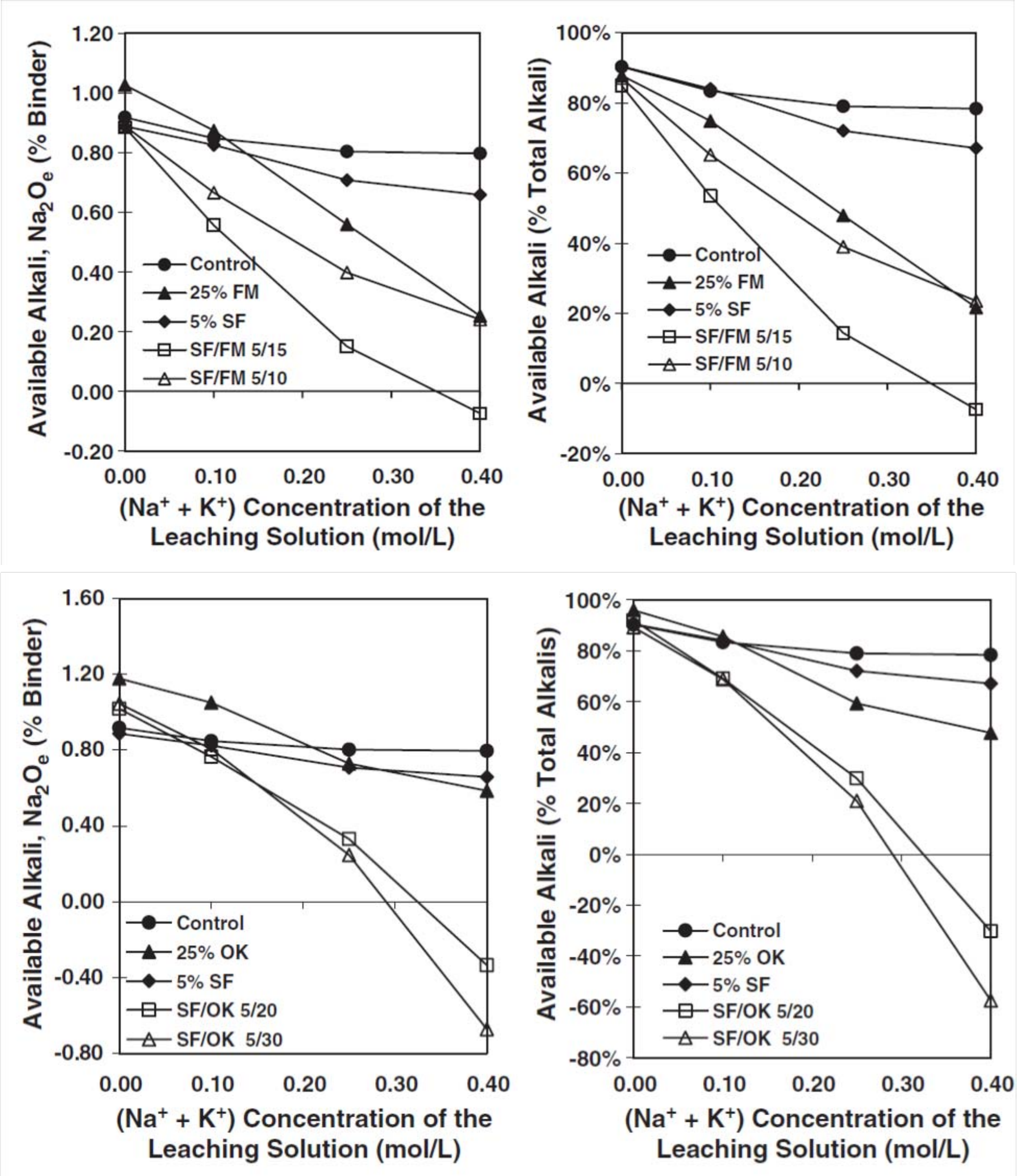

Figure 5.8. Available Alkalis in Pastes with SCM Stored in Solutions of Varying Reactivity (Shehata and Thomas 2006)

The ability of SCMs to reduce the pore solution alkalinity is linked to their effect on the composition and alkali-binding capacity of the hydrates (especially C-S-H). Bhatty and Greening (1978) found that C-S-H with a low Ca/Si ratio was able to retain more alkali (Na + K) compared to hydrates of higher lime to silica ratios. The addition of fly ash reduces the Ca/ Si ratio of the C-S-H hydrates, and there is a concomitant increase in the alkali content. Rayment (1982) observed significant differences in the C-S-H composition of portland cement and fly ash pastes after just 8 days curing at 20ºC. However, Uchikawa et al. (1989) found little difference in pastes after 91 days at 20ºC but substantial changes due to the incorporation of fly ash after 60 days at 40ºC, indicating the role of the pozzolanic reaction in the CSH composition. Thomas et al. (1991), reporting results for 7-year-old concretes containing reactive flint sand, showed that the alkali binding capacity of C-S-H hydrates in concretes was increased significantly by the addition of fly ash. Uchikawa et al. (1989) showed that slag has a similar effect to low-calcium fly ash on hydrate composition. Glasser & Marr (1985) explain the differences in alkali absorption on the basis of the surface charge on the C-S-H, which is dependent on the Ca/Si ratio. At high ratios, the charge is positive and the C-S-H tends to repel cations. As the Ca/Si ratio decreases the positive charge reduces, becoming negative at low Ca/ Si ratios, e.g., less than 1.3 (Glasser 1992). Negatively charged C-S-H has an increased capacity to sorb cations, especially alkalis. Hong and Glasser (1999) confirmed the importance of the Ca/Si ratio on the alkali-binding capacity of synthesized single-phase C-S-H but subsequently showed that the binding capacity could be greatly increased by introducing alumina into the C-S-H to form C-A-S-H (Hong and Glasser 2002).

Many of the studies on the alkali-binding of C-S-H have involved microanalysis (e.g., using scanning electron microscopy equipped with energy dispersive x-ray analysis) of the inner-product C-S-H forming around remnant alite and belite grains. However, outer-product C-S-H also forms by reaction between $Ca(OH)_2$ and pozzolans, but this phase is more difficult to identify and analyze separately than the inner-product C-S-H. The pozzolanic reaction is actually very similar to the alkali-silica reaction. The reactive silica in the pozzolan reacts first with the alkali-hydroxides in the pore solution, and alkali-silica gel containing small amounts of calcium is formed. 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 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. 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 (sub-180 microns in Stanton's studies) expansion is eliminated. Figure 5.9 shows data from Thomas (2011) relating to the effect of ground Vycor glass (sub-100 micron) 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 5.10 (from Maas et al. 2007) shows a back-scattered electron image of a mortar bar containing agglomerated silica fume after storage in 1M NaOH solution at 80ºC 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.

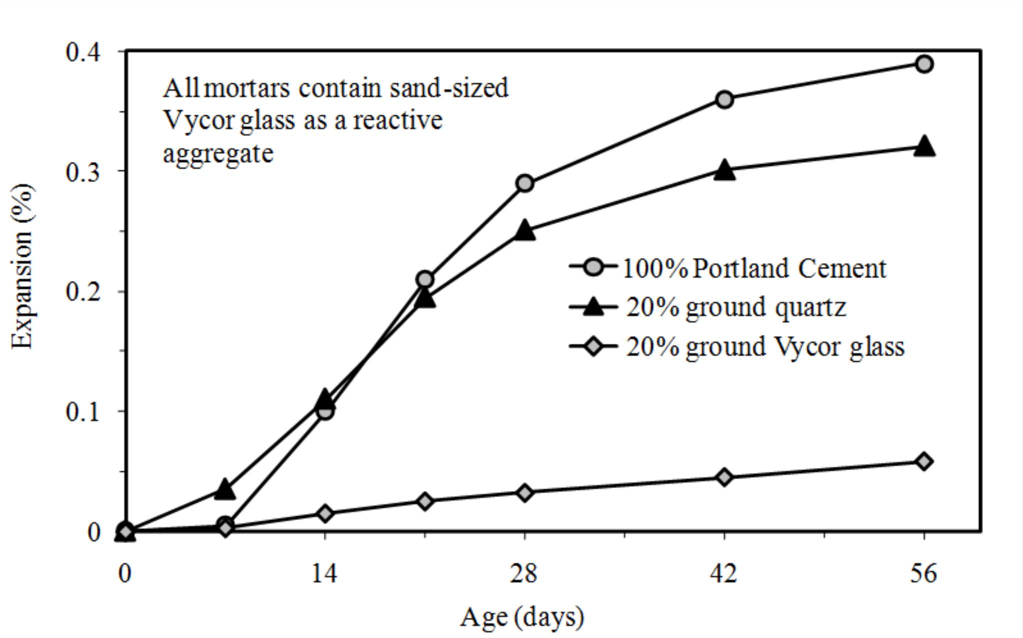

Figure 5.9. Role of Particle Size on the Behavior of Vycor Glass (Thomas 2011)

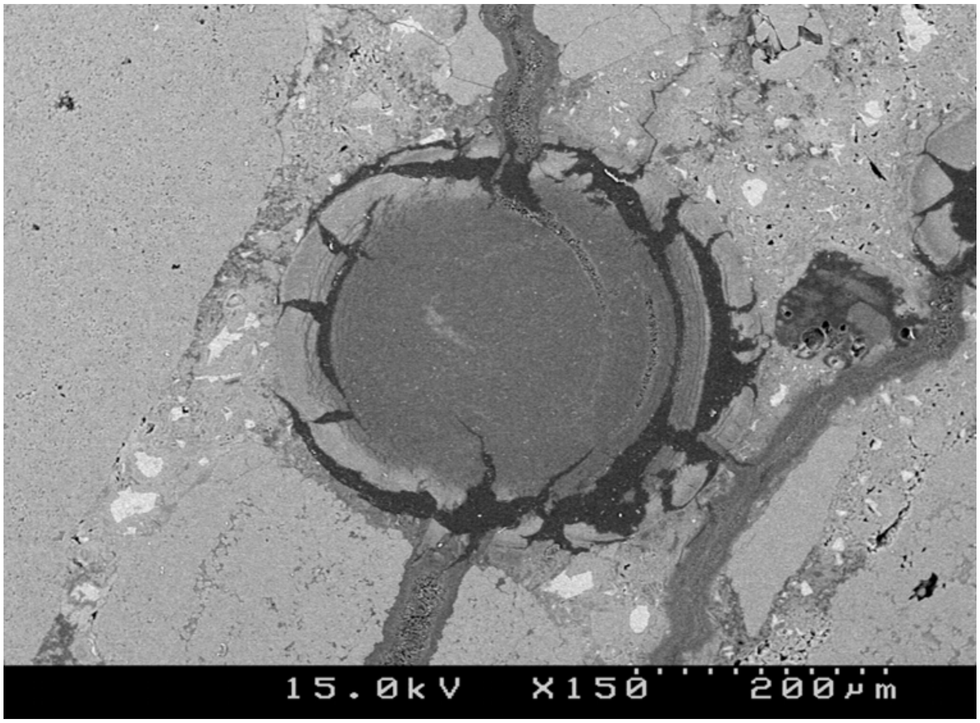

Figure 5.10. Agglomerated Silica Fume Particle Behaving as a Reactive Aggregate and Source of ASR Expansion (Maas et al. 2007)

5.5.2 Effect of SCM on the Expansion of Concrete

Stanton proposed a test method for evaluating the potential for cement-aggregate combinations to expand due to ASR in his first major paper on the subject (Stanton 1940). This involved the manufacture of small mortar bars and storing them over water in sealed containers. Subsequent modifications included elevating the temperature to 38ºC (100ºF), and the test was eventually standardized to become ASTM C 227. Pozzolans were often evaluated using a modified version of this test with either the job aggregate or a standard reactive aggregate, Pyrex glass; the test with Pyrex became standardized as ASTM C 441. ASTM C 227 is no longer widely used as it fails to detect many slowly-reacting aggregates because the small sample size and test conditions promote the leaching of alkalis from the bars; such a drawback is also a problem for evaluating SCMs (Thomas et al. 2006). ASTM C 441 is a much more rapid test (typically 14 to 56 days) because of the high reactivity of the Pyrex glass, and leaching is less significant during the test. However, this test fails to account for the nature of the reactive aggregate which is known to impact the amount of SCM required, and tends to overestimate the amount of SCM required to control expansion with natural aggregates (Thomas et al. 2006). However, the use of Pyrex glass does allow a comparative evaluation of pozzolans, and a number of investigators have used the test to observe the reduced efficacy of high-calcium ash compared with low-calcium ash (Dunstan 1981; Buck and Mather 1987; Klieger and Gebler 1987; Smith 1988;

Carrasquillo and Snow 1987). Dunstan's (1981) work was the most comprehensive with regards to fly ash composition. He reported results from Pyrex mortar bar tests for 17 ashes of varying chemistry and showed a reliable correlation between the calcium content of the ash and the expansion of mortar bars at 14 days.

The test methods most commonly used today to evaluate the efficacy of SCM in controlling ASR expansion are the concrete prism test (ASTM C 1293) and the accelerated mortar bar test (ASTM C 1567). The accelerated test is by far the most widely used test, however, it is only intended as a screening test and should not be relied upon for phenomenological studies. The test involves the immersion of small mortar bars in 1 M NaOH solution at 80ºC, and this tends to mask the importance of the alkalis in the system under test. Since SCMs control ASR expansion mainly by reducing the availability of alkalis, providing an inexhaustible supply of alkalis is not desirable as eventually the beneficial effects of the SCM will be swamped by the ingress of alkalis from the storage solution. Furthermore, the very high temperature used in this test is not representative of the conditions that concrete encounters in the field. The review presented here focuses on expansion tests involving concrete exposed either to field conditions or to accelerated conditions (up to 38ºC) in the laboratory (e.g., ASTM C 1293 concrete prism tests and similar tests). Data from ultra-accelerated tests, such as the accelerated mortar bar test (ASTM C 1567), are not included.

5.5.2.1 Effect of SCM Composition on Expansion

As discussed above and illustrated in Figure 5.4, almost any SCM can be used to control expansion due to alkali-silica reaction provided it is used in sufficient quantity. The amount of SCM required is, of course, dependent on the composition of the SCM, but also on the reactivity of the aggregate, the quantity of alkalis supplied by the portland cement (and other sources), and whether the concrete will be exposed to alkalis (e.g., seawater, deicing chemicals) during service. On one extreme, a highly efficient pozzolan with a high level of reactive silica and negligible alkali content may be expected to eliminate damaging expansion with a moderately reactive aggregate when used with a moderate-alkali cement at replacement levels of about 10%; this scenario is represented by the left-hand curve in Figure 5.4. On the other extreme, as represented by the right-hand curve in Figure 5.4, an SCM with a higher alkali and lower silica content might need to be used at a replacement level of 50 to 60% or more with a highly reactive aggregate and high-alkali cement.

Figure 5.11 shows the expansion of concretes at 2 years as a function of the type and amount of SCM used; the tests were performed in a single laboratory using a reactive aggregate from a single source (siliceous limestone from the Spratt quarry in Ontario, Canada) and were, generally, performed in accordance with ASTM C 1293. Silica fume and metakaolin are the most efficient with regards to reducing the expansion at 2 years, followed by low-calcium fly ash. Slag, high-calcium fly ash, and high-alkali fly ash were less efficient and had to be used at significantly higher replacement levels to control expansion to below 0.040% at 2 years. Generally, the effect of the different SCMs on the expansion of concrete prisms was consistent with their effect on pore solution alkalinity.

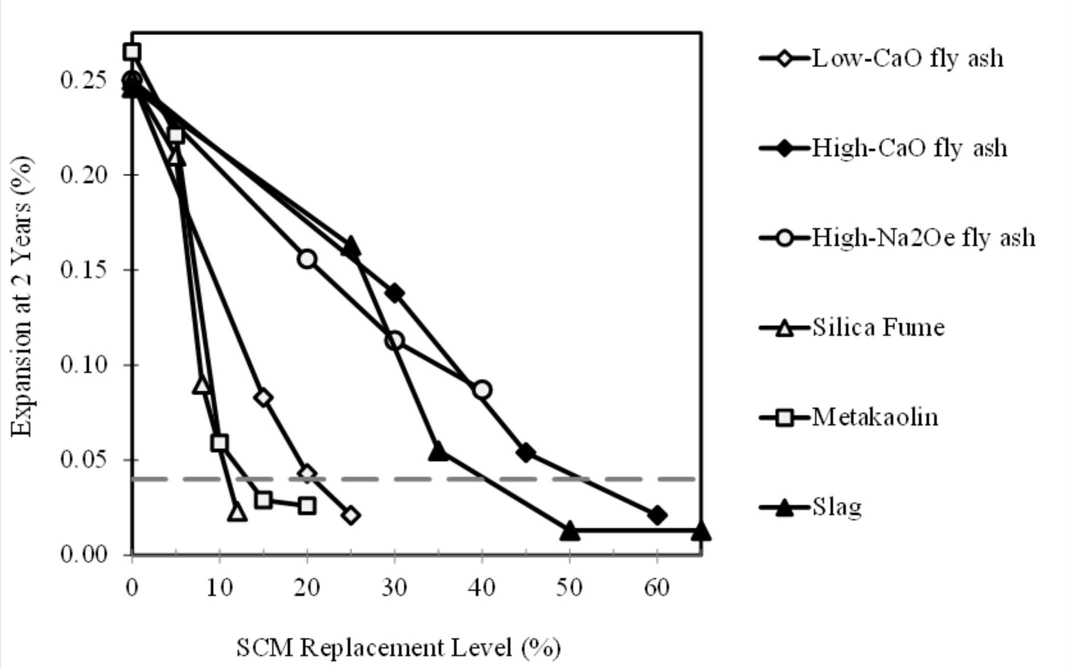

Figure 5.11. Effect of SCMs on Two-Year Expansion of Concrete Containing Siliceous Limestone (Shehata and Thomas 2002; Ramlochan et al. 2000; Thomas and Innis 1998)

Figure 5.12 (modified from Shehata and Thomas 2000) shows the 2-year expansion of concrete containing 25% fly ash and a siliceous limestone (Spratt) aggregate; fly ashes from 29 different sources are shown in this figure. Fly ashes with low to moderate alkali (≤ 4% Na_2Oe) and calcium contents (≤ 20% CaO) are generally effective in controlling expansion below 0.040% at 2 years. As the calcium content increases above 20% CaO there is a marked increase in expansion with increasing calcium content. Fly ashes with high alkali contents (> 5% Na_2Oe) are not effective in controlling expansion when used at a replacement level of 25% regardless of the calcium content of the fly ash. High-calcium fly ashes may be effective in controlling ASR expansion when used at increased levels of replacement. Figure 5.13 shows the expansion of concrete prisms at 2 years plotted as a function of the level of fly ash replacement for Class F fly ashes with less than 10% CaO and Class C fly ashes with more than 20% CaO. Whereas 20 to 25% low-CaO Class F fly ash was sufficient to control expansion, replacement levels of 50% or more were required with the high-CaO Class C fly ashes.

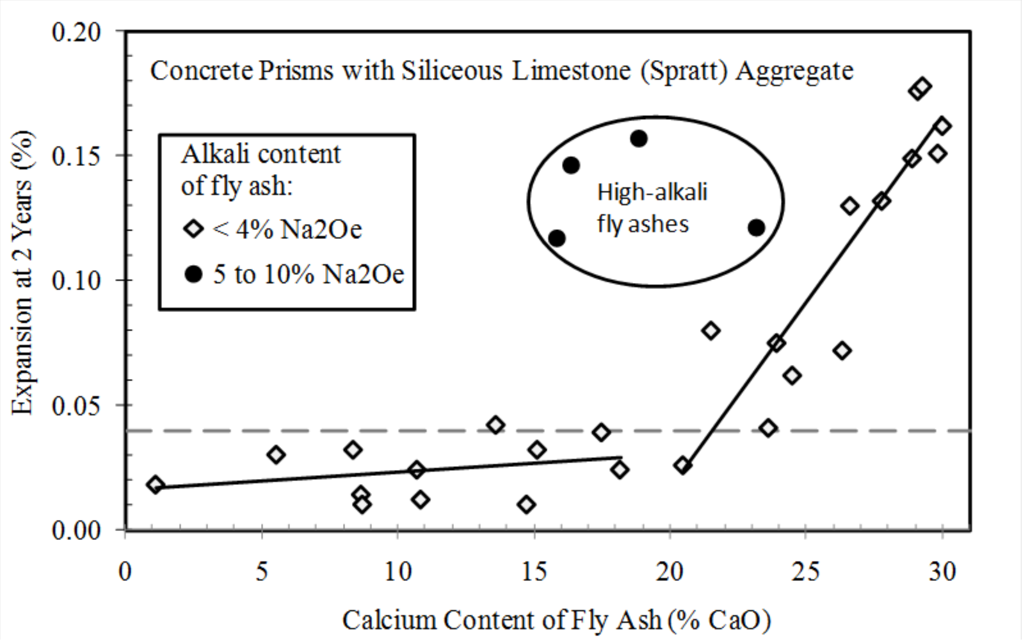

Figure 5.12. Effect of the Calcium Content of Fly Ash on the Two-Year Expansion of Concrete Containing Siliceous Limestone (Shehata and Thomas 2002; Thomas 2011)

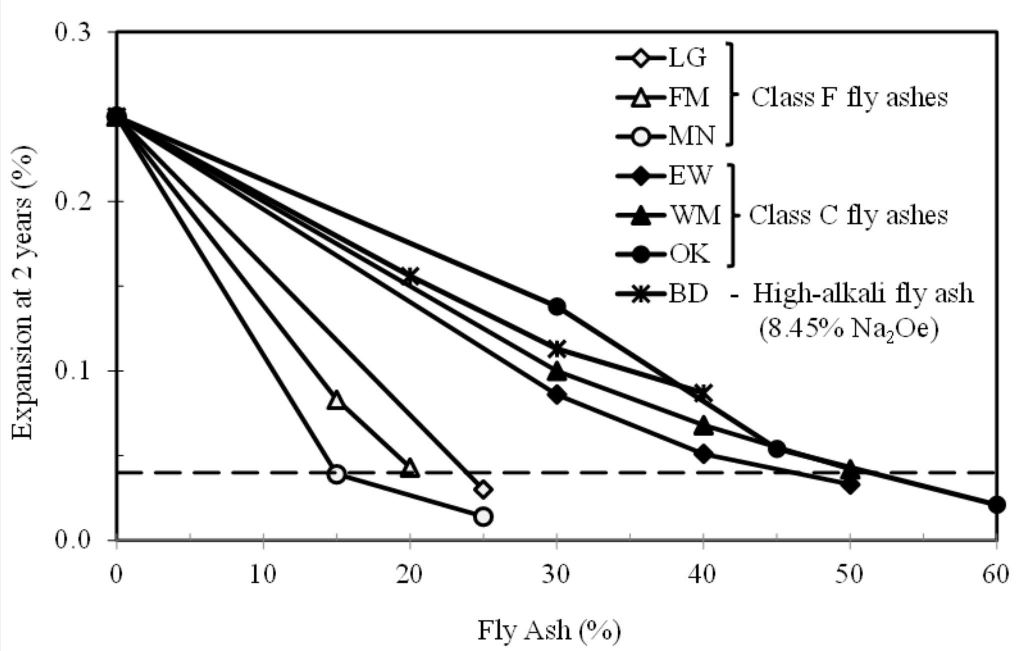

Figure 5.13. Effect of Fly Ash Replacement Level on Expansion of Concrete (Shehata and Thomas 2000)

Ternary blends of portland cement with two SCMs are also effective in controlling ASR expansion. Figure 5.14 shows the expansion of concrete containing 5% silica fume in combination with fly ashes with different calcium contents. Combinations of 5% silica fume with either 10 to 15% low-CaO fly ash or 20 to 30% high-CaO fly ash are effective in controlling expansion (≤ 0.040% at 2 years). Similarly Figure 5.15 shows that combinations of moderate amounts of silica fume (2 to 6%) are effective with moderate amounts of slag (15 to 35%).

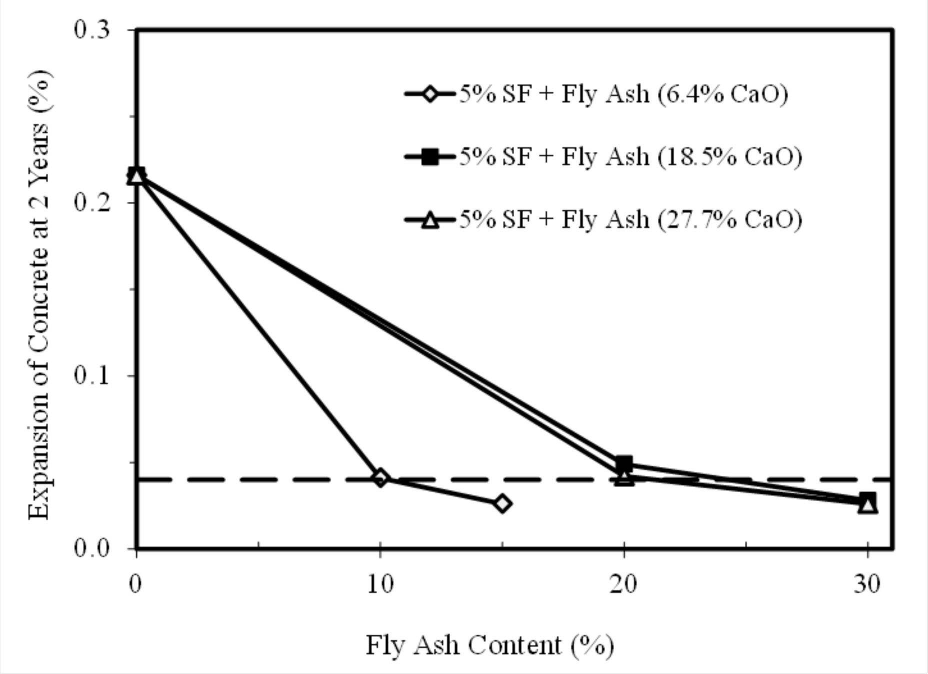

Figure 5.14. Expansion of Concrete with Blends of Silica Fume and Fly Ash (Shehata and Thomas 2002)

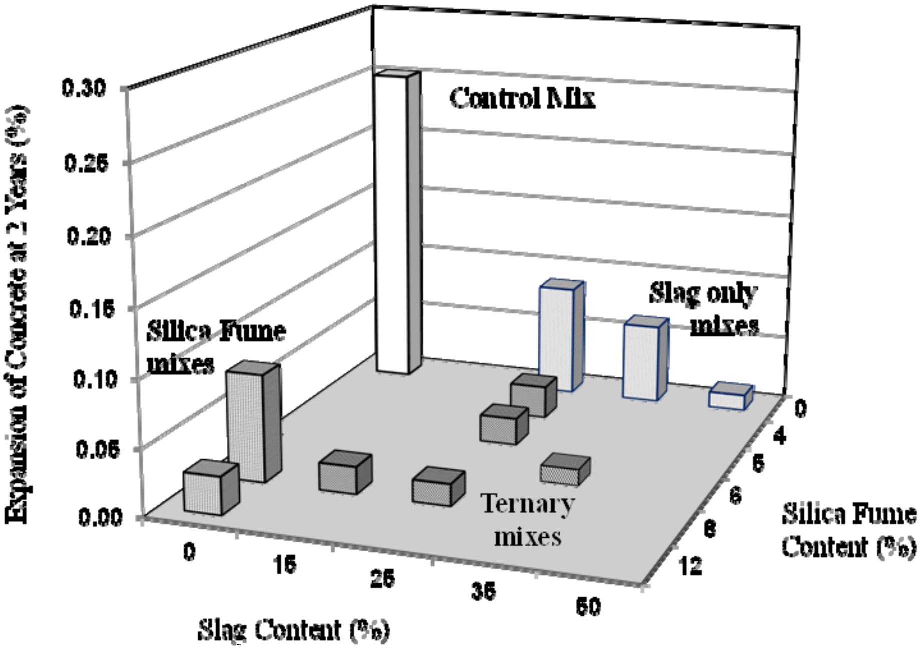

Figure 5.15. Expansion of Concrete with Blends of Silica Fume and Slag
(Bleszynski 2002; Bleszynski et al. 2002)

Figure 5.16 shows an empirical relationship between the expansion of concrete at 2 years and a "chemical index" derived from the chemical composition of the cementing materials to produce 132 different concrete mixes which were tested in accordance with ASTM C 1293 (Thomas and Shehata 2004). The cementing materials used to produce these concretes were the same as those used for the pore solution study discussed above. The reactive coarse aggregate was siliceous limestone (Spratt). The best fit between expansion and chemical composition was found to be with the following index: $[(Na_2Oe)^{0.33} \times CaO]/(SiO_2)^2$. This relationship is not intended as a method for predicting expansion based on the chemical composition of the binder phase, but merely to examine what constituents of the binder tend to most influence ASR expansion. The relationship is likely quite different if a different reactive aggregate or, even, a different test method is used. However, the relationship does indicate that expansion is likely to increase as the alkali and calcium content of the binder increase or as the silica content decreases, and this is somewhat intuitive. It is interesting that the alkali content of the binder appears to play a less important role in determining expansion compared with the pore solution composition, but this is likely an artifact of the test conditions as significant leaching of alkalis occurs during the concrete prism test, and this may reduce the apparent importance of the initial alkali content. This effect can be observed when looking at the expansion data for the concrete mixes produced with low-alkali cement. The expansion is lower than that expected based on the chemical composition. However, it is known that the concrete prism test will likely underestimate the expansion with low-alkali cement because of leaching (Thomas et al. 2006). As with the relationship with pore solution, the alumina content of the binder does not appear to significantly affect the expansion of concrete. The role of alumina, however, is not yet well understood, and further study is needed to determine its impact.

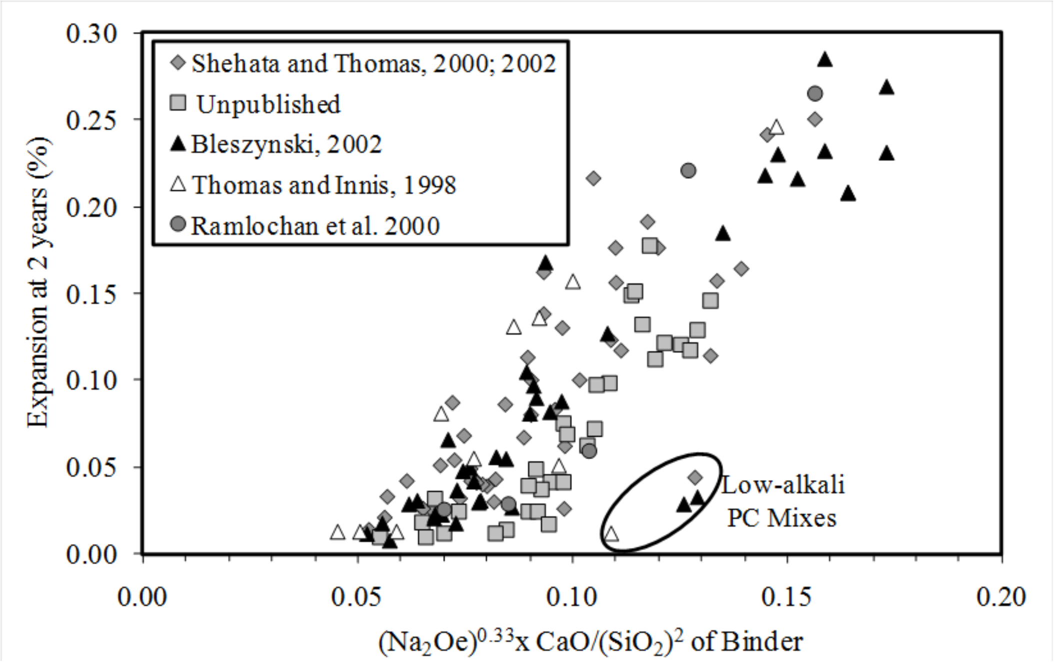

Figure 5.16. Effect of Binder Composition on the Expansion of Concrete Containing Siliceous Limestone

5.5.2.2 Effect of Cement Alkalis

One of the drawbacks of the concrete prism test is that significant alkali leaching occurs during the test, which means it cannot generally be used to determine the threshold alkali content required to initiate expansion with a specific aggregate or to determine how the minimum amount of SCM required varies as the alkali content of the cement changes (Thomas et al. 2006). Larger samples exposed under natural conditions should be relatively immune from the effects of alkali leaching, however, much longer testing periods are required.

Figure 5.17 shows the expansion of 300-mm concrete cubes containing a reactive hornfels aggregate and stored outdoors at the National Building Research Institute (NBRI) in South Africa (Oberholster and Davies 1987; Oberholster 1989). Two series of mixes were cast with cementitious contents of approximately 350 and 450 kg/m^3. Within each series 5% or 10% of the portland cement by mass was replaced with an equal volume of silica fume, resulting in silica fume levels of 3.5% and 7.0% by mass. The "active" alkali content was maintained at a constant level within a given series by addition of alkali hydroxide (using the same Na_2O to

K_2O ratio as the cement). The "active alkalis" included the available cement alkalis (using ASTM C 311) plus the alkali hydroxide, but excluded alkalis in the silica fume. The use of 3.5% or 7% silica fume delayed the onset of expansion and time to cracking in all cases. However, only the mixture at the lower cement content with 7% silica fume failed to expand after just over 7 years field exposure. These data clearly show the effect of alkali content on the efficacy of silica fume in controlling expansion. A replacement level of 7% silica fume appears to have been sufficient to provide long-term prevention of expansion when the "active alkalis" of the mix were just less than 4 kg/m^3 Na_2Oe but not at the higher alkali content of 5 kg/m^3 Na_2Oe.

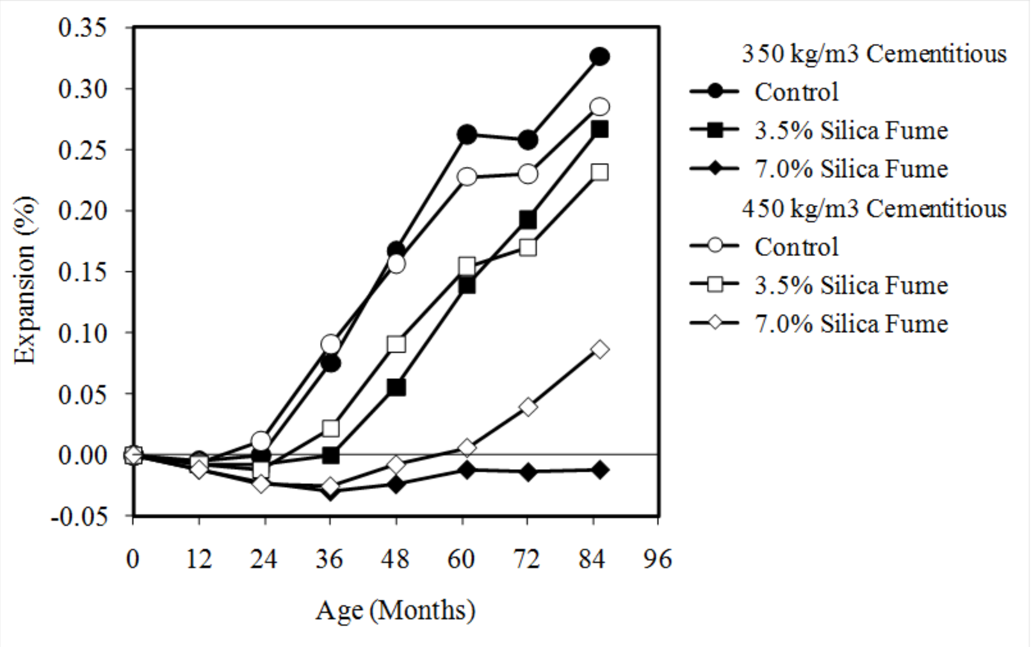

Figure 5.17. Effect of Cement Alkalis and Expansion of Concrete Containing Silica Fume (Oberholster 1989)

Figure 5.18 shows data from Fournier and co-workers (2004) for concrete blocks stored outdoors in Ottawa, Canada. The blocks contain a reactive greywacke coarse aggregate (Springhill Quarry in New Brunswick) and 420 kg/m^3 of cementing material. High-alkali portland cement with 0.90% Na_2Oe was used to manufacture the blocks and in some cases the alkali content of the portland cement component of the concrete was boosted to 1.25% Na_2Oe by the addition of NaOH to the mix water as per ASTM C 1293. The data in Figure 5.18 show that although the differences in expansion between boosted and unboosted blocks is not large, increased amounts of SCM are clearly needed to control expansion to acceptable levels when the alkali content of the mixture is increased.

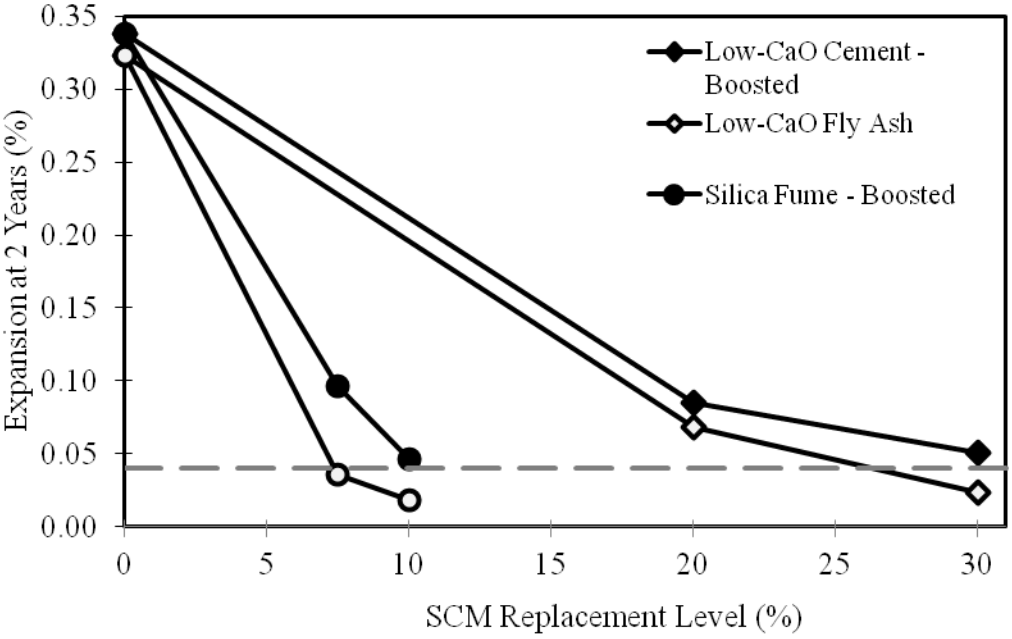

Figure 5.18. Effect of Cement Alkalis on Expansion of Concrete Containing Fly Ash or Silica Fume (Fournier et al. 2004)

5.5.2.3 Effect of Aggregate Reactivity

Figure 5.19 shows 2-year expansion data for concrete prism tests containing various reactive aggregates and different amounts of slag (Thomas and Innis 1998) or metakaolin (Ramlochan et al. 2000). It can be seen that the amount of slag required to limit expansion below 0.040% at 2 years varies between 35% and 50% depending on the aggregate type. Figure 5.20 shows similar data for blocks exposed on an outdoor exposure site in Ottawa (Fournier et al. 2004); the blocks contain high-alkali cement (boosted to 1.25% Na_2Oe), either low-CaO Class F fly ash or silica fume, and aggregates of varying reactivity. For three of the reactive aggregates, which produced expansion levels between 0.097% and 0.219% when tested with 100% portland cement, a fly ash replacement level of 20% or a silica fume replacement level of 7.5% was sufficient to reduce expansion below 0.040%. For the aggregates that produced an expansion of 0.338% when tested with 100% portland cement, neither 30% fly ash nor 10% silica fume was quite sufficient to reduce the expansion below 0.040%, although expansions were much reduced (0.051% and 0.046% with 30% fly ash and 10% silica fume, respectively). For the aggregate that produced the largest expansion when tested with portland cement (0.386%), significant expansion (0.148%) still occurred with 30% fly ash. In this figure, it would appear that the amount of fly ash required increases as the reactivity (as determined by the expansion when tested with portland cement alone) of the aggregate increases.

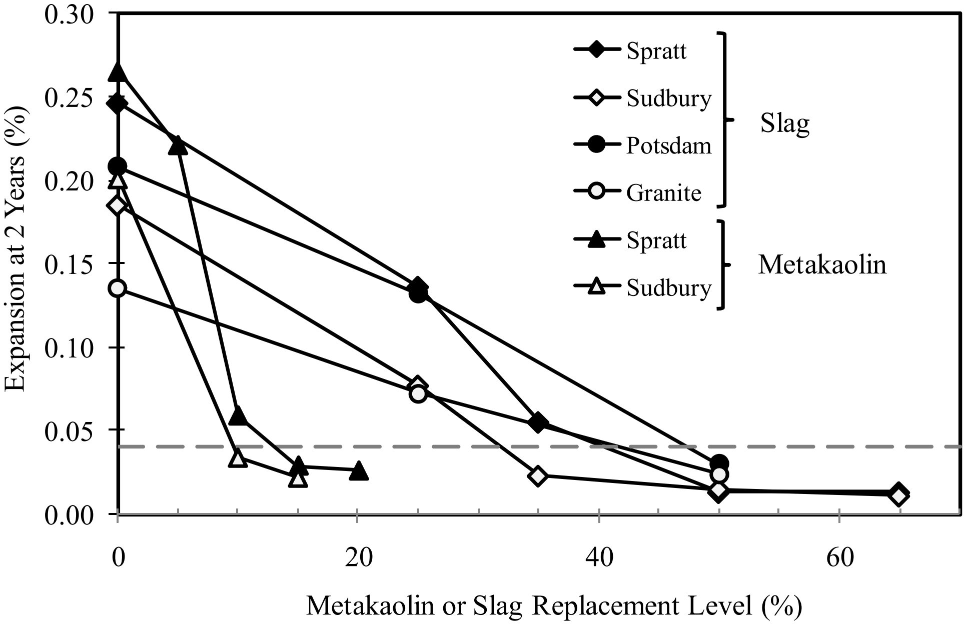

Figure 5.19. Effect of Aggregate Type on the Amount of Metakaolin or Slag Required to Control ASR Expansion (Thomas and Innis 1996; Ramlochan et al. 2000)

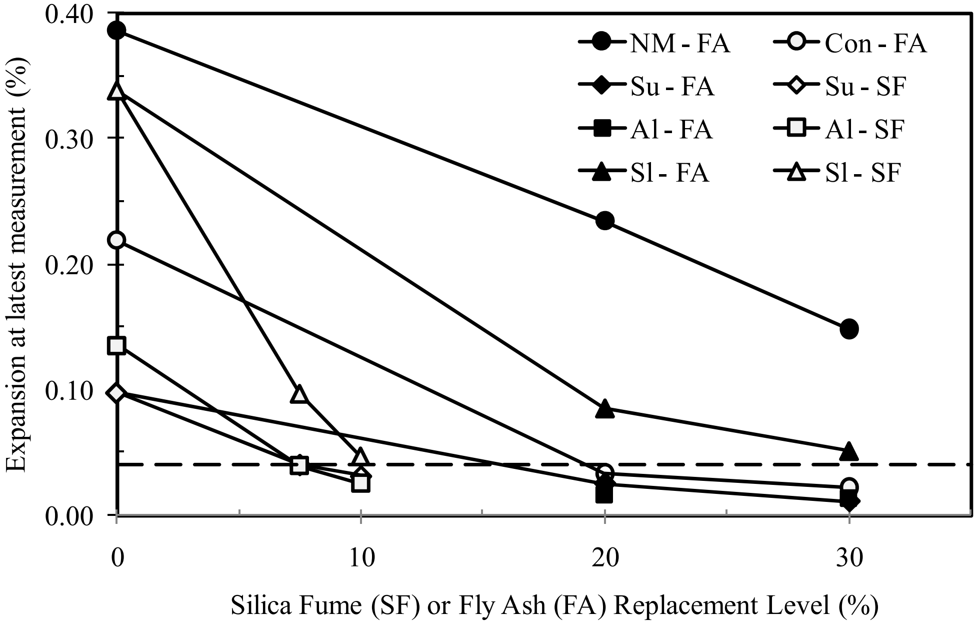

Figure 5.20. Effect of Aggregate Type on the Amount of Silica Fume (SF) or Fly Ash (FA) Required to Control ASR Expansion (Fournier et al. 2004). NM, Con, Su, Al and Sl are different aggregates. FA and SF are fly ash and silica fume, respectively.

5.5.3 Summary on Effect of SCM on ASR

Supplementary cementing materials are an effective means for controlling expansion due to alkali-silica reaction and most, if not all, SCMs can be used in this role provided they are used at a high enough level of replacement. The level of SCM required generally increases with the following parameters:

- The alkali available from the portland cement increases (If significant alkalis are available from the aggregates – e.g., feldspars, greywackes – or from external sources, this will likely also increase the level of SCM required.)
- The alkali from the SCM increases
- The CaO/SiO_2 of the SCM increases
- The reactivity of the aggregate increases

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, critical structure with extended service life).

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 5.21 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 (≤ 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.

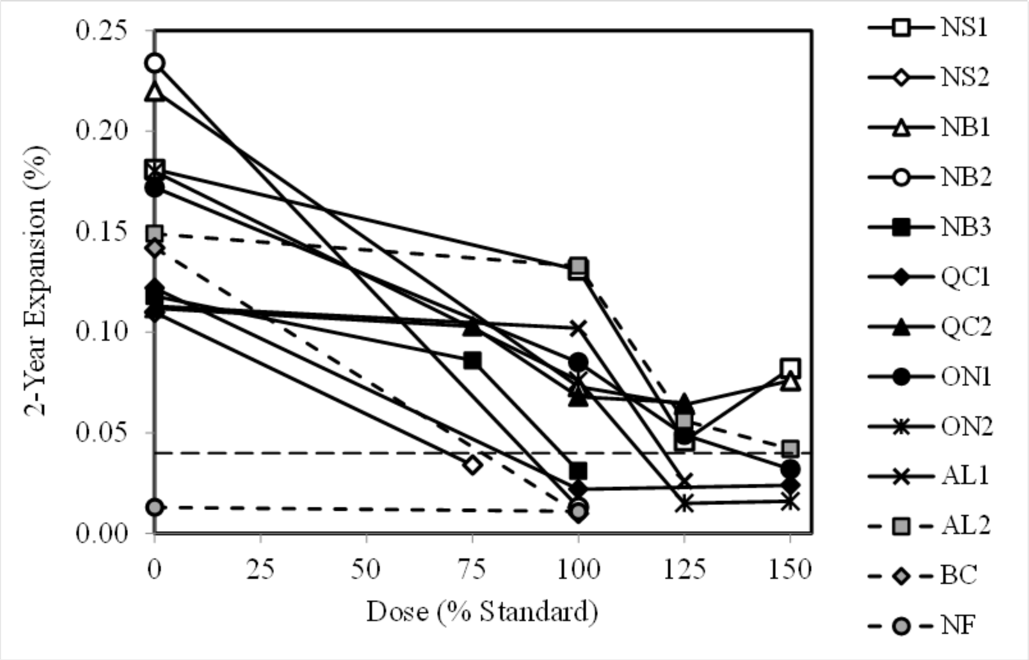

Figure 5.21. Effect of Lithium Dose on the Expansion of Concrete with Different Reactive Aggregates (from Tremblay et al. 2007)

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.

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6 – Alkali-Aggregate Reactions: Specifications

6.1 INTRODUCTION

This chapter 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 alkalies 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% alkalies calculated as sodium oxide equivalent ($Na_2Oe = Na_2O + 0.658 \times K_2O$),”
- there is a satisfactory service record evaluation, or
- 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[5]. 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 (≤ 0.60% Na_2Oe)
- 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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[5] 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 (≤ 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 (≤ 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 C441" 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)

The practice is based on the 2004 version of the practice developed in Canada: *CSA A23.2-27A Standard practice to identify degree of alkali-reactivity of aggregates and to identify measures to avoid deleterious expansion in concrete.* Further information on the development of this approach can be found in Report No. FHWA-HIF-09-001 (Thomas et al. 2008). The rationale for preventive measures recommended in the "prescriptive approach" is also documented in a report by the Federal Highway Administration (Thomas et al. 2012).

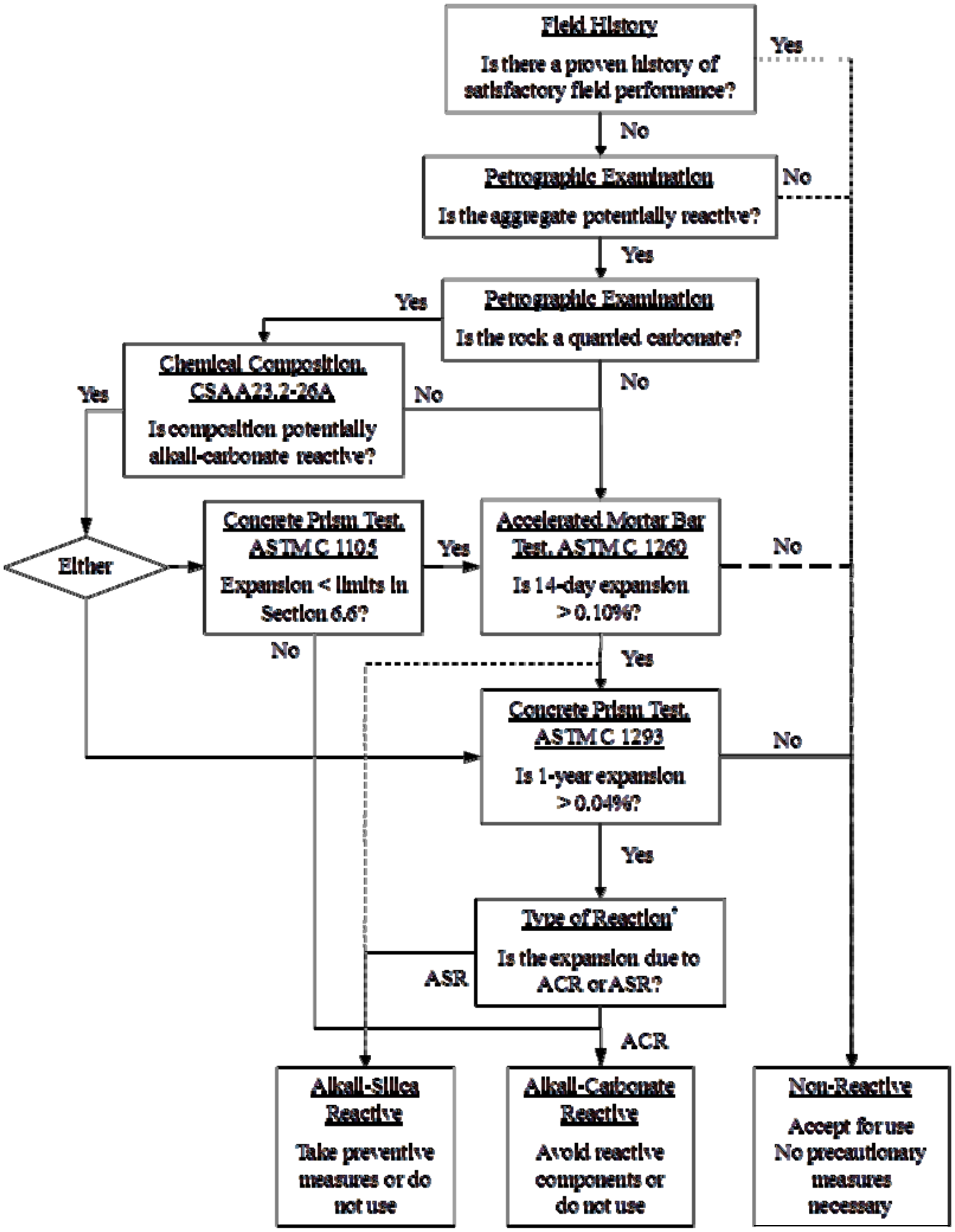
6.3.1 Evaluating Aggregate Reactivity

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). A flow chart showing the sequence of testing is presented in Figure 6.1. 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. 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.

Prior to expansion testing, quarried carbonates must be subjected to chemical analysis to determine whether there is a risk of alkali-carbonate reaction (ACR). The composition of the aggregate is plotted on a graph of CaO/MgO vs. Al_2O_3 (see Figure 6.2) and 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. Two options are allowed for the concrete prism test. One is to follow ASTM 1105 using a total alkali of no more than 1.8 kg/m^3 (3 lb/yd^3) Na_2Oe. If the expansion of the concrete prisms is equal to or greater than 0.025 % at 6 months or 0.030 % at 1 year, the aggregate shall be considered to be alkali-carbonate reactive and shall not be used in concrete. If the expansion of the concrete is below these limits, it is not considered to the alkali-carbonate reactive and can be evaluated in the same manner as non-carbonate rocks for alkali-silica reaction. The second option is to follow ASTM C 1293 (5.25 kg/m^3, 8.75 lb/yd^3 Na_2Oe). If the expansion of concrete prisms exceeds 0.040% at one year, the aggregate is deemed to be reactive and the concrete prism must be subjected to a petrographic examination to determine if ACR has contributed to the expansion. If ACR is considered to have contributed to the expansion the aggregate must be rejected for use in concrete; however, if there is no sign of ACR, the aggregate is deemed to be alkali-silica reaction and can be used in concrete with appropriate preventive measures. If the aggregate does not produce expansion in excess of 0.040% at 1 year in the concrete prism test it is considered to be non-deleteriously reactive and may be used in concrete without preventive measures.



†The type of reaction only needs to be determined after the concrete prism test if the aggregate being tested is a quarried carbonate that has been identified as being potentially alkali-carbonate reactive by chemical composition in accordance with test method CSA A23.2-26A.

The solid lines show the preferred approach. However, some agencies may want to reduce the amount of testing and accept a higher level of risk and this can be achieved by following the direction of the hashed lines.

Figure 6.1. Sequence of Testing in AASHTO PP65-11 for Evaluating Aggregate Reactivity

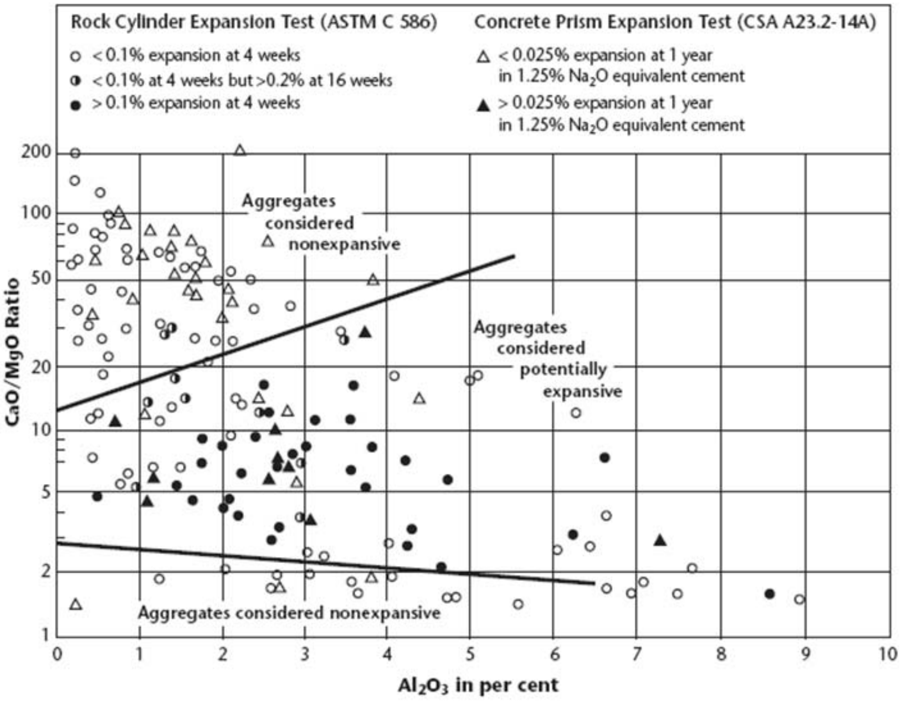


Figure 6.2. Potential for Alkali-Carbonate Reactivity of Quarried Carbonates based on Chemical Composition (from CSA A23.2-26A)

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 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[6], 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[7] (Table 6.6 and Table 6.7)

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

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.

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

[7] For Option 2, the minimum amount of SCM determined from Table 6.6 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, ≤ 0.12	> 0.10, ≤ 0.30
R2	Highly reactive	> 0.12, ≤ 0.24	> 0.30, ≤ 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

	Classification of Structure (Table 6.4)			
Level of ASR Risk (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	Acceptability of	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_2Oe)	
	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%)	$\leq$ 3.0	15	20	25	35	Table 6.7
	> 3.0, $\leq$ 4.5	20	25	30	40	
Slag	$\leq$ 1.0	25	35	50	65	
Silica Fume[2] ($SiO_2 \geq 85\%$)	$\leq$ 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_2Oe)	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

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 recommends that, before using ASTM C 1567 to determine the amount of SCM (or lithium required), the aggregate is tested in both the concrete prism test and the accelerated mortar bar test (ASTM C 1260) to ensure that the latter test gives a reliable prediction of the aggregate reactivity. The expansions from the two tests are plotted on the graph shown in Figure 6.3 to determine the validity of using the accelerated test.

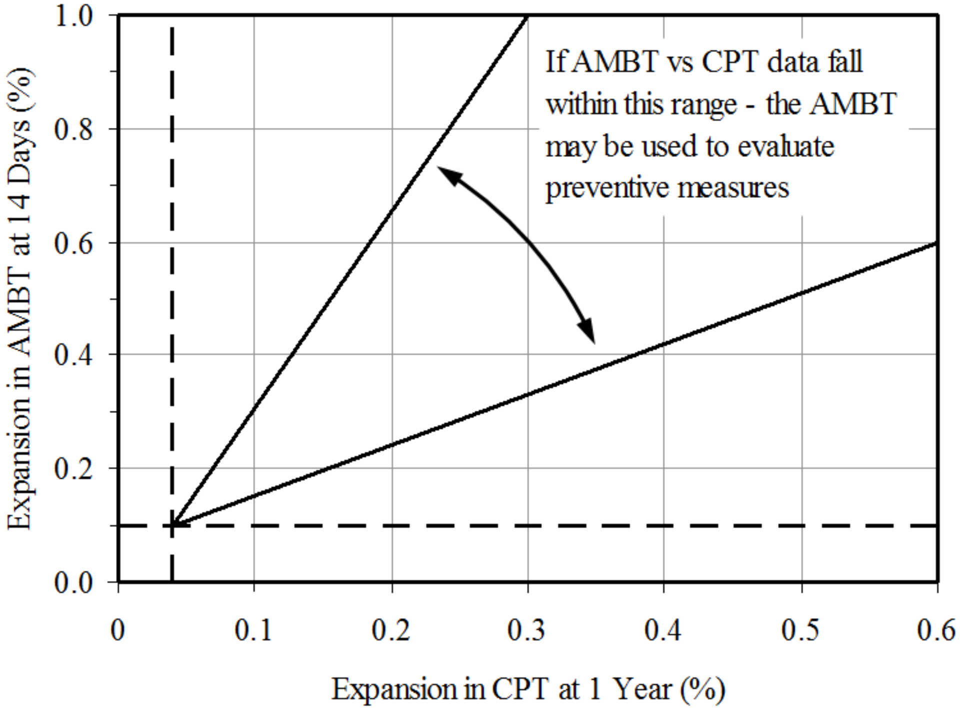

Figure 6.3. Comparison of Accelerated Mortar Bar Test (AMBT) and Concrete Prism Test (CPT) Data for the Purpose of Determining Whether the AMBT is Suitable for Evaluating Preventive Measures with a Specific Aggregate

AASHTO PP65-11 also includes a modified version of the accelerated mortar bar test for evaluating lithium-based compounds based on the approach proposed by Tremblay et al (2008). The approach is summarized as follows:

- Test the aggregate using the standard accelerated mortar bar test (AASHTO T 303, ASTM C 1260). Extend the duration of the test such that the mortar bars are exposed to sodium hydroxide for a period of 28 days. Let E1 = expansion of bars without lithium nitrate at 28 days.
- Test the aggregate in a modified version of the accelerated mortar bar test. In this test add sufficient lithium nitrate to the mortar bar mixture and the soak solution to achieve lithium-to-alkali molar ratios of [Li]/[Na+K] = 0.74 in the mortar and [Li]/[Na+K] = 0.148 in the soak solution (see Appendix A2 on calculation of lithium nitrate additions). Conduct the rest of the test in accordance with AASHTO T 303, extending the period in sodium hydroxide to 28 days. Let E2 = expansion of bars with lithium nitrate at 28 days.
  Note: To achieve [Li]/[Na+K] = 0.74 in the mortar add 4.6 liters of 30%-$LiNO_3$ *solution for every 1 kg of alkali (as* Na_2Oe*) in the mix (70.4 fl.oz. of 30%-*$LiNO_3$ *solution for every 1 lb of alkali).*

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.

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

7.1 INTRODUCTION

Alkali-silica reaction (ASR) 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 above components of the structure are subjected;
- the in-situ monitoring of deterioration (especially signs of expansion/deformation, extent and progress of cracking, spalling, etc.); 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 (especially those showing features of deterioration typically associated with or diagnostic of AAR).

7.2 MANAGEMENT PROGRAM FOR ASR-AFFECTED STRUCTURES

A global approach was recently proposed by FHWA for the diagnosis and prognosis of ASR in transportation structures (Fournier et al. 2010); the main features of the above protocol are illustrated in Figure 7.1. This step-by-step approach aims at evaluating the cause of concrete distress (**diagnosis**) and the progress or 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 proposed in the above approach will need to be implemented in a particular case will depend upon different factors, including the nature/extent of the problem, the criticality of the structure, the potential impact on the safety of users, etc.

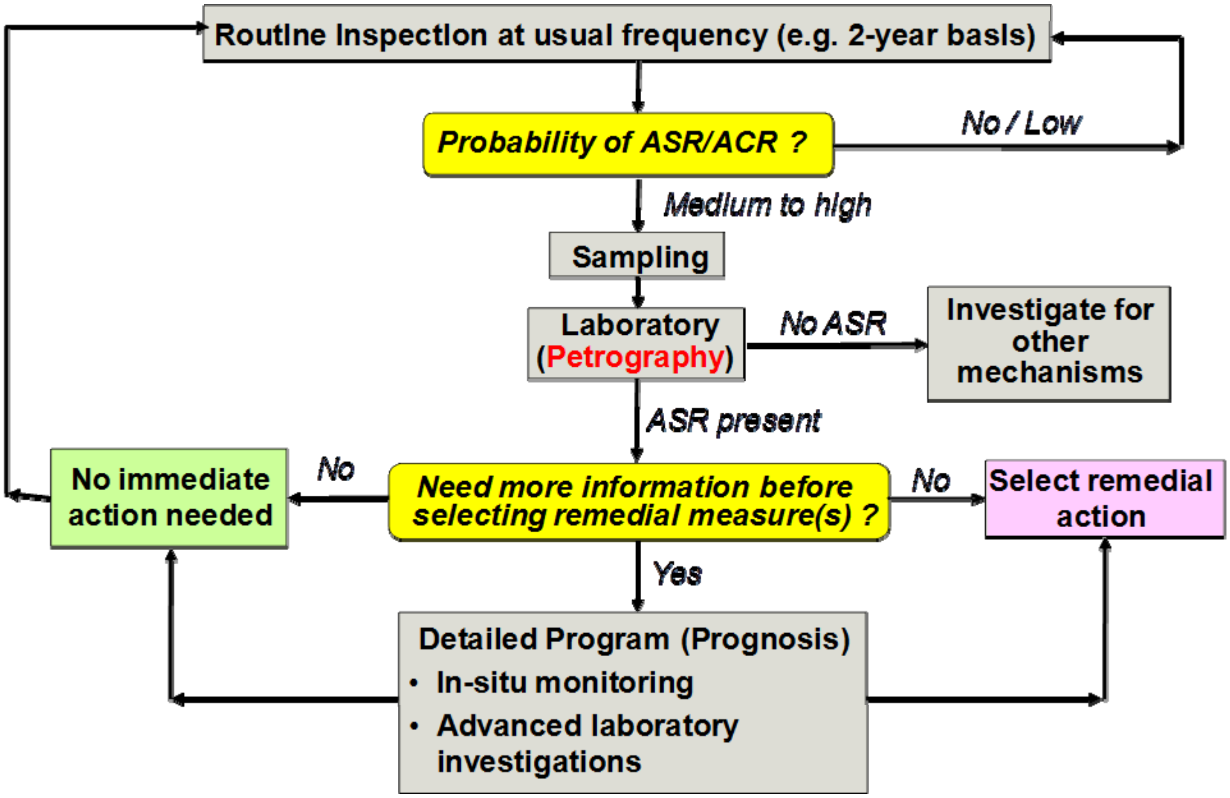


Figure 7.1. Global Approach for the Diagnosis and Prognosis of ASR in Concrete Structures (modified from Fournier et al. 2010)

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 of concrete structures (condition survey). For example, FHWA recently proposed *Alkali-Silica Reactivity Surveying and Tracking Guidelines* (Thomas et al. 2012a) that are intended to assist engineers and inspectors in tracking and surveying ASR-induced features of deterioration in bridges, pavements, and tunnels.

In the case of *bridge structures*, basic inspection data can first be collected for the various elements/components of the structure, for example in accordance with the new AASHTO Guide Manual for Bridge Element Inspection (AASHTO 2011). The concept is to identify defects that are specific/unique enough so the manifestation of distress may be attributed to ASR. The presence and extent of ASR-related defects are then noted and quantified in accordance with the four condition states described in Table 7.1. The *Alkali-Silica Reactivity Field Identification Handbook* (Thomas et al. 2011, updated from Stark (1991)) provides detailed coverage on the causes and effects of ASR and a wide range of photographs illustrating common features of ASR, as well as examples of condition states (or severity ratings) for a range of transportation structures. Such features 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). 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. For instance, concrete that is subjected directly to moisture and exposed to sun (i.e., wetting-drying cycles) is likely to exhibit more severe symptoms of ASR (e.g., cracking) than concrete that is less exposed to moisture in the same structure of even the same element (see chapter 3).

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.

Table 7.2. Recommended Defects and Condition States for Bridge Elements Potentially Affected by ASR (Thomas et al. 2012a)

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)	

Table 7.3. Recommended Defects and Condition States for Bridge Elements Potentially Affected by ASR (Thomas et al. 2012a)

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

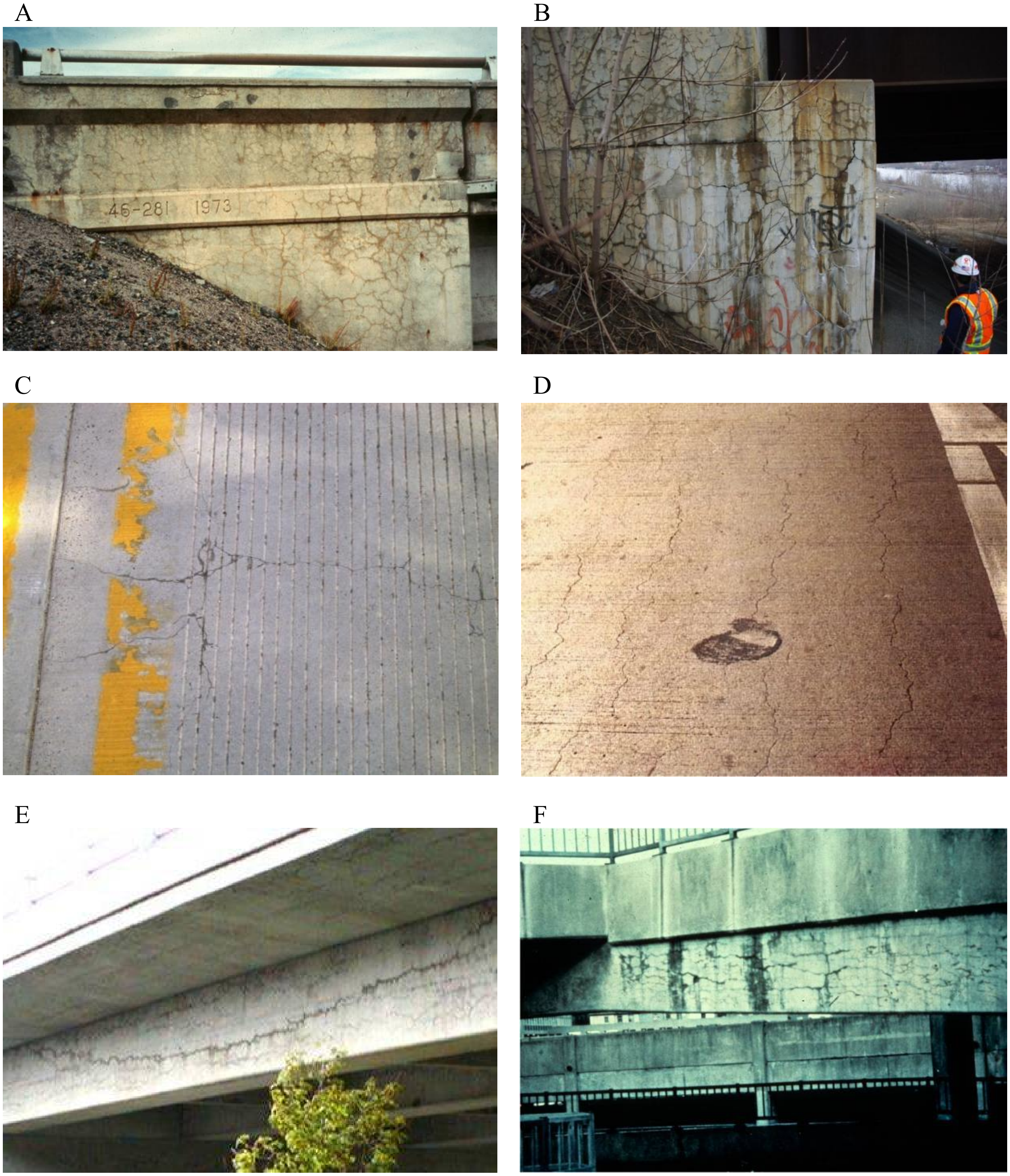


Figure 7.2. ASR-related defects in bridge elements and severity assessment. A. Map cracking on bridge abutment (condition state 2). B. Map cracking on bridge abutment (condition state 3). C. Map cracking in bridge deck (condition state 1). D. Aligned (longitudinal) cracking in bridge deck (condition state 2). E. Aligned (longitudinal) cracking in post-tensioned concrete bridge girder (condition state 2). F. Aligned (longitudinal) cracking in reinforced concrete beam (condition state 3).

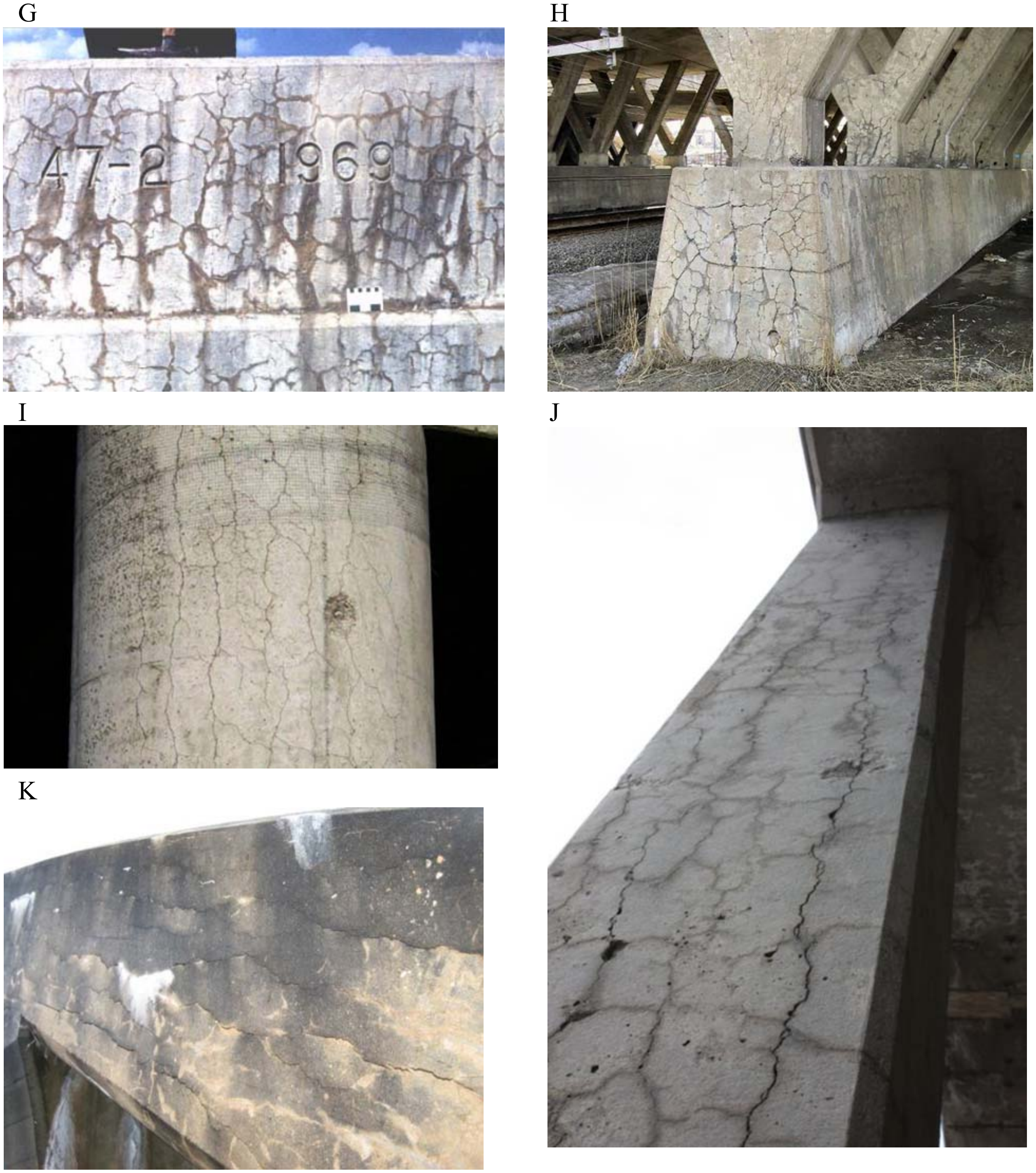


Figure 7.2 (cont'd). ASR-related defects in bridge elements and severity assessment. G. Map cracking on a parapet wall (condition state 3); moderate gel exudations are also present, which give the impression of permanent dampness. H. Map cracking in massive column foundation block of a bridge structure (condition state 3). I. Aligned (vertical/longitudinal) cracking in reinforced concrete column (condition state 2). J. Severe (condition state 3) aligned (longitudinal) cracking connected by moderate map cracking in a reinforced concrete column. K. Aligned (longitudinal) cracking in the top chord of a reinforced concrete arch bridge (condition state 3 to 4).

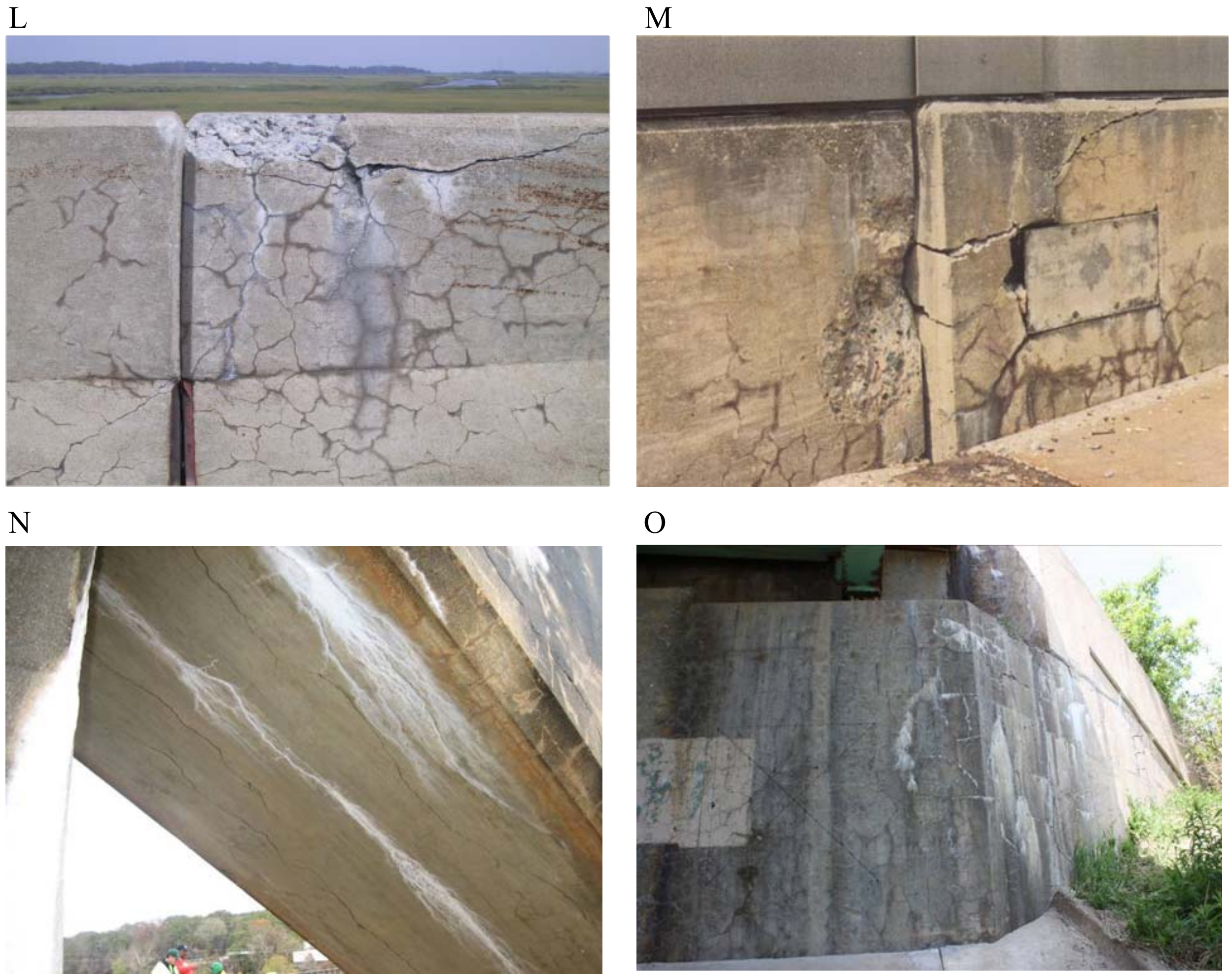


Figure 7.2 (cont'd). ASR-related defects in bridge elements and severity assessment. L. Closure of joint between adjacent parapet wall sections causing the buildup of concentrated stresses which led to crushing of the concrete. M. Severe relative dislocation/misalignment, leading to loss of clearance and localized spalling (photo from SHRP C-315 1991). N. Aligned (longitudinal) cracking with associated white exudations (mainly calcium carbonate with lesser amounts of potassium indicating both lime and alkali leaching from the concrete) on the underside of the arch shown in Figure 7.2K. O. Moderate aligned cracking and exudations in the exposed portions of abutment and wing wall of a bridge structure (condition state 2).

A similar approach can be adopted for pavements. The distresses most commonly associated with ASR in pavements include *map cracking*, *joint deficiencies/deterioration*, and, in some cases, *popouts*. The latter are generated by reactive aggregate particles causing expansion close to the surface of the pavement (or other concrete elements), thus inducing the detachment of a conical portion of the surface leaving the reactive aggregate in the bottom (Figure 7.3J). Popouts can also be caused by the expansion of frost-susceptible aggregate particles (such as laminated, schistose and argillaceous, clayey or porous particles or certain varieties of chert, ironstones), as well as by a poor bond between the cement paste and dusty coarse aggregate particles.

Table 7.4. Recommended Distress and Condition States for Pavements Potentially

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	

Affected by ASR (Thomas et al. 2012a)

Table 7.5. Recommended Distress and Distress Description for Pavements Potentially Affected by ASR (Thomas et al. 2012a)

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 []*

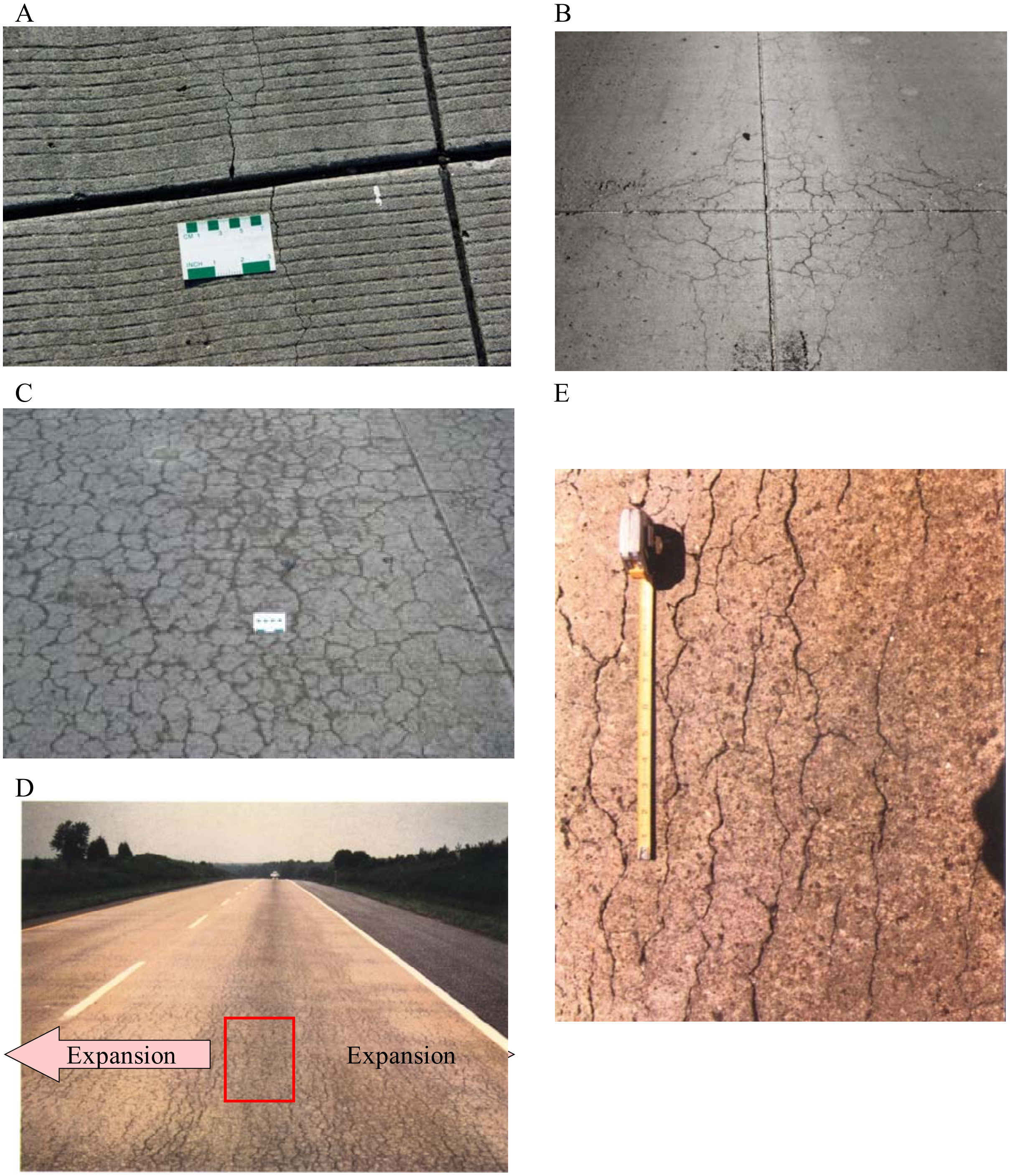


Figure 7.3. ASR-related defects in pavements and severity assessment. A. Minor cracking in a concrete pavement (condition state 1). B. Cracking at the corner of paving slabs (condition state 2). C. Map cracking of concrete pavement; cracking is much more evident after rainy periods (condition state 2). D. Longitudinal cracking connected by map cracking in severely damaged concrete pavement (condition state 3) (photo from SHRP C-315 1991). E. Enlargement (red rectangle) of a section of the pavement in figure D showing the aligned (longitudinal) cracking in pavement with connecting map cracking.

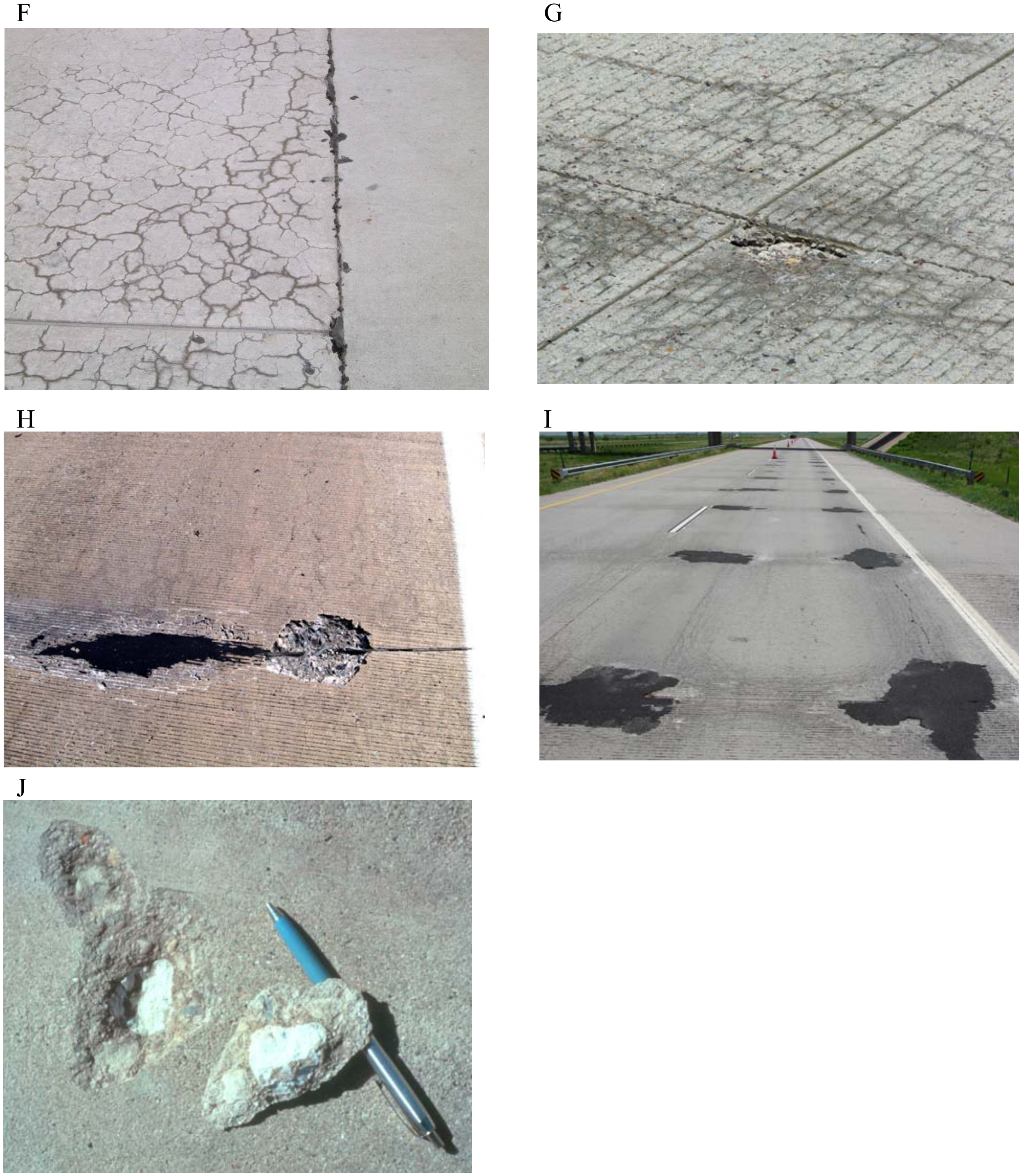


Figure 7.3 (cont'd). ASR-related defects in pavements and severity assessment. F. Expansion (with related map cracking) in pavement slabs causing the extrusion of joint sealant material. G. Spall occurring at a joint in a concrete pavement; as ASR advances the cracked concrete at the vicinity of the joint may be further distressed by the action of freezing and thawing, and vehicular loading. Eventually spalling starts to occur at the joint. H. Joint Sealant Failure in Concrete Pavement (condition state 3). I. Joint deterioration in concrete pavement (condition state 3). J. Popout involving a porous (leached) chert particle (with complements of C.A. Rogers).

It is important to mention that 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.

Conclusions from the Condition Survey

Signs of premature deterioration in concrete pavement and bridge structures that could be related to ASR can generally be detected during routine site inspections that are performed regularly by trained personnel (e.g., inspectors from State Highway Authorities) (Figure 7.1). Visual symptoms of deterioration are noted and compared to those commonly observed on structures affected by ASR (Figures 7.2 and 7.3).

It is often difficult to determine only from field observations whether ASR is the only/main factor responsible for the observed distresses since some of the visual signs of deterioration generally associated with ASR may have been caused by other processes such as internal sulphate attack, or plastic or drying shrinkage. Table 7.6 classifies the occurrence of the features obtained from the condition survey as indicative of low, medium, and high potential of ASR contribution in the deterioration observed. The assessment of the exposure conditions should also contribute to support the observations of the symptoms of distress listed in Table 7.6, as follows:

- Low potential for ASR: Element in dry and sheltered environment.
- Medium potential for ASR: Element exposed outdoors but sheltered from constant wetting.
- High potential for ASR: Parts of components frequently exposed to moisture–e.g., rain, groundwater, or water due to natural function of the structure (e.g., hydraulic dam or bridge).

Table 7.6. Classification System for the Condition Survey (modified from CSA A864-00)

Feature	**Probability for ASR**		
	Low	**Medium**	**High**
Expansion and/or displacement of elements	None	Some evidence (e.g., closure of joints in pavements, jersey barriers, spalls, misalignments between structural members)	Fair to extensive signs of volume increase leading to spalling at joints, displacement and/or misalignment of structural members
Cracking and crack pattern	None	Some cracking pattern typical of ASR (e.g., map cracking or cracks aligned with major reinforcement or stress)	Extensive map cracking or cracking aligned with major stress or reinforcement
Surface discoloration	None	Slight surface discoloration associated with some cracks	Many cracks with dark discoloration and adjacent zone of light colored concrete
Exudations	None	White exudations around some cracks; possibility of colorless, jelly-like exudations	Colorless, jelly-like exudations readily identifiable as ASR gel associated with several cracks

If the probability of ASR is low or no visual signs suggestive of ASR are noted during the routine inspection program, further work can be postponed until the next inspection. However, when the visual signs of deterioration observed on the structure(s) examined are such that AAR is a possibility, sampling of the structure under evaluation is recommended to confirm the first diagnostic obtained from the visual survey.

7.2.1.2 Documentation

Any documents (i.e., testing reports of materials, construction and/or 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 the nature of operation(s) 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 "original" 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 the inspection of other local structures that may have been constructed with 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 details in the case of many structures; however, it is important to collect whatever data are available.

7.2.1.3 Sampling of Structural Components

At this stage of the program, sampling is carried out on component(s) of the structures showing typical defects suggestive of ASR, which most often correspond to structural components exposed to a constant or renewable supply of moisture, with/without cycles of wetting and drying (e.g., Figure 7.4A-7.4C). This is essentially done to determine whether or not the concrete contains petrographic evidence of ASR. For comparison purposes, it will also be useful/appropriate to collect cores from structures/components that are less deteriorated than the structure in question, not deteriorated, or not exposed to the severe environmental elements.

Figure 7.4. A-C. Sampling of concrete structures/elements affected by ASR for laboratory investigations (e.g., petrographic examination, mechanical testing). D. Cores are properly identified and wrapped in plastic film/bag to avoid moisture loss.

The samples collected should be labeled carefully, photographed, and, immediately after their preliminary examination (for obvious signs of degradation….), wrapped in a plastic film and sealed in a plastic bag to prevent alkali-silica gel and surfaces to carbonate, become contaminated, or dry out during subsequent transport and storage (Figure 7.4D). More details regarding sampling (i.e., type and sample size) and preliminary treatment/analysis of samples can be found in Fournier et al. (2010).

7.2.1.4 Diagnosis – Petrographic Examination

Petrographic examination is a powerful technique in the diagnosis of the cause of concrete deterioration. ASTM C 856 outlines procedures for the petrographic examination of samples of hardened concrete. Interesting information regarding petrographic features of ASR-affected concrete can be found in several publications, including BCA (1992), St. John et al. (1998), CSA (2000), Walker et al. (2006) and Fournier et al. (2010).

Macroscopic Features of ASR

Macroscopic signs of concrete deterioration due to ASR (but sometimes not exclusively) can be detected by examining the cores immediately after the extraction or in the laboratory in an *as received* condition. Such features can consist of (Figure 7.5): macrocracks in the outer portion (or "skin") of the concrete member turning into microcracking in the "internal" part of the concrete member, gel staining surrounding surface cracks, dark reaction rims at the periphery of reacted aggregate particles, cracks within reactive aggregate particles sometimes extending into the cement paste (with/without reaction products gels), alkali-silica gel in air voids of the cement paste, and deposits of reaction products on the cracked surfaces of cores.

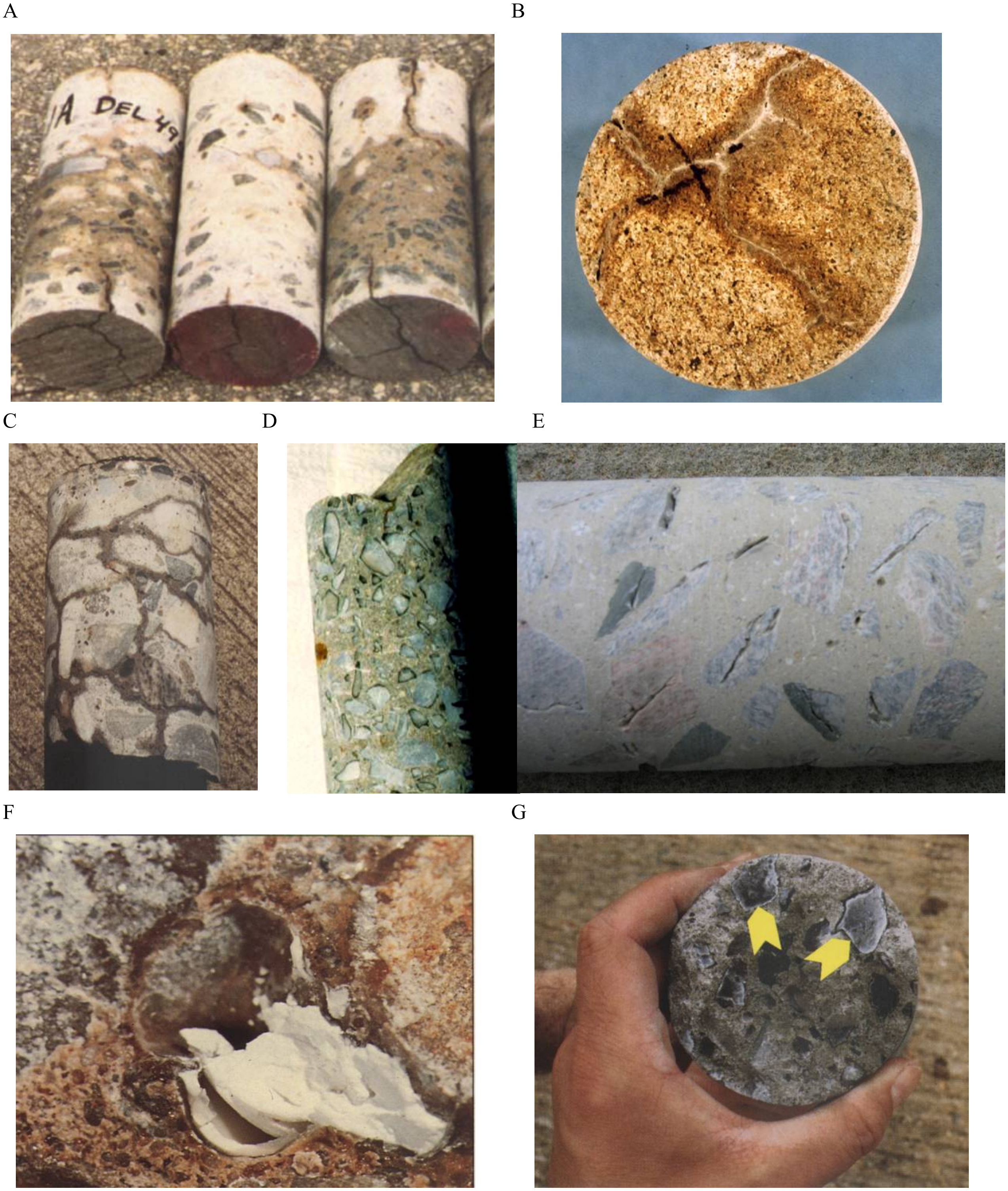


Figure 7.5. Macroscopic observations on cores (as received condition). A. Cores extracted from a concrete pavement affected by ASR and showing macrocracks penetrating from the upper and lower parts of the pavement. B. Gel staining surrounding cracks and gel exudations at the surface of a core extracted from a sidewalk section affected by ASR. C. Fine cracking pattern showing up after re-wetting of the core. D. Dark rim surrounding reactive aggregate particles. E. Macrocracks in reactive coarse aggregate particles. F. Deposits of alkali-silica gel in a void on core surface. G. Deposits of alkali-silica reaction products on the broken surface of a core extracted from a highway bridge structures affected by ASR.

Microscopic Features of ASR

Petrographic features of ASR can be detected on various types of specimens prepared from the drilled cores (e.g., polished sections or slices, broken (fresh) surfaces, and thin sections). Although not necessarily exclusive to ASR, these features generally consist of the following:

- Microcracking in aggregates and/or cement paste
- Reaction product “gel”
- *Reaction rims*
- Loss of the cement paste-aggregate bond

Microcracking - BCA (1992) and St. John et al. (1998) compared “idealized” cracking patterns in concrete specimens affected by various deleterious mechanisms (Figure 7.6). In the case of concrete affected by ASR when the reactive fraction is part either of the fine aggregate (Figure 7.6E) or coarse aggregate (7.6F) fraction, a network of microcracks develops in the inner part of the concrete, with only a few “macrocracks” being observed in its outer (surficial) portion. The microcracks are found connecting the aggregate particles; when the reactive material is found in the coarse aggregate particles, cracks typically run through the particles (Figure 7.6F).

In the early stages of the reaction, microcracks are generally limited to the reacting aggregate particles and the cement paste-aggregate interface. With the progress of expansion, microcracks, more or less filled with alkali-silica gel, will extend from the aggregate particles into the cement paste (Figures 7.7A-C); depending on the extent of expansion, the cracks will cover considerable distances through the paste where they are often filled with secondary reaction products (Figures 7.7A, 7.7D, 7.7E). In badly deteriorated concrete specimens, cracks, even filled with gel, may run through non-reactive aggregate particles. Consequently, great care should be taken to correctly identify the sites (or the aggregate particles) that have generated the expansive forces.

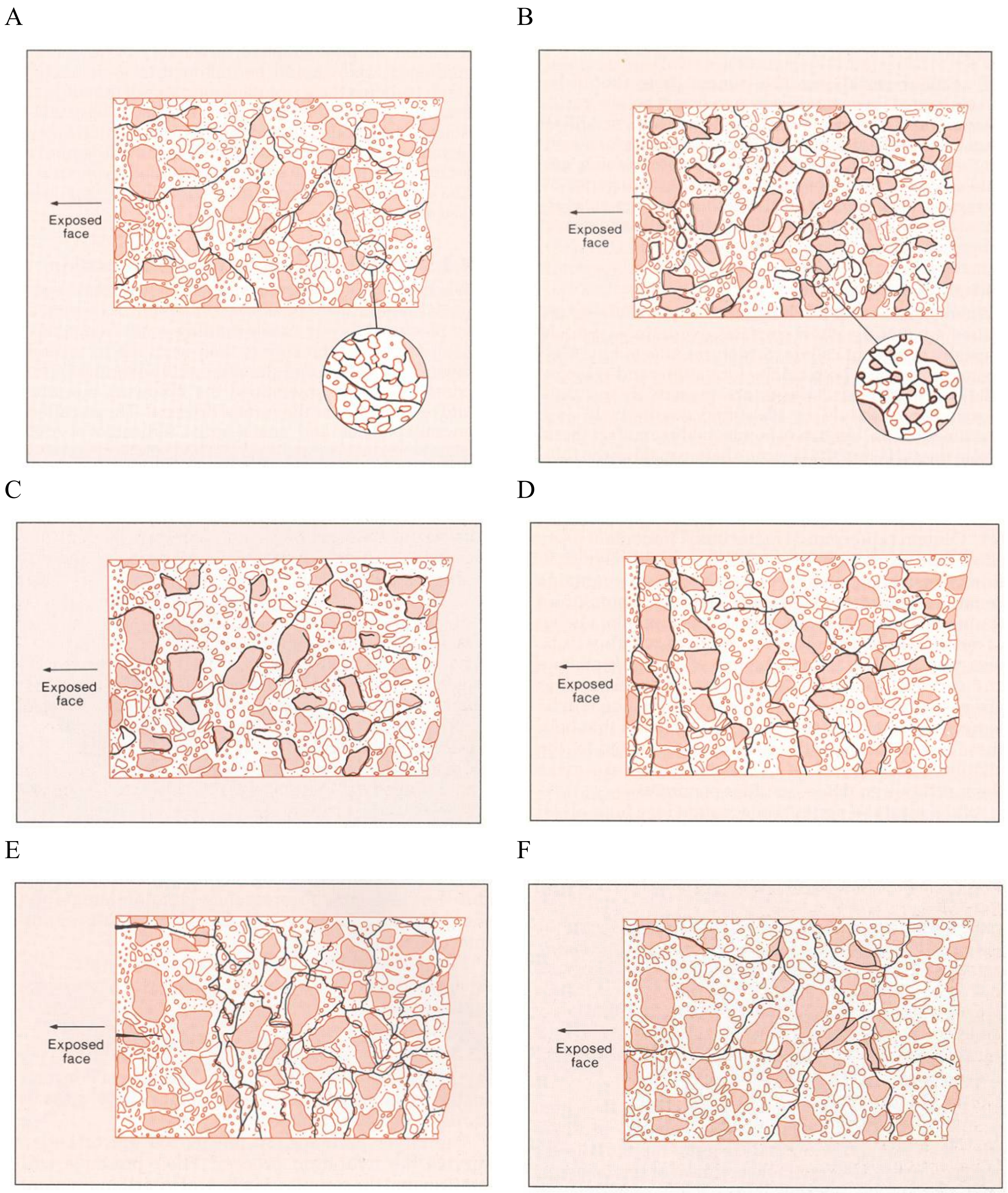
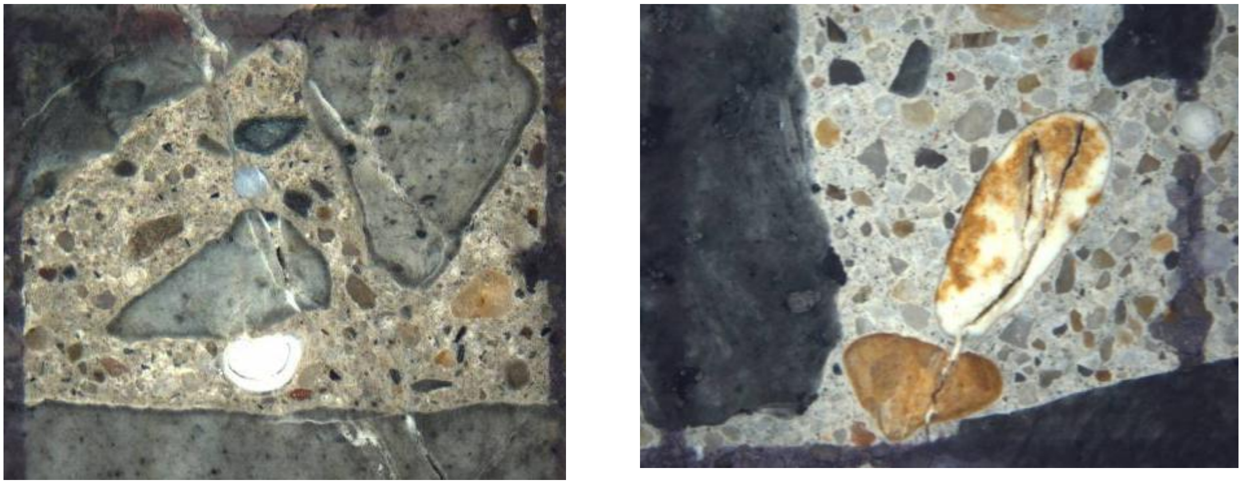


Figure 7.6. Typical cracking patterns in concrete affected by different deleterious mechanisms (from BCA 1992). A. Internal crack pattern which can be induced by drying shrinkage. B. Internal crack pattern which can be caused by internal sulfate attack for delayed ettringite formation (DEF), or from sulfates derived from the aggregates. C. Internal crack pattern which can be induced by shrinkage of the coarse aggregate. D. Internal crack pattern which can be induced by frost attack. E. Internal crack pattern which can be caused by ASR: reactive silica in the sand fraction. F. Internal crack pattern which can be caused by ASR: reactive silica in the coarse aggregate.

A

B (distance between the vertical lines = 1 cm) C (distance between the vertical lines = 1 cm)



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

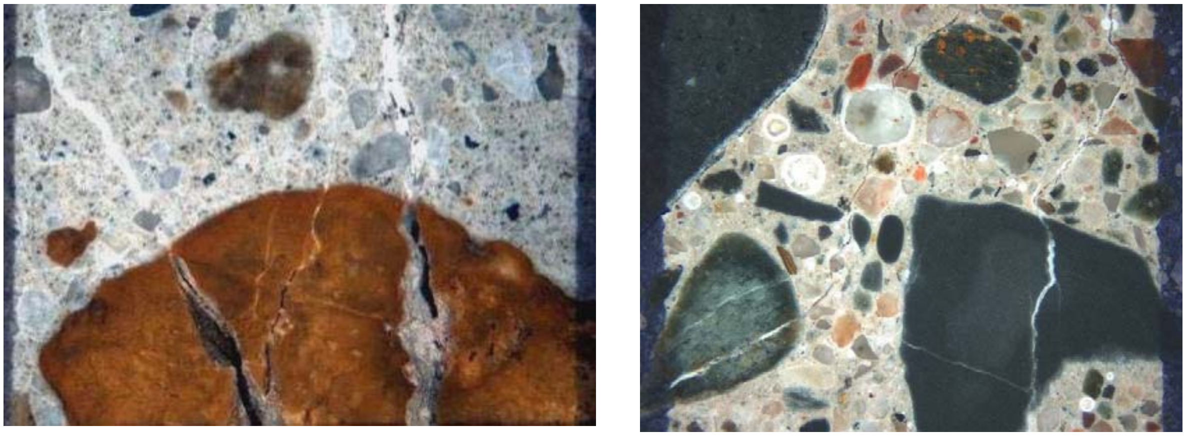


Figure 7.7. Petrographic features of ASR observed on polished concrete sections. Cracks with reaction products in coarse (A, C, E) and fine (C) aggregate particles. Cracks running from one aggregate particle to another through the cement paste (A-C). Reaction rims around reactive coarse aggregate particles (E, B). Reaction products (ASR gel) in cracks (B, D) and voids (A, B, E) of the cement paste.

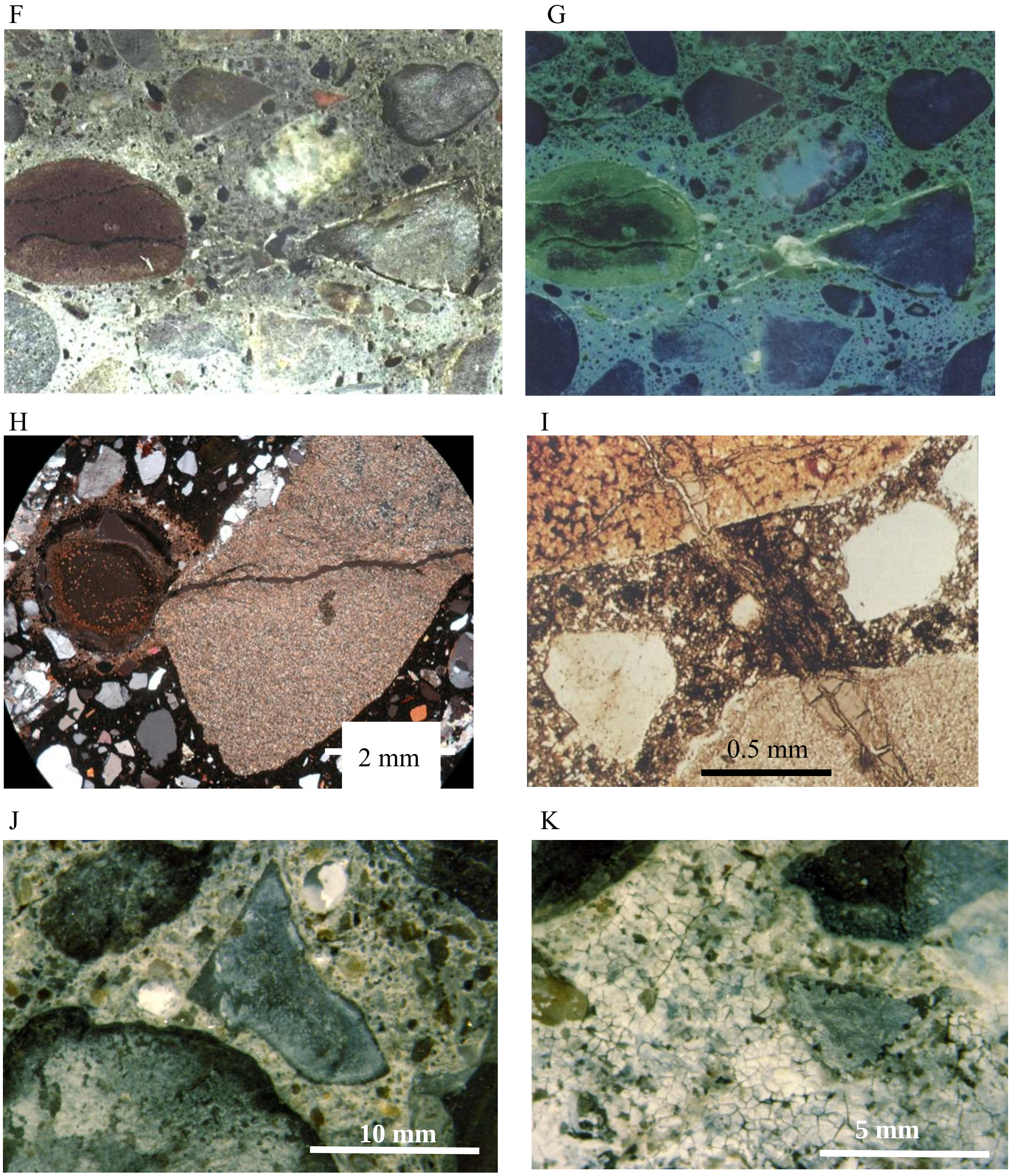


Figure 7.7 (cont'd). Petrographic features of ASR. F&G. Polished concrete sections treated with uranyl acetate solution to enhance the presence of alkali-silica gel (F. natural light; G. under UV illumination showing the gel in greenish-yellow staining color filling cracks in the cement paste in the vicinity of reactive aggregate particles). H&I. Thin sections micrograph showing reaction products (ASR gel) in cracks of coarse aggregate particles and of the cement paste (H), as well as in an adjacent air void of the cement paste (I). J&K. Micrographs of concrete sample under the stereomicroscope showing alkali-silica reaction products on broken surfaces of reactive coarse aggregate particles and in voids (J), or covering cracked surfaces of the cement paste (K).

Reaction product “gel” - ASR generates secondary reaction products containing silica, alkalis and calcium as typical constituents. The so-called “alkali-silica gel” will be found filling cracks within the aggregate particles (Figures 7.7B-7.7E), lining or filling voids (Figures 7.7B,7.7H,7.7J) and fractured surfaces of the cement paste and the aggregate particles (Figures 7.7J,7.7K). These deposits will cover more or less important surfaces depending on many factors, such as the extent of the reaction-expansion processes that have occurred, the availability of water, etc. However, the abundance of gel deposits is not necessarily indicative of the magnitude of any resultant expansion and cracking (BCA 1992); cracking due to ASR has been observed in many concrete structures while very little gel was found in concrete specimens taken from the affected members.

The confirmation of the presence and the nature of reaction products is not easy. This is often done under the scanning electron microscope equipped with an energy dispersive X-Ray analyzer (Fournier et al. 2010). Staining techniques have also 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). Natesaiyer et al. (1991) proposed a method that consists of 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. 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” (Figures 7.7F-7.7G). This technique, which has also been applied to field structures (Stark 1991; AASHTO 1993; ASTM C 856-02), should be used with great care following appropriate health and safety procedures because of the potentially hazardous nature of the product. Also, 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. Yellow stained regions would correspond to alkali-bearing siliceous reaction products resulting from ASR.

Reaction rims - Dark reaction rims are observed at the internal periphery of a number of alkali -silica reactive aggregates in deteriorated concrete specimens; these are particularly evident on polished sections or slabs of affected concrete cores (Figures 7.7B,7.7E). However, these rims must not be mixed up with dark rims that are often found in the outer (but also internal) portion of "weathered" gravel particles. When concrete cores are fractured for examining “fresh” broken surfaces, cracks that have formed within the aggregate particles and the cement paste, due to the ASR processes, will form zones of weakness where the core will preferentially break. The fractured surfaces thus created (which in many cases correspond to "ASR cracking surfaces") often show a dark rim surrounding internal deposits of whitish color (Figure 7.7J). Such a feature does not correspond to a "reaction rim" per se; it actually corresponds to a typical arrangement of reaction products *deposited* on the cracking surface, i.e., 1) a dark rim covering the immediate internal periphery of the particle, and 2) white deposits going through the central portion of the particle showing a powdery aspect.

Loss of the cement paste-aggregate bond - The interfacial region between the cement paste and the aggregate particles certainly represents, because of its nature and the arrangement of hydrates that form herein, a preferential zone of weakness where cracks will initiate and run. Loss of the cement paste-aggregate bond has been reported as a petrographic consequence but is not necessarily indicative of AAR.

Conclusions from the Petrographic Examination

Table 7.7 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 (Figure 7.1). 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 desirable/ necessary to determine the potential for the progress of ASR in the structure (prognosis).

Table 7.7. 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

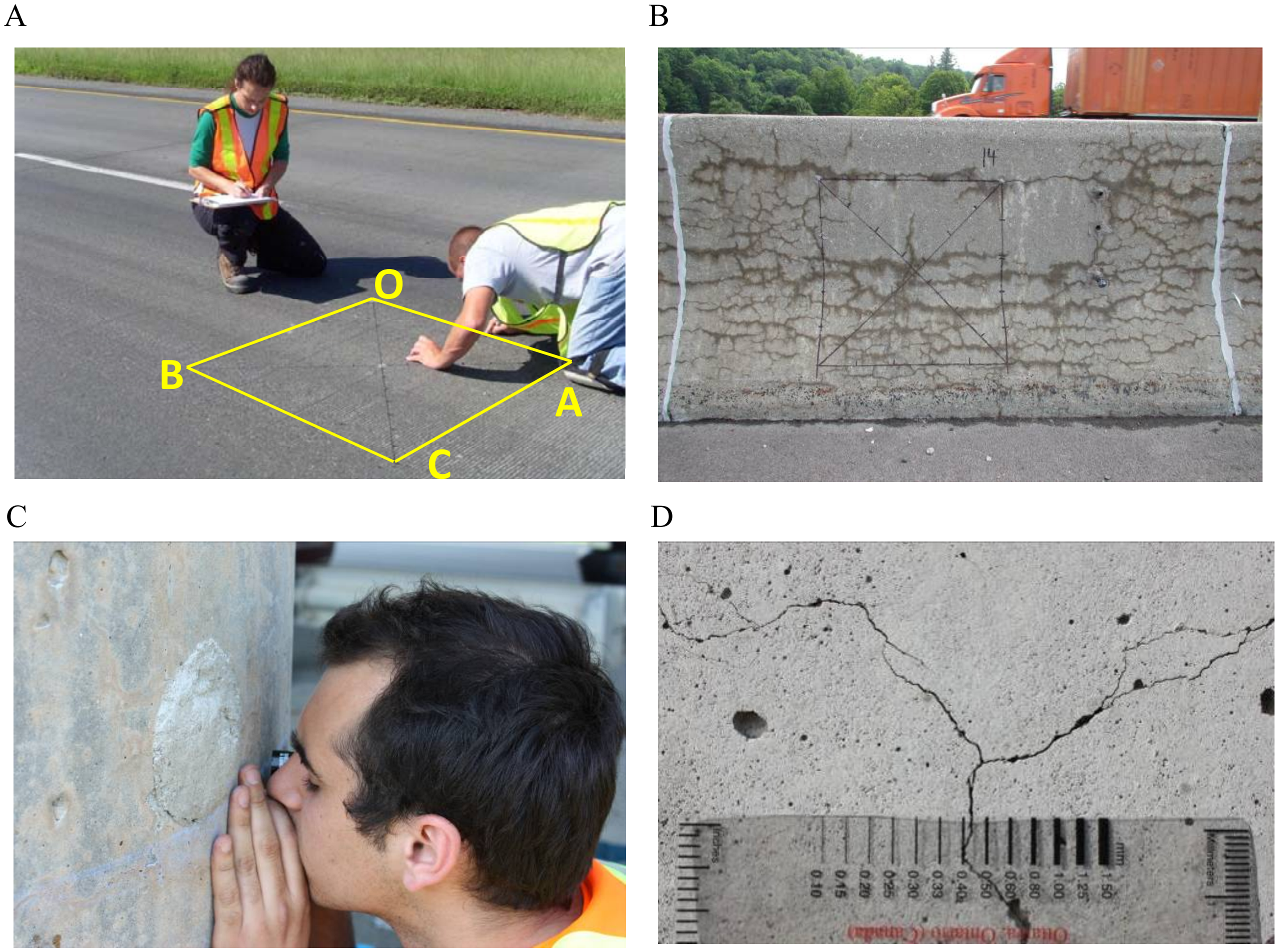
This part of the study aims at generating additional technical information, leading to a more complete assessment of the degree of damage due to ASR in the concrete structure and the selection of most appropriate remedial measures (Figure 7.1). The selection of the activities will depend on the criticality 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 could include monitoring of expansion, deformation, availability of moisture (to account for seasonal temperature changes), and non-destructive testing, can provide “prognostic” information for ASR-affected structural members when carried out at regular intervals over several years. The in-situ investigation program can include one or several of the activities described hereafter. Details on the procedures/methods can be found in Fournier et al. (2010).

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 of 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 7.8).



E

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 7.8. Measurements of the *Cracking Index* on ASR-affected pavement (A) and jersey barrier (B). The cracks can be examined with a magnifying lens (C). Using a transparent crack card to measure their width along the measurement lines (D). An example of the data generated in the *Cracking Index method* is given in (E); the values of C.I. are given separately for the vertical and horizontal measurements.

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).

For example, Francoeur et al. (2012) used the CI method to study the progress of expansion and cracking in metric size concrete blocks exposed outdoors and suffering from deleterious aggregates reactions (oxidation of sulphide-bearing aggregates). The authors found a good correlation between the expansion of the concrete blocks and the progress of the surface cracking measured through the CI Method (Figure 7.9).

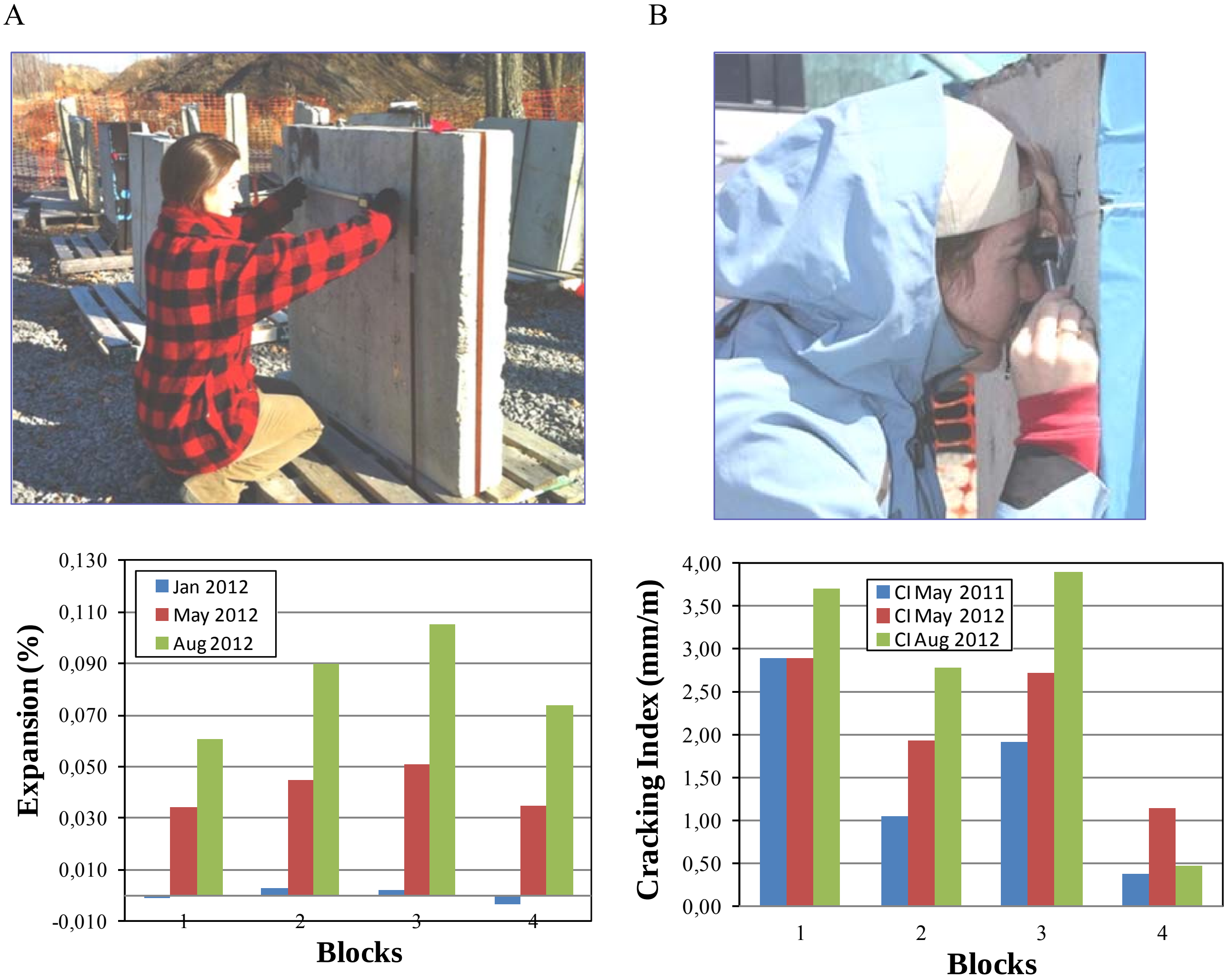


Figure 7.9. Measurements of Expansion and *Cracking Index* on Concrete Housing Foundations Suffering from Deleterious Aggregate Reactions (Francoeur et al. 2012)

Expansion and Deformation Measurements

Measurements of expansion/deformation can be performed using different means (e.g., demec points, metallic references drilled into 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 7.10). Fiber-optic and vibrating wire systems can also be used, deformation measurements being performed and the data transmitted automatically to central servers for further treatment. On a larger scale, instruments such as inclinometers and inverted pendulums can be installed in strategic parts of the structure to evaluate the relative movement, deflections, clearances at joints, etc. (e.g., Thompson et al. 1995; Danay et al. 1993; Gaudreault 2000).

Figure 7.10. In-situ measurements of expansion and deformation in ASR-affected concrete elements. A-E. Length-change measurements in a concrete highway barrier wall, reinforced concrete columns and a concrete pavement affected by ASR. F-G. Invar-bar extensometer used for the in-situ monitoring of deformation/movement associated with ASR (Gaudreault 2000).

Temperature and Humidity Measurements

The relative humidity is one of the essential conditions to maintain ASR in a concrete structure; it 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 7.11). The monitoring of relative humidity provided very useful information for the understanding of the beneficial effect of silane treatments on median barriers affected by ASR (Bérubé et al. 2002a).

Figure 7.11. In-situ measurements of humidity in concrete. A. Wooden-stick method. B-C. Portable humidity probes. D. Automatic in-situ monitoring of temperature, humidity and expansion (vibrating wire) in a bridge deck affected by ASR (Siemes and Gulikers 2000).

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 7.12).

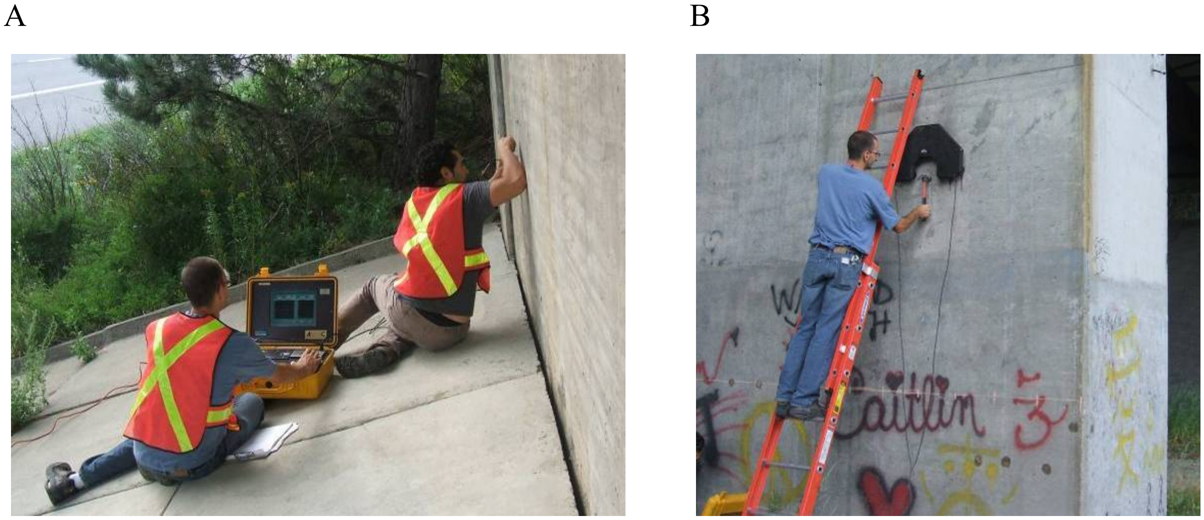


Figure 7.12. Non-destructive testing of ASR-affected concrete structures. A. Impact-echo. B. Non-linear acoustic technique.

7.2.2.2 Advanced 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 in a grid system drawn at the surface of polished concrete sections (Grattan-Bellew 1992) (Figure 7.13). 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 (Grattan-Bellew and Mitchell 2006) (Table 7.8). The method gives a quantitative assessment of the extent of internal damage in structural concrete members, exposed or not to moisture, either punctually (i.e., at a specific time - diagnosis) or as a progressive process when performing the measurements on a regular basis on cores extracted at the same location (prognosis).

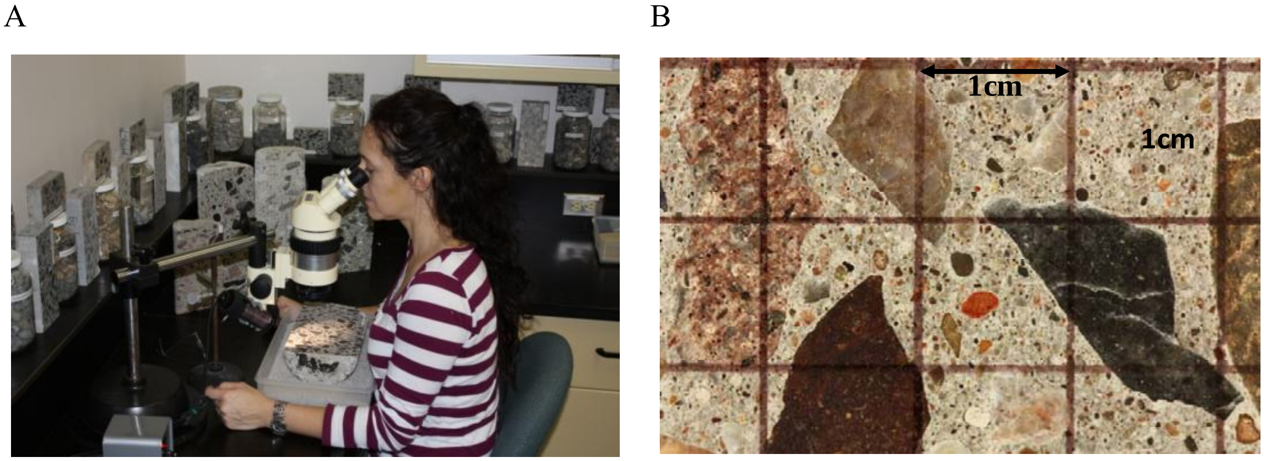
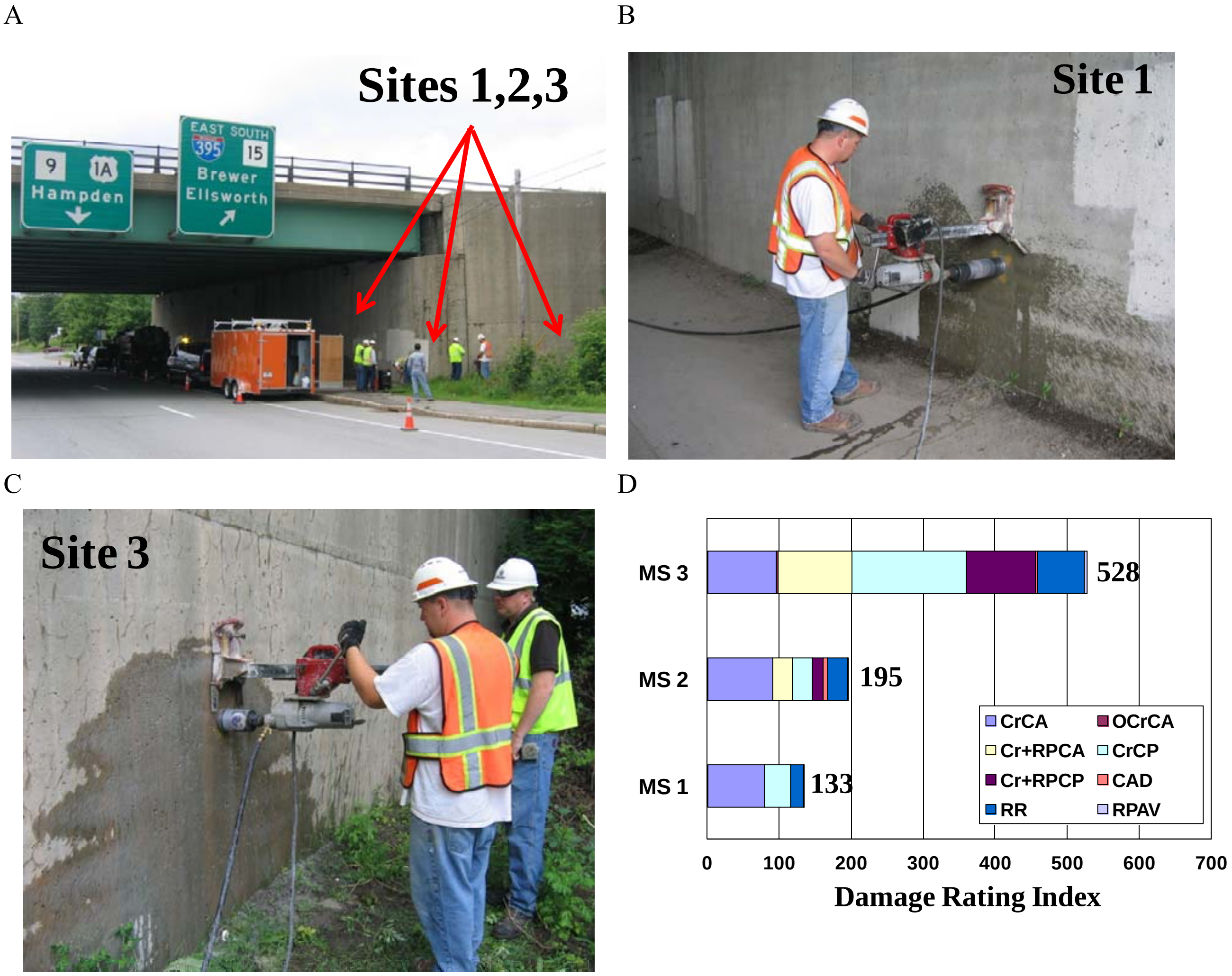


Figure 7.13. Examination of the polished concrete section under the stereomicroscope for the determination of the *Damage Rating Index*. Petrographic features of deterioration (Table 7.8; Figure 7.7) are counted in a one cm x one cm grid system drawn at the surface of the polished concrete section.

Table 7.8. 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

Thomas et al. (2012) found a strong correlation between the extent of ASR (obtained from the DRI) and the exposure conditions of structural bridge members (Figure 7.14). Rivard et al. (2000) used the method to estimate the amount of expansion reached by concrete specimens cored from a large concrete dam affected by ASR.



E (distance between the vertical lines = 1 cm)

F (distance between the vertical lines = 1 cm)

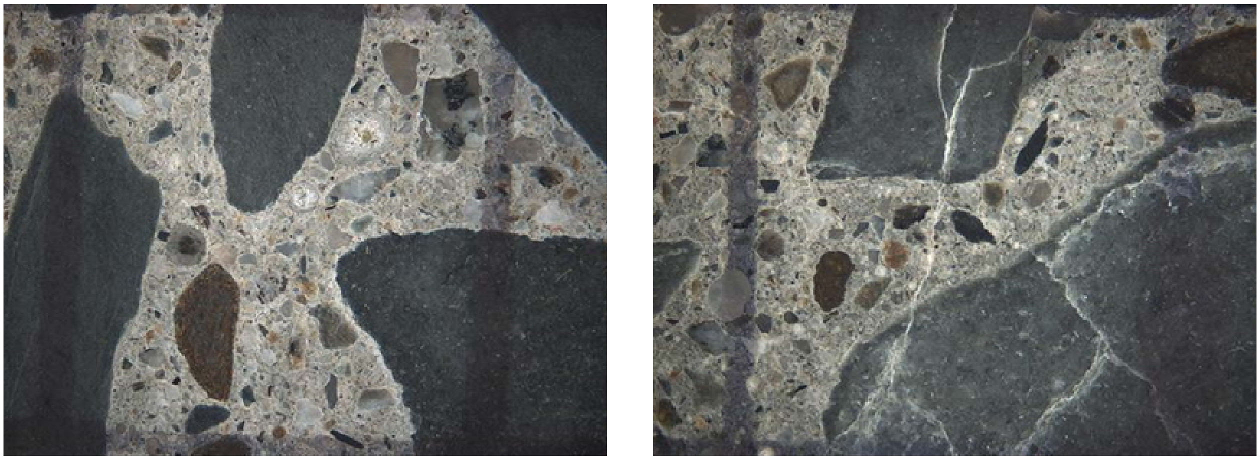


Figure 7.14. A. Highway bridge affected by ASR in Bangor, Maine. Coring sites in the abutment (not exposed (site 1 - B) and the wing wall (exposed (site 3 - C)). D. DRI results for cores obtained from sites 1 to 3; the higher the DRI, the higher the degree of ASR damage in the concrete. E. Polished section from site 1 showing very limited signs of deterioration. F. Polished section from site 3 showing extensive cracking filled with alkali-silica gel in aggregate particles and the cement paste.

Mechanical Testing

Several authors have shown that tensile strength is a much better indicator of the progression of ASR than compressive strength (Blight et al. 1981; Swamy and Al-Asali 1986; Clayton 1989; ISE 1992; Siemes and Visser 2000; Smaoui et al. 2006) which is generally affected only at much higher expansion levels (Figures 7.15 and 7.16). Direct tensile strength (traction tests) is also more sensitive to ASR than splitting tensile (Brazilian) strength (Clayton 1989; ISE 1992; Siemes and Visser 2000; Smaoui et al. 2006). The use of the tensile-to-compressive strength ratio was suggested as a good indication of internal damage due to AAR (Nixon and Bollinghaus 1985). The tensile-to-compressive strength ratio of sound concrete typically varies from 0.07 to 0.11. In investigations dealing with AAR, it was suggested that a ratio less than 0.06 was indicative of internal deterioration due to ASR. Since the ultimate compressive strength of concrete supplied in the field is generally greater than the designed 28-day strength, a reduction in compressive strength due to AAR is unlikely to be critical in current practice, and failure through loss in compressive strength is also unlikely (ISE 92; Swamy 1995).

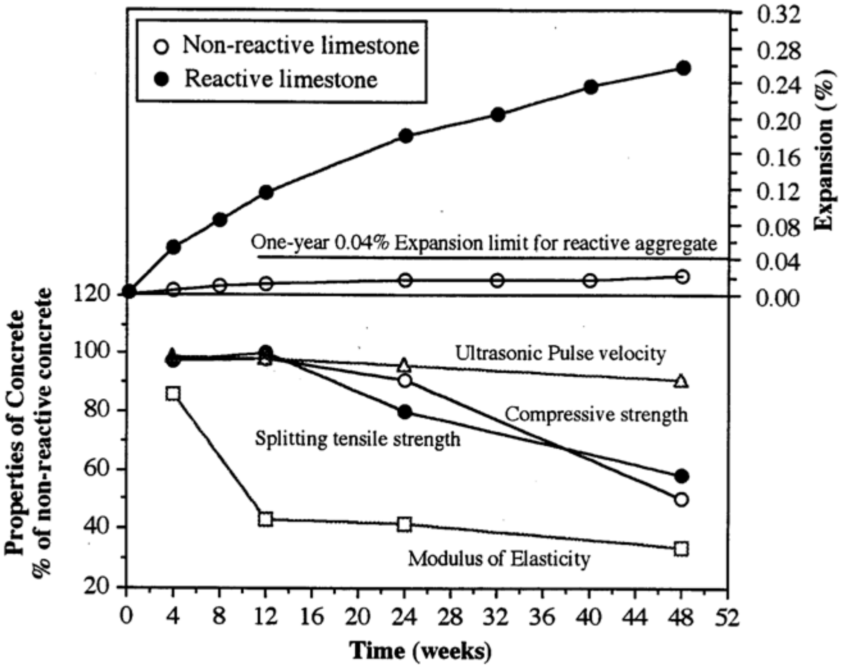


Figure 7.15. Residual mechanical properties for concrete affected by ASR, as percentage of values obtained at the same age from unaffected concrete; the expansions of the companion test prisms are also given (Pleau et al. 1989)

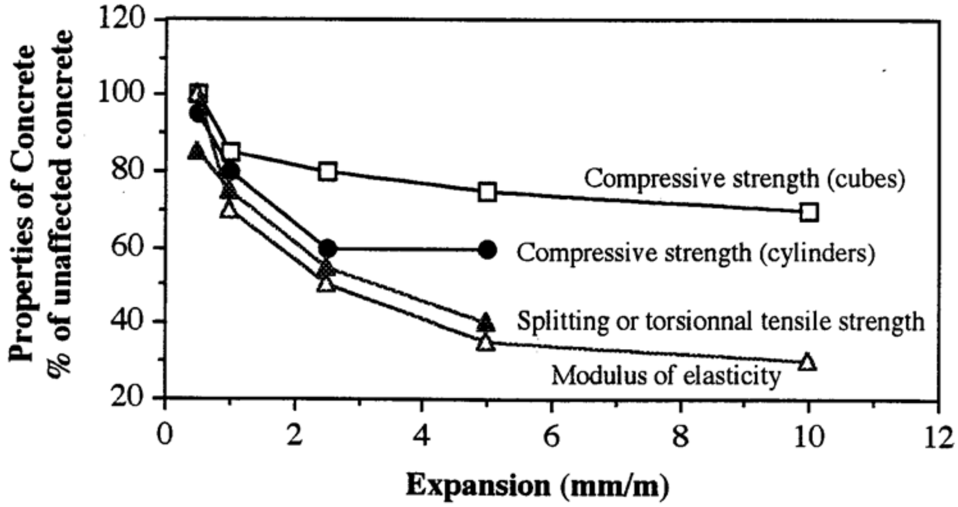


Figure 7.16. Lower Bound of Residual Mechanical Properties for ASR-Affected Concrete Specimens, Expressed as a Percentage of Values Obtained for Unaffected Concrete at 28 Days (ISE 1992)

A number of tests have shown that losses in elastic modulus and flexural strength could lead to substantial reductions in flexural rigidity and structural stiffness of affected members. Hobbs (1986) and Smaoui et al. (2006) noted that the modulus of elasticity is more affected than the compressive and indirect tensile strengths, both of which having been quite similarly affected by ASR. The rate of reduction in modulus of elasticity can vary according to the type of reactive aggregate (Smaoui et al. 2006); important reduction in the modulus of elasticity can occur even at a low level of AAR expansion – actually even when compressive strength of the affected concrete is still increasing (Pleau et al. 1989) (Figure 7.15). According to Clark (1990), there would be no significant reduction in modulus of elasticity for expansions less than 0.05%. Losses in modulus of elasticity and flexural strength between 20 and 60% were reported for expansions ranging from 0.1 to 0.3% (ISE 1992), and could even reach 80% for the former at very high expansion levels. For the two reactive aggregates tested in their study, Smaoui et al. (2006) found relatively similar reductions in the modulus of elasticity measured in compression and in traction. Swamy and Al-Asali (1986, 1988) observed that the dynamic modulus of elasticity is affected at an early age, even before reaching the 0.1% expansion level.

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 7.17A), 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 7.17B). 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 7.17C).

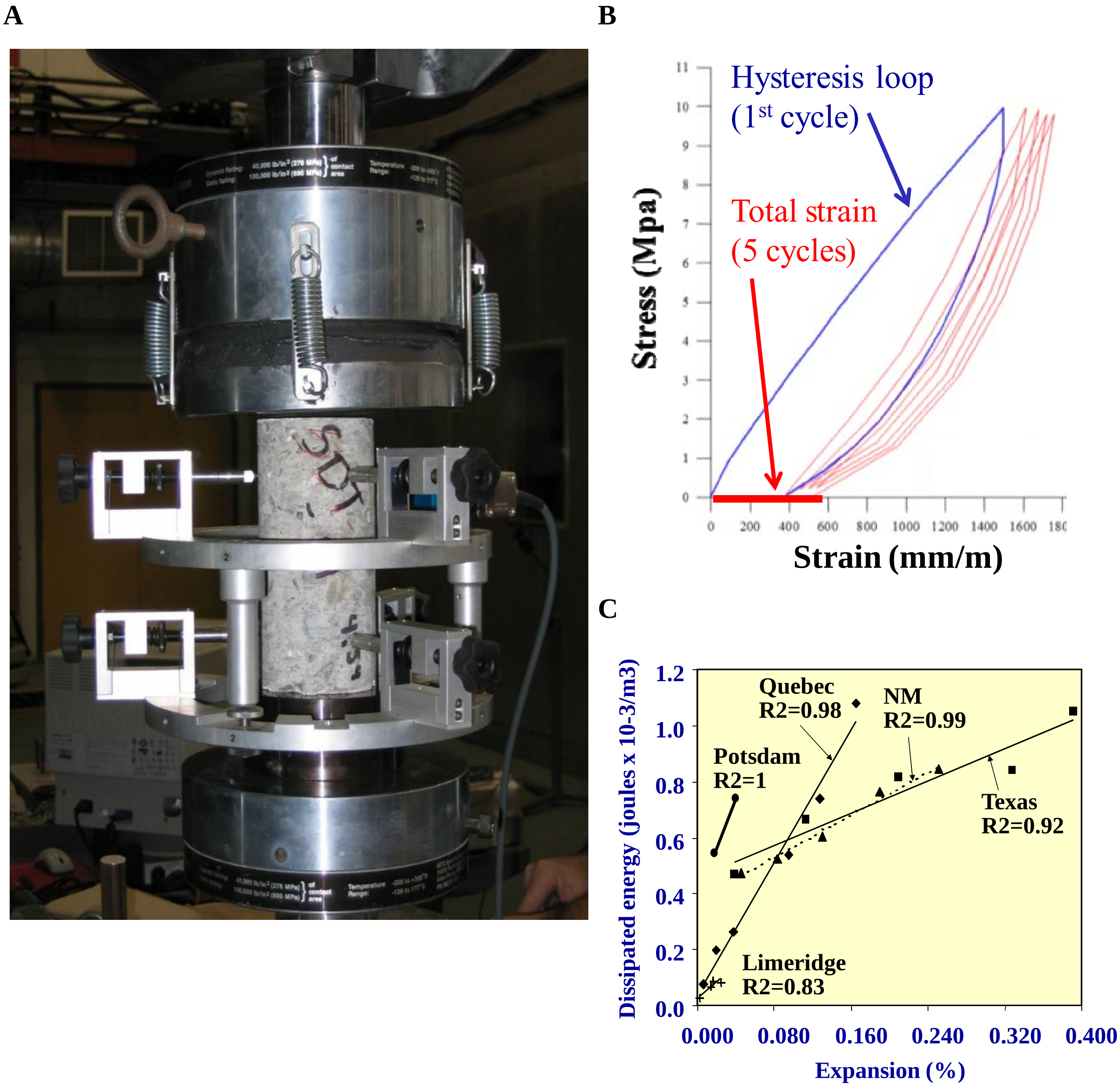
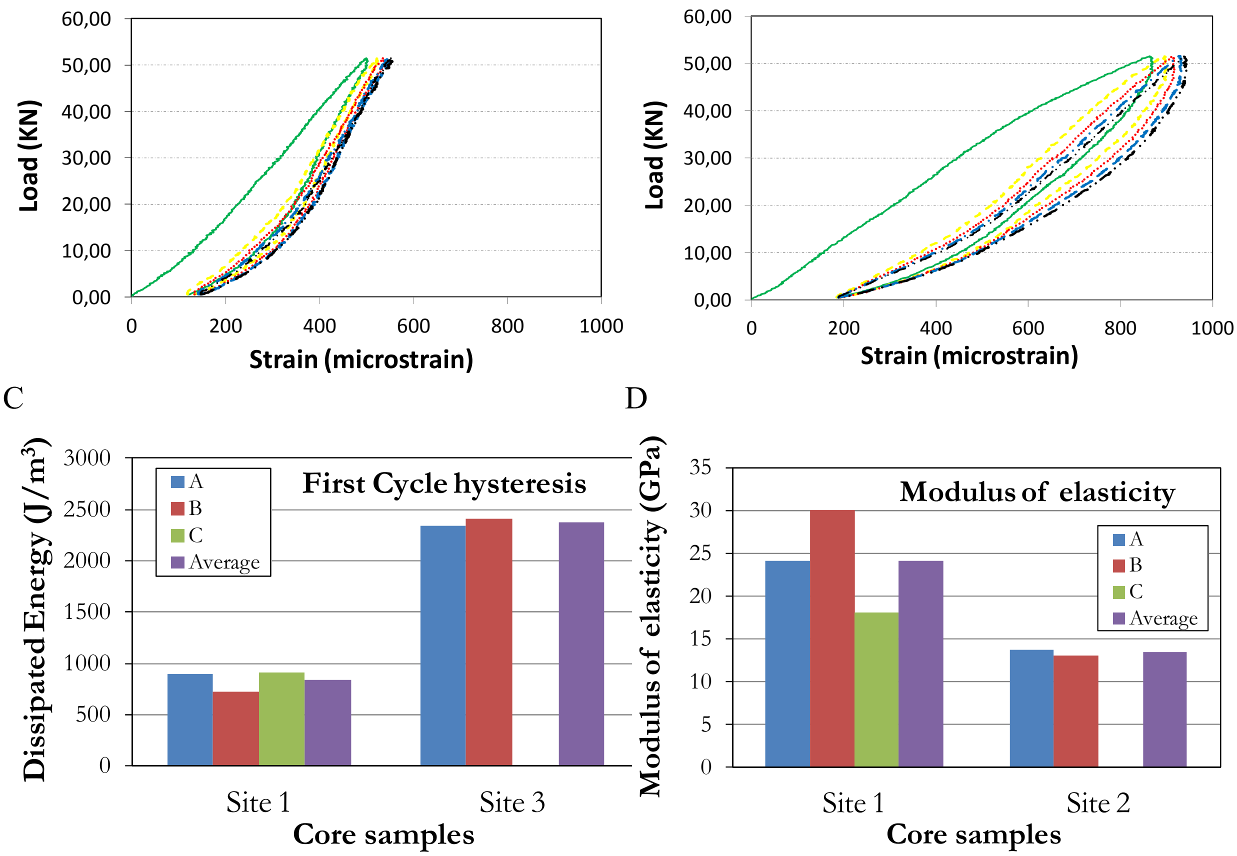


Figure 7.17. Mechanical testing of concrete cores using the Stiffness Damage Test (SDT). A. Cores subjected to 5 loading/unloading cycles up to 10 MPa. B. The best output parameters correspond to the energy dissipated during the first cycle (hysteresis loop) and the accumulated plastic strain after the 5 cycles. C. Calibration curves for different reactive aggregates (Smaoui et al. 2004).

Similar to the DRI, the SDT can provide a quantitative assessment of the extent of internal damage in structural concrete members, exposed or not to moisture, either punctually (i.e., at a specific time - diagnosis) (e.g., Figure 7.18) or as a progressive process when performing the measurements on a regular basis on cores extracted at the same location (prognosis).


A – Core from Site 1

B – Core from Site 3

C

D

Figure 7.18. Results of Stiffness Damage Testing on cores extracted from the highway bridge structure illustrated in Figure 7.14. A. SDT result for a core extracted from site 1 (abutment not exposed). B. SDT result for a core extracted from site 3 (wing wall exposed). C. Comparison of SDT results, for instance the first cycle hysteresis, for cores extracted from sites 1 (under bridge deck - less deteriorated) and 3 (exposed to moisture – more deteriorated). Higher dissipated energy values are obtained for ASR-affected concrete cores (site 3). D. Comparison of SDT results, for instance the average values of the modulus of elasticity for the 2nd and 3rd cycles, for cores extracted from sites 1 (under bridge deck - less deteriorated) and 3 (exposed to moisture – more deteriorated). Lower modulus of elasticity values are obtained from ASR-affected concrete cores (site 3).

Expansion Tests on Cores

The most commonly used method involves the testing of concrete cores maintained at 38°C and 95% R.H. They are expected to 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. 2002b; CSA 2000) (Figure 7.19). After an initial "conditioning" period during which the concrete core will reach a volumetric equilibrium with respect to its new condition (i.e., unrestrained, high temperature and humidity, additional expansion of preexisting ASR gel due to unrestrained and very humid test conditions), expansion is measured and tentatively related to a potential for further expansion due to ASR. However, difficulties are often encountered in the interpretation of the test results (Bérubé et al. 2002b), mainly because of: (1) unknown true correlation between "free expansion" of cores and the actual expansion in reinforced concrete members; (2) uncertain correlation of ASR expansion with respect to temperature, which is normally lower in the field; (3) extreme humidity in the expansion tests; and (4) the possibility that the tested concrete may be abnormally cracked and porous with respect to the overall field concrete member under study. A further complication arises from the leaching of alkalis from relatively small specimens stored at 100% relative humidity; this can lead to an underestimation of the residual potential for ASR. Immersion tests on cores in 1N NaOH solution at 38°C are generally considered to indicate the absolute degree of reactivity of the aggregates present in the concrete under study (Bérubé et al. 2002b).

A

B

C

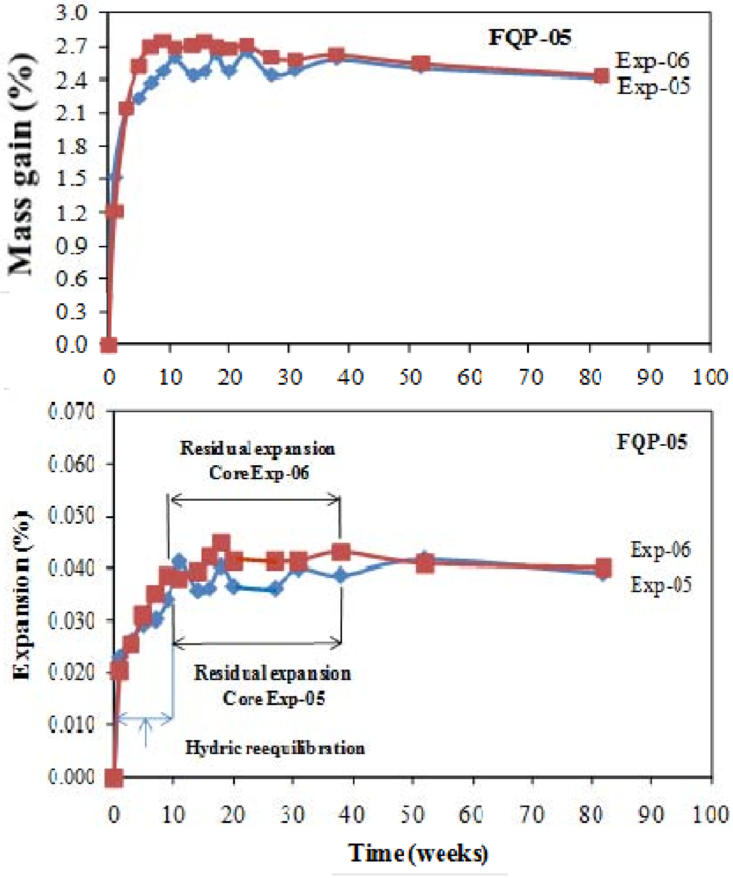


Figure 7.19. Residual expansion testing carried out on core samples (38°C, R.H. > 95%). A. Expansion measurement of the core. B. Mass measurements are also carried out in parallel to the expansion measurements to determine the moisture uptake capacity of the core. C. Example of comparative mass and residual 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.

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). Over time, significant alkalis can be released from aggregates and contribute to an increase in the concrete alkali content (Bérubé et al. 2002c). Since the alkali content can be subject to considerable variation within a single concrete element, or between different concrete components of the same structure, separate determinations should be made on a number of samples taken from different components at different depths, and representing concrete showing varying severity of deterioration and subjected to different exposure conditions (rain, sun, buried or underwater portions of the structures, etc.).

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 laboratory and 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 <u>quantify</u> the current condition of the concrete, and to evaluate the potential for future deterioration (<u>prognosis</u>). 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 <u>management actions</u> to be taken for each of the particular applications.

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8 – Mitigation Methods for ASR-Affected Structures

8.1 INTRODUCTION

This chapter 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 chapter 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.

- **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 8.3. Section 8.4 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 8.5 and 8.6, respectively.

The Federal Highway Administration (FHWA) established the ASR Development and Deployment Program (under FHWA Contract DTFH61-06-D-00035) in 2006 to provide technical assistance to States and practitioners to address ASR issues. As part of these efforts, field trials were initiated that included the implementation of various mitigation measures in highway infrastructure, including bridges, barrier walls, and pavements. Figure 8.1 provides an overview of the FHWA-funded field trials, including information on the various mitigation measures (Ahlstrom 2012). Discussion on several of these field trials is included in this chapter, but it should be kept in mind that the key findings from these studies may not be known for several years, and certainly beyond the conclusion of the FHWA project in 2013.

Table 8.1. FHWA-Funded Field Trials (adapted from Ahlstrom 2012)

State	**Structure**	**Mitigation Technique**
Alabama	Historic bridge arch	Silane sealer
Arkansas	Pavement	Silane sealer
Delaware	Pavement	Topical application of lithium nitrate
Delaware	Pavement	Monitoring an asphalt overlay of pavement with lithium nitrate
Hawaii	Aggregates	Testing aggregates and development of field exposure site
Massachusetts	Median barrier	Silane sealers; topical application of lithium nitrate
Massachusetts	Aggregates	Testing aggregates and development of field exposure site
Maine	Bridge abutments and piers	FRP wrap; silane sealer; electrochemical application of lithium nitrate
Texas	Bridge Columns	Electrochemical application of lithium nitrate; vacuum impregnation of lithium; silane sealers
Texas	Precast Bridge Girders	Aggregate testing and investigation of specific mixture designs
Vermont	Bridge barrier walls	Silane sealers

8.3 REDUCING INTERNAL RELATIVE HUMIDITY

The availability of moisture is critical for ASR to cause significant expansion and cracking. In the laboratory, 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. The data in Figure 8.2 are for concrete prisms composed of various reactive aggregates and stored at different relative humidities. Although the effects of moisture on expansion did vary from aggregate to aggregate, reducing the relative humidity to 80 percent greatly reduced the observed expansions after two years of testing for all aggregates in the study.

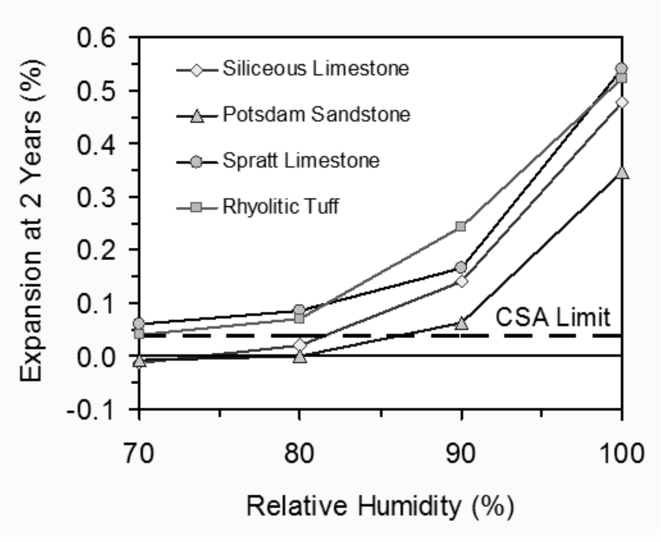


Figure 8.2. Influence of Relative Humidity on Expansion due to ASR (after Pedneault 1998)

The effects of moisture availability on ASR are also quite evident in the field. It is quite common for part of a structure that is directly exposed to water (e.g., rain water) to exhibit signs of ASR-induced expansion and cracking while part of the same structure that is protected from direct contact with water will show very little symptoms of ASR. Figure 8.3 illustrates the importance of moisture availability on ASR-induced cracking – the bent cap, from a bridge in Houston, Texas, only exhibits visual cracking on the exterior faces that are subject to direct rainfall. The portions of the bent cap well below the bridge deck, and thus protected from direct rainfall, show only negligible signs of distress. This is typically the case with precast bridge girders – only the exterior girders tend to show signs of ASR-induced expansion and cracking, whereas interior girders that are shielded from rain often show negligible symptoms of ASR.

Figure 8.3. Photograph of bridge in Houston, Texas, illustrating effects of moisture availability on ASR-induced expansion and cracking. Only the exposed face of the bent cap (left side of photo) shows visual cracking, whereas the portions of the bent cap protected by the bridge deck show very little cracking.

Figure 8.4 shows a photograph of a bridge structure included as part of the FHWA ASR Development and Deployment Program. Prior to applying various sealers/coatings to the bridge, cores were extracted from Location A (under the bridge deck, sheltered from rain) and Location B (exposed directly to precipitation). Petrographic evaluations of the cores showed that the concrete mixture proportions were identical, meaning the same concrete was used in each location of the bridge. Using the Damage Rating Index (DRI), a quantitative petrographic technique described in chapter 7, it was found that the DRI of a core extracted from Location A was 528, whereas the DRI of a core extracted from Location B was only 133 (note – the higher the DRI, the greater the extent of ASR). Thus, the effects of exposure conditions on ASR are evident both to the naked eye, as shown in Figure 8.3, and under a petrographer's microscope, as evidenced by the elevated DRI values for cores extracted from portions of the bridge directly exposed to precipitation in Figure 8.4.

A great deal can be learned from the examples shown in Figures 8.3 and 8.4, where there is a clear link between moisture availability and ASR-induced expansion and cracking. Examples of this type are ubiquitous in highway infrastructure, where the influence of moisture availability on ASR is evident not only in bridges, but also in pavements, where joints tend to deteriorate first due to the increased availability of water at or near joints. These observations can be synthesized and applied to mitigation measures for ASR-affected concrete – whenever possible, one should try to limit the availability of moisture. This can involve improving drainage for a given structure, for example, by diverting drainage from a bridge deck away from an ASR-affected column or cap. Likewise, improving the drainage conditions for an ASR-affected pavement, when possible, can help to reduce future expansion and cracking.

Figure 8.4. Photograph of bridge in Bangor, Maine. Location A is sheltered by the bridge deck, whereas Location B is exposed directly to precipitation. Sampling of concrete cores from each location showed strong correlation between moisture availability and ASR-induced expansion and cracking.

It is not always a feasible option to simply improve drainage for a given ASR-affected structure, and other, more advanced or aggressive measures may be warranted. For example, 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. The use of ventilated cladding slowed down the progression of ASR in a race course grandstand by minimizing access to external moisture while still allowing the concrete to dry out (Hobbs 1989).

For instances where it is not possible to adequately dry concrete out through improved drainage or where it is not feasible (technically or economically) to apply ventilated cladding, the use of coatings or sealers to reduce the internal relative humidity should be considered. A coating or penetrating sealer that will trigger this reduction in internal relative humidity must provide the following characteristics (after CSA 2000):

1. be resistant to water absorption;
2. penetrate to a measurable depth;
3. resist deterioration from ultraviolet (UV) radiation;
4. possess long-term stability in an alkaline environment;
5. be of long-term stability in an alkaline environment; and
6. allow vapor transmission

There are a range of products, some proprietary in nature, that satisfy the above characteristics, but silicone-based products have been used most commonly 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). Silicones exhibit several characteristics that make them well-suited for such applications, including (after Mayer 1998):

- strong repellence to liquid water,
- chemical inertness in most cases,
- stability in a wide range of temperatures, and
- excellent insulative properties.

Members of the silicone family that have been used the most for such applications are silanes, siloxanes, and silicone resins, with silanes being the most commonly used, as well as the most commonly evaluated with regard to ASR. Because of the relevance of silanes to ASR research and implementation, discussion follows on the mechanisms by which silanes are able to reduce the internal relative humidity.

When silanes are applied to the concrete surface, a chemical reaction known as crosslinking or polycondensation occurs within the concrete substrate or at the surface to form a silicone resin network, which is responsible for water repellency, permeability to water vapor, and durability (Mayer 1998). Silanes achieve their unique properties by chemically bonding a water-repellent hydrocarbon molecule to the substrate, as illustrated in Figure 8.5. As a result, the critical surface tension of the concrete substrate is reduced, and if the surface tension of the substrate falls below that for water, it will be water repellent (McGettigan 1992).

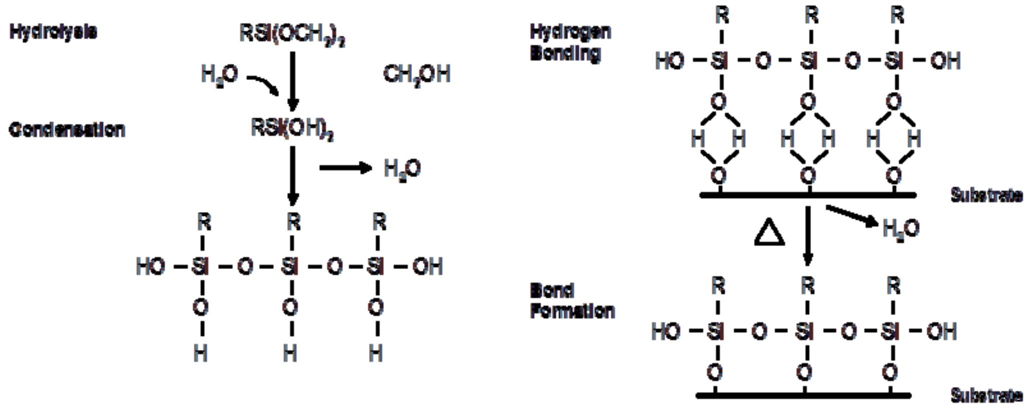


Figure 8.5. Mechanism by which Silanes React with Concrete (adapted from Dow Corning 2005)

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).

Silanes are almost always applied topically (see Figure 8.6), at a coverage rate typically in the range of 2.4 – 4.8 m^2/L (or 100 – 200 ft^2/gal). The depth of penetration will vary from product to product and based on environmental conditions. Lute (2008) reported depths of penetration of silanes into concrete mixtures typically used in highway applications to be in the range of 2 to 6 mm. This depth of penetration is sufficient to form a functional barrier, preventing water from entering but allowing moisture vapor to escape.

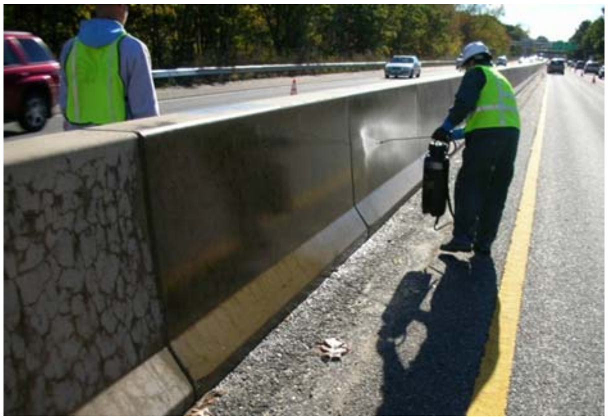


Figure 8.6. Photograph Showing Topical Application of 40%-Silane Solution (solvent-based) to ASR-Affected Highway Barrier in Massachusetts (part of FHWA Project DTFH61-02-C-00097)

There have been several studies that have confirmed the benefits of applying 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 8.7), as well as future expansion (Figure 8.8).

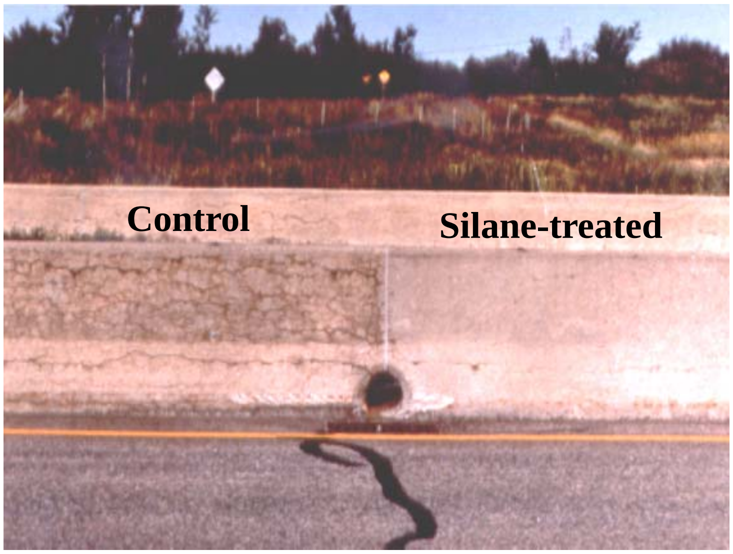


Figure 8.7. Reduction in Cracking of Highway Barriers in Canada, after Application of Silane (Bérubé et al. 2002)

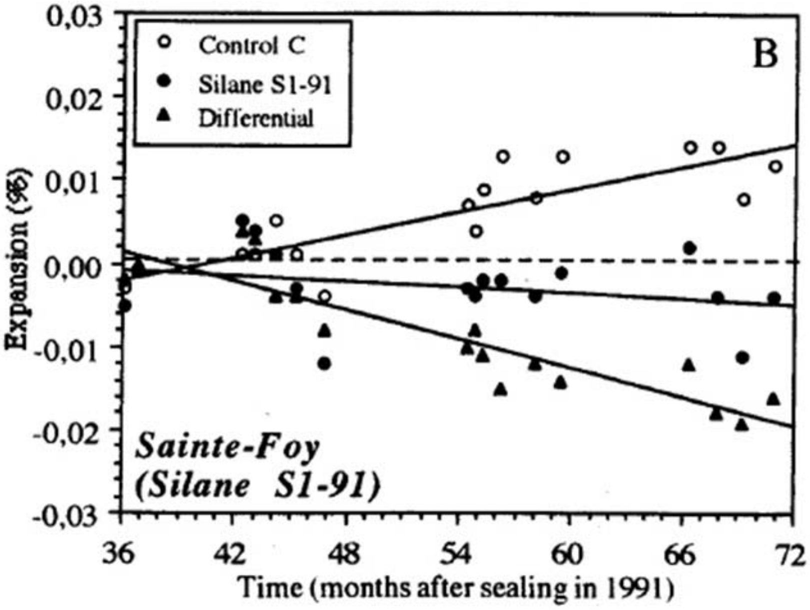


Figure 8.8. Reduction in Expansion of Highway Barriers in Canada, after Application of Silane (Bérubé et al. 2002)

Badly cracked concrete piers supporting the Hanshin Expressway in Japan were repaired at an age of 7 years by first filling the cracks with an epoxy resin injected under pressure and then either coating with an epoxy resin or impregnating with silane followed by a cosmetic coating of a polymer cement paste (Kojima et al. 1992), as shown in Figure 8.9. Of all the treatments shown in Figure 8.9, the columns treated with silane performed the best, and those columns treated with non-breathable coatings, such as epoxy and acrylic, performed the worst as moisture vapor was not allowed to escape the concrete.

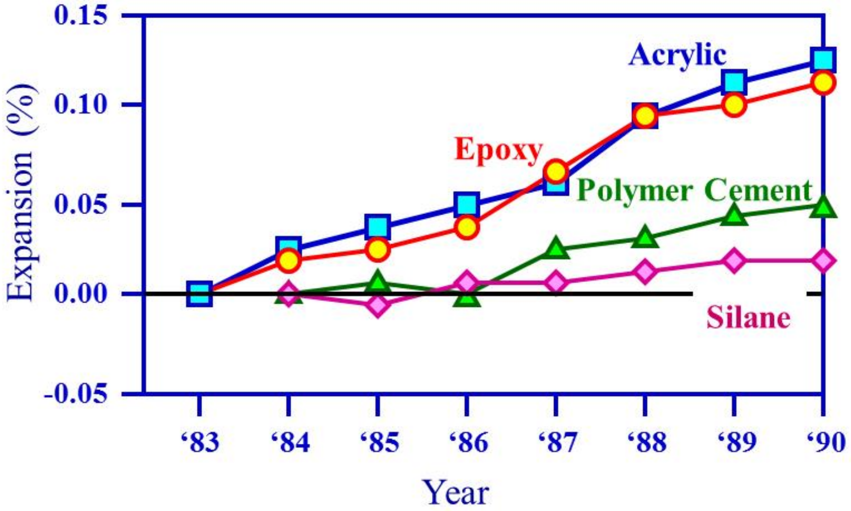


Figure 8.9. Reduction in Expansion of Bridge Columns (Hanshin Expressway, Kobe, Japan) Triggered by Use of Silane Coating (Kojima et al. 1992)

Several FHWA-funded field trials over the past few years (under FHWA Project DTFH61-06-D-00035) have included the topical application of silane-based products, as detailed in Table 8.1. Silanes were topically applied to highway barriers, pavements, and bridges. Most of the ASR-affected elements treated by silane-based products were treated quite recently (2010-2012), and as such, it is too premature to make conclusions regarding the efficacy of the various coatings and sealers in reducing the relative humidity inside concrete and reducing future ASR-induced expansion.

Fortunately, highway barriers treated under a previous FHWA project (DTFH61-02-C-00097) focusing on implementation of lithium technologies have been included in the latest FHWA project (DTFH61-06-D-00035), and as such, the barriers were included in the ongoing monitoring efforts and have been monitored for a total of about five years. Figure 8.10 shows the significant reduction in cracking of highway barriers in Massachusetts, approximately three years after the topical application of various silane-based products. Figure 8.11 shows the expansion data for the barriers, confirming that all of the silane products studied reduced expansion, compared to the control. In fact, the barriers that were most affected by silane applications underwent shrinkage, which is consistent with the work done in Canada on highway barriers (see Figures 8.7 and 8.8).

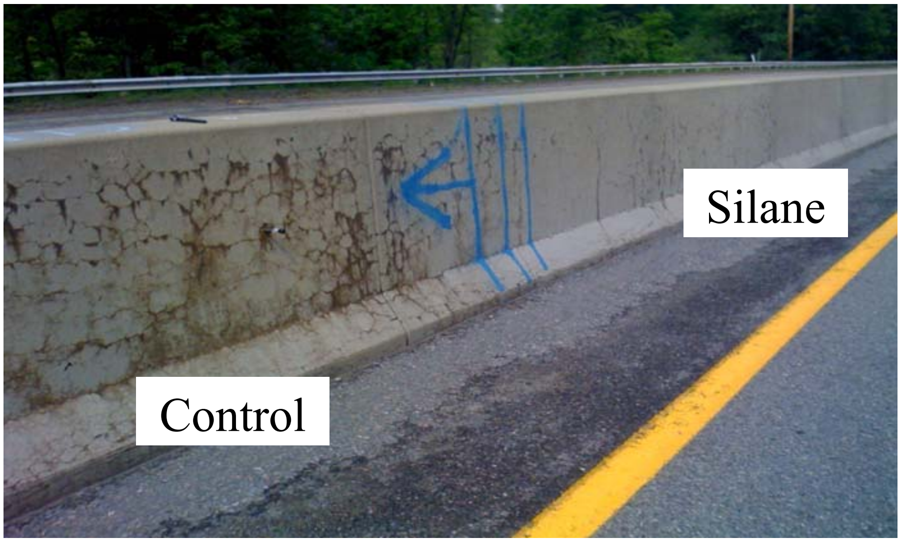


Figure 8.10. Reduction in Cracking of Highway Barriers in Massachusetts, after Application of Silane (FHWA Project DTFH61-06-D-00035)

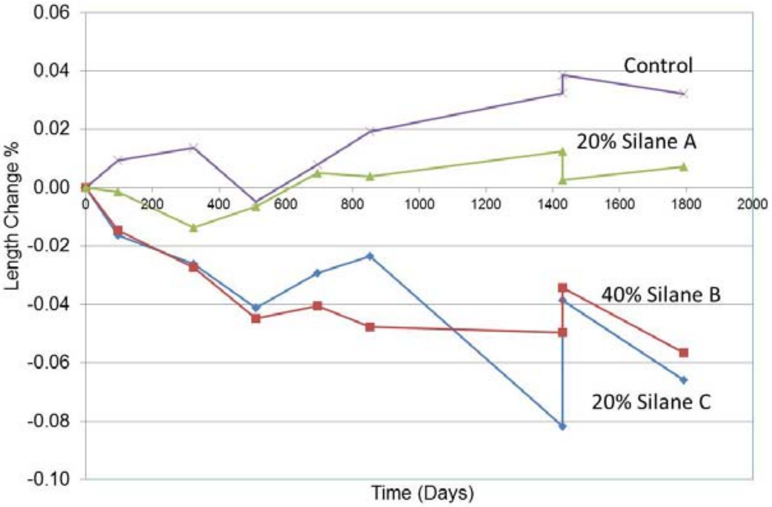


Figure 8.11. Effects of Silane Treatment on the Expansion of Highway Barriers in Leominster, MA (note Silanes A and B are water-based. Silane C is solvent-based)

Based on the experience with highway barriers, silanes can be quite effective in reducing internal relative humidity, expansion, and cracking. However, this is somewhat of an ideal situation for applying silanes – the barriers are relatively thin and are treated on both sides, thereby allowing for the relative humidity to be reduced from both sides of the wall. There are certainly other applications where silanes or products that work in a similar manner may not be effective. For pavements, slabs on grade, wingwalls, or other applications where moisture is available from below (or beneath), silanes will likely not be as effective (or effective at all) 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. Figure 8.12 shows that immersing concrete prisms in lithium nitrate reduced the future expansion of these prisms when subjected to storage conditions similar to ASTM C 1293 (Stokes et al. 2000). Although one cycle of lithium application slightly reduced expansion, multiple cyclical applications of lithium nitrate significantly reduced long-term expansion. It should also be noted that prisms that were treated by multiple cycles of immersion in water, as opposed to lithium nitrate, exhibited reduced expansion as well, presumably due to the leaching of alkalis from the prisms to the water during the immersion period.

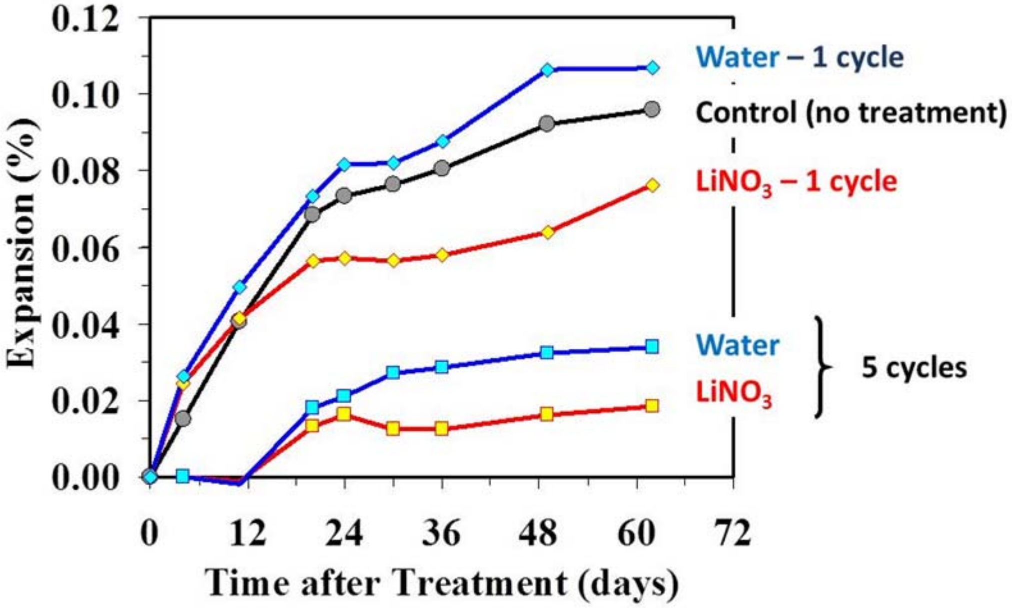


Figure 8.12. Effects of Submerging Concrete Prisms in Lithium Nitrate (or water) on Expansion of Prisms in ASTM C 1293 Environment (after Stokes et al. 2000)

Although the mechanisms by which immersing concrete or mortar specimens in lithium compounds may reduce future expansion are not fully understood, it is generally believed that it changes the nature and behavior of the gel from expansive to essentially non-expansive. Because of the positive results in laboratory-based work and driven by the need for viable mitigation measures for ASR-affected structures and pavements, 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 8.13), highway barriers (Figure 8.14A), and bridge decks. There have also been a handful of field trials where lithium was applied either by vacuum (Figure 8.14B) or through electrochemical means, both aimed at increasing the depth of penetration of lithium.

Figure 8.13. Photograph Showing Topical Application of 30%-$LiNO_3$ Solution to Concrete Pavement in Idaho

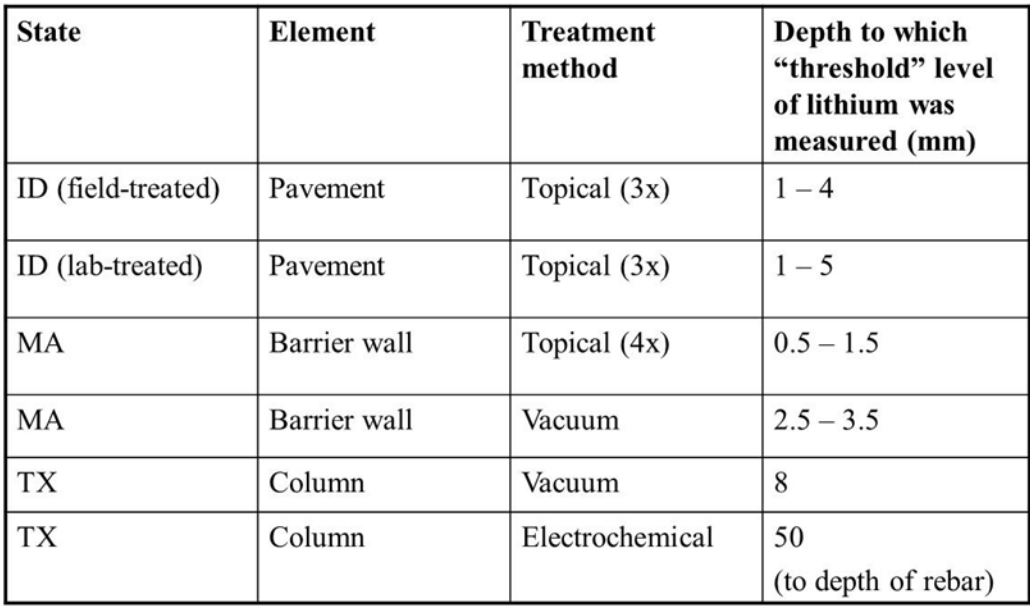
A B

Figure 8.14. Photographs 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). Recognizing the need to obtain such critical data, field trials were initiated under FHWA funding (under FHWA Project DTFH61-02-C-00097) in Idaho, Massachusetts, and Texas. The primary focus of these studies was primarily on the topical application of lithium nitrate, but the applications of lithium nitrate by vaccum and by electrochemical methods were also evaluated.

Table 8.2 summarizes the lithium penetration data for field trials in three states (after Folliard 2008). When evaluating lithium profile data, it is important to view it in context of how much lithium is needed to control expansion. Consider the following example: Assuming a plain concrete mixture contains 3 kg/m^3 Na_2O_e, a "standard dose" of 30% lithium nitrate solution (4.6 L of solution per 1 kg of Na_2O_e) would amount to 13.8 L/m^3 of lithium nitrate solution. This dosage of $LiNO_3$ solution contains 504 g of Li, and assuming that half of this lithium gets bound in early hydration products (as well documented in literature), approximately 250 g of lithium per m^3 will remain in the concrete to combat ASR. If one assumes a concrete density of 2350 kg/m^3 the concentration of lithium ions in pore solution would be approximately 100 ppm, and this concentration would be enough to control expansion when used as an admixture. However, it should be noted that the "standard dose" of 30% lithium nitrate solution (4.6 L of solution per 1 kg of Na_2O_e) has been reported to be sufficient for controlling expansion in about half of the aggregates recently tested in North America (Tremblay et al. 2008). Some aggregates, especially greywackes, still expand considerably at over twice the "standard dose." For the purpose of this discussion, it is assumed that the aggregate in the concrete being treated responds favorably to lithium (4.6 L of solution per 1 kg of Na_2O_e) and that 100 ppm would be required in pore solution for suppression of excessive expansion.

Table 8.2. Summary of Lithium Penetration Data for Highway Applications of Lithium Nitrate



State	Element	Treatment method	Depth to which "threshold" level of lithium was measured (mm)
ID (field-treated)	Pavement	Topical (3x)	1 – 4
ID (lab-treated)	Pavement	Topical (3x)	1 – 5
MA	Barrier wall	Topical (4x)	0.5 – 1.5
MA	Barrier wall	Vacuum	2.5 – 3.5
TX	Column	Vacuum	8
TX	Column	Electrochemical	50 (to depth of rebar)

Following the above discussion and assuming a requisite lithium threshold of 100 ppm, the data in Table 8.2 was not very positive with regard to topical or vacuum applications of lithium nitrate. 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 penetration 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.

Figure 8.15 shows that none of the topical lithium applications reduced expansion in the highway barriers treated in Leominster, MA, unless the lithium treatment was followed up with a subsequent application of 40% silane. Given that multiple treatments of lithium nitrate did not reduce expansions, it is logical to conclude that the reduction in expansion can be attributed solely to the benefits of silane in reducing the internal relative humidity of the concrete. Similar results were obtained for the barriers in which a vacuum was applied to try to drive lithium nitrate in further. Reductions in expansion were only observed when the vacuum treatment was followed up with an application of 40% silane.

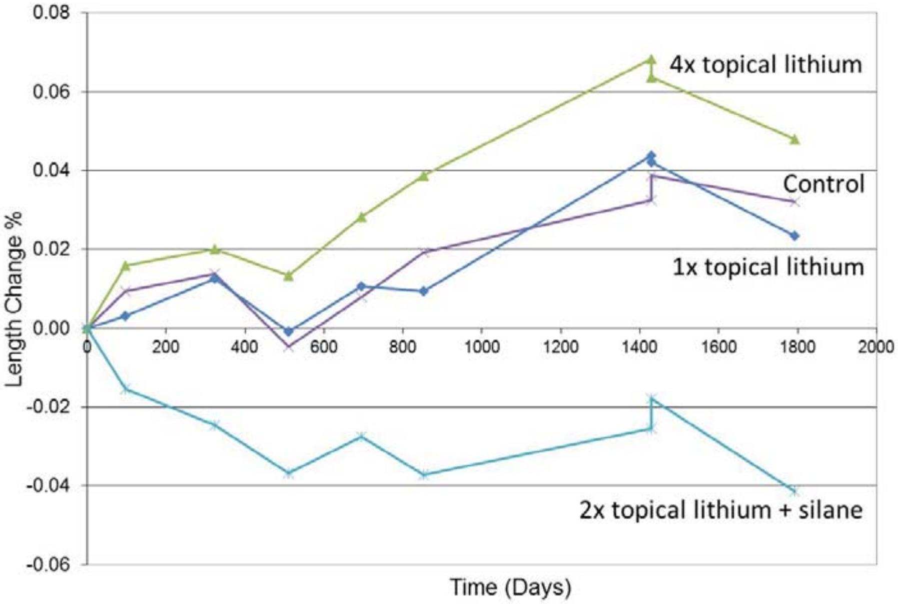


Figure 8.15. Effects of Topical Lithium Nitrate Application on Expansion. Note that only the barriers treated with silane reduced long-term expansion.

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 turns 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. 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 will 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 8.16) and more recently in Vermont as part of ongoing FHWA field trials (under FHWA Project DTFH61-06-D-00035).

Figure 8.16. 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 (under FHWA Project DTFH61-06-D-00035) 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.

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9 – Alkali-Carbonate Reaction

9.1 INTRODUCTION

As discussed in the introduction, alkali-carbonate reaction (ACR) is a reaction between alkali hydroxides (NaOH and KOH) in the pore solution 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.

Cases of ACR-induced damage in concrete are much less widespread than cases of ASR and, consequently, the carbonate reaction has attracted significantly less attention by researchers. The precise mechanisms of expansion are equivocal, and it has been conjectured that ACR may be a form of ASR involving reactive silica (e.g., cryptocrystalline quartz) present in the dolomitic limestone; however, this theory has not been universally accepted.

There are some distinct features of ACR that differentiate it from ASR including, obviously, the nature and composition of the rock, the timeframe of the reaction, the influence of aggregate size, the inability of some standard tests to identify reactive aggregates, the threshold alkali content required to generate expansion, and the impact of preventive measures such as pozzolans, slag, and lithium-based compounds. Generally measures that have been found to be effective in preventing expansion due to ASR do **not** work with alkali-carbonate rocks and it is generally recommended that such rocks are avoided for use in concrete.

This chapter presents an overview of alkali-carbonate reaction (ACR) including a discussion of the chemistry of the reaction, the mechanisms of expansion, contributing factors, and differences between ACR and ASR. Field symptoms in ACR-affected concrete structures are similar to those observed in structures affected by ASR, and these are discussed in chapter 3. Chapter 4 discusses test methods for both types of alkali-aggregate reaction. Recently developed AASHTO PP65, for identifying reactive aggregates, includes procedures for detecting and, subsequently, avoiding alkali-carbonate reactive rocks. This practice is presented in chapter 6.

9.2 CHARACTERISTICS OF ALKALI-CARBONATE REACTIVE ROCKS

Alkali-carbonate reactive rocks can generally be described as argillaceous dolomitic limestones and tend to have a characteristic texture and structure comprising dolomite [$CaMg(CO_3)_2$] rhombohedra (typically 10 to 50 μm) dispersed or "floating" in a fine-grained matrix of fine calcite, clay, and, usually, silica. Figure 9.1 shows the microstructure of typical reactive rock. Typically the carbonate-mineral composition of the rock consists of substantial amounts of calcite ($CaCO_3$) and dolomite, and the (HCl-acid) insoluble residue comprises of a significant amount of clay (ACI 221 2000; Ozol 2006). A modification of this typical texture has been observed in "late expanding" alkali-carbonate rocks which may have interlocking dolomite rhombs in a coarser-grained matrix.

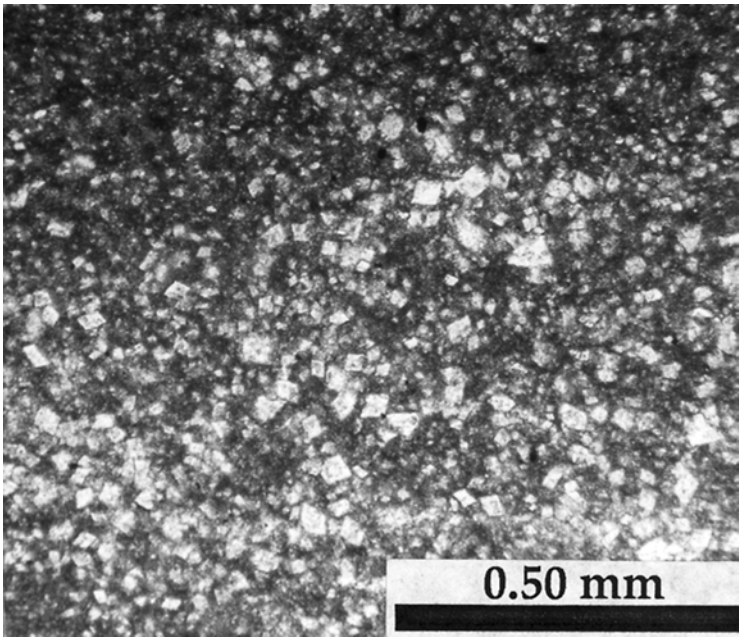

Figure 9.1. Photograph of Petrographic Thin Section Showing Dolomite Rhombs (clear equilateral parallelograms) "Floating" in a Matrix of Fine-Grained Calcite, Clay and Silica (dark background)

The bulk composition of some early and late expanders is shown in Table 9.1. Rogers (1986) compared the composition of expansive and non-expansive limestones in Ontario and observed that the expansive rocks fell within a fairly distinct region when plotted on a graph of CaO:MgO ratio versus either the alumina (Al_2O_3) or (HCl-acid) insoluble residue (see Figure 9.2). This observation forms the basis of the chemical screening test (CSA A23.2-26A) for potential alkali-carbonate reactivity which has been adopted by the Canadian Specification (CSA A23.2-27A) and the AASHTO PP65 Standard Practice (see chapter 6).

Table 9.1. Composition of Alkali-Carbonate Reactive Rocks
(adapted from Walker 1978; ACI 221 2000; Ozol 2006)

	Acid-insoluble residue (%)	Dolomite (% total carbonate)
Kingston, Ontario, Early Expanders (Swenson, 1957; Swenson and Gillott, 1960; Dolar-Mantuani, 1964)	5 to 15	Approx. 50
Iowa, Illinois & Indiana Early Expanders (Hadley, 1961, 1964)	10 to 20	40 to 60
Virginia Early Expanders (Newlon and Sherwood, 1962; Sherwood and Newlon, 1964)	13 to 29	46 to 73
Gull River, Ontario, Late Expanders (Dolar-Mantuani, 1964)	21 to 49	75 to 87
Virginia Late Expanders (Newlon, Ozol, and Sherwood, 1972)	33	> 90

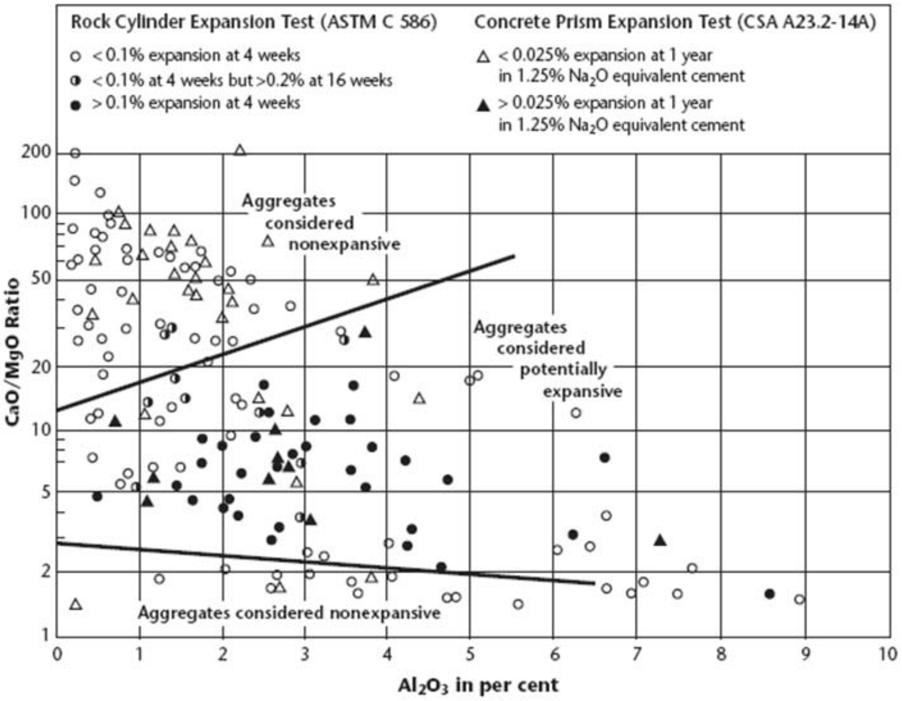

Figure 9.2. Using Chemical Composition as a Basis for Determining Potential Alkali-Carbonate Reactivity of Quarried Carbonates (from CSA A23.2-26A)

Observation of the characteristic texture and composition is considered to be a reliable indication of potential alkali-carbonate reactivity (Ozol 2006), and rocks that display the characteristic texture or composition should be tested further to evaluate their reactivity in concrete.

9.3 CHEMISTRY OF ALKALI-CARBONATE REACTION

Dolomitic limestones with the characteristic microscopic texture described in the previous section will react in an alkaline environment and undergo a process termed dedolomitization[8], which can be written as follows:

$$CaMg(CO_3)_2 + 2ROH \rightarrow CaCO_3 + Mg(OH)_2 + R_2CO_3 \qquad \text{Eqn 9.1}$$

dolomite + alkali hydroxide → calcite + brucite + alkali carbonate

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[8] ACI 221.1R-98 states "…there appear to be no known exceptions to the observation that all limestones that possess the characteristic texture and composition will react, or dedolomitize, in an alkaline environment. Also, all limestones that dedolomitize in an alkaline environment possess the characteristic texture and composition.

where R represents K or Na. In some cases the magnesium may further react to form magnesium silicate.

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

$$R_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2ROH \qquad \text{Eqn. 9.2}$$

thereby "regenerating" alkalis for further reaction. This reaction may result in "carbonation halos" forming around reacted aggregate particles.

9.4 MECHANISMS OF EXPANSION DUE TO ACR

The dedolomitization reaction shown in Equation 9.1 is accompanied by a reduction in solid volume. In other words the molar volume of $CaCO_3$ plus $Mg(OH)_2$ is less than the molar volume $CaCO_3{\cdot}MgCO_3$ and, since Na_2CO_3 is expected to pass in to solution, no volume expansion is expected (Gillott 1964). Hence, dedolomitization does not provide a simple mechanical explanation for expansion. Several theories have been proposed to explain the expansion mechanism, and these include:

- Swelling of clay matrix (Swenson and Gillott 1964; Gillott 1964; Gillott and Swenson 1969). Dedolomitization of the reactive material exposes previously "trapped" clay minerals (illite and chlorite) to alkali solution. The previously unhydrated clay minerals are in an "active" state. Exchange sites on the surfaces of the clay, which may be vacant or occupied by Ca^{2+} or Mg^{2+} ions, absorb Na^+ (and K^+) ions from solution (cation exchange) and a double layer develops accompanied by water uptake, which leads to swelling of the clay minerals and, consequently, an increase in the solid volume of the aggregate. The expansion essentially results from the wetting (and cation exchange) of previously unhydrated clay minerals that was made possible by the dedolomitization reaction which opened access channels for the moisture.
- Growth and rearrangement of the products of dedolomitization (i.e., brucite and calcite) through a topochemical reaction (Tang et al. 1987). Clay is not embedded in dolomite rhombs but forms a network between the fine calcite grains surrounding the rhombs. The clay network provides channels for alkali hydroxides in the pore solution to "attack" the dololomite rhombs resulting in dedolomitization. After reaction, brucite is observed to form around the outside of the original rhomb in rings approximately 2 µm thick, the original structure of the rhomb being maintained by a framework of calcite which largely remains in place after the migration of the magnesium. The original rhomb with the peripheral brucite layer requires more space resulting in the generation of expansive forces. A schematic illustrating the phenomenon is presented in Figure 9.3.

- Alkali-silica reaction of cryptocrystalline quartz in matrix surrounding the dolomite rhombs (Katayama 1992; 2010). Alkali-silica gel has been observed in mortars and concretes containing alkali-carbonate reactive aggregates, and it has been postulated that reaction of cryptocrystalline quartz contained within the carbonate is responsible for the expansion of the mortar and concrete, and that the accompanying dedolomitization is "harmless".

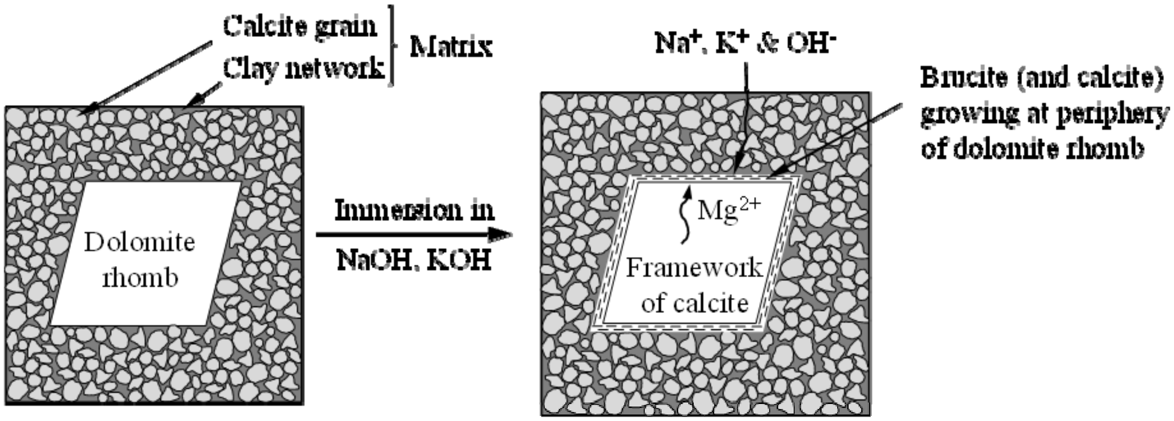

Figure 9.3. Schematic Showing Topochemical Formation of Brucite (and Calcite) Ring around Dolomite Rhomb (modified from Tang et al. 1987)

Although the clay-swelling mechanism appears to have lost support, there is still debate among researchers whether the expansion is due to the dedolomitization reaction or the alkali-silica reaction.

9.5 FACTORS CONTRIBUTING TO EXPANSION DUE TO ALKALI-CARBONATE REACTION

Regardless of the mechanism(s) causing expansion there are a number of contributing factors that increase the expansion of concrete containing reactive argillaceous dolomitic limestones. The petrological features of the rock that appear to maximize reactivity/expansion are (Ozol 2006):

- 5 to 25% clay or insoluble residue,
- Ratio of calcite:dolomite of approximately 1:1,
- Increase in dolomite content up to point at which interlocking of the dolomite rhombs becomes a restraining factor, and
- Small size of the discrete floating rhombs.

Other factors that increase expansion are (Ozol 2006):

- Increasing coarse aggregate size,
- Moisture availability,
- Increased alkali content of concrete (or pH of solution),
- Increased proportion of reactive stone in the coarse aggregate, and
- Lower concrete strength.

Clearly if the expansion truly does result from alkali-silica reaction the quantity and form of the reactive silica will be a major determining factor. Grattan-Bellew et al. (2010) have demonstrated that there is a correlation between the amount of quartz in the insoluble residue and expansion of concrete prisms produced with three different reactive limestones.

9.6 DIFFERENCES BETWEEN ACR AND ASR

As discussed above, the debate between the cause(s) of expansion in concrete containing reactive argillaceous dolomitic limestones is ongoing and, at the time of writing, there appears to be no overall consensus. However, regardless of whether ACR or ASR produces the expansion, there are some features of reactive aggregates that undergo the dedolomitization reaction that set them apart from aggregates that are unequivocally alkali-silica reaction; these are summarized as follows:

- Expansion in the laboratory or damage in the field occurs in a much shorter timeframe with ACR.
- Expansion increases with an increase in the size of the reactive aggregate particles.
- Many tests used for the detection of alkali-silica reaction are not capable of detecting ACR rocks; these include the mortar bar test (ASTM C 227), the accelerated mortar bar test (ASTM C 1260), and the quick-chemical test (ASTM C 289).
- Expansion occurs in concrete with alkali contents below the threshold level generally considered necessary for ASR.
- The use of pozzolans and slag at levels normally sufficient for controlling ASR is not effective with ACR rocks, even when combined with low-alkali cement.
- Lithium-based compounds are not effective in controlling expansion with ACR rocks. Indeed, concrete microbars with ACR aggregate and immersed in LiOH solution have been observed to expand, albeit at 150ºC (Tang et al. 2000).

- Little (or no) gel is observed after expansion has occurred. Even when alkali-silica gel has been observed it is often difficult to reconcile the high amount of expansion with the meagre content of gel.

Proponents of the ASR mechanism have provided partial explanation for some of the apparent anomalies between the behaviour of these rocks and that of established alkali-silica reactive rocks (Katayama 2010; Grattan-Bellew et al. 2010). However, it is clear that rocks that cause expansion and undergo dedolomitization need to be treated as a separate category with regards to AAR, whatever the predominant expansion mechanism (ACR or ASR).

9.7 PREVENTION OF EXPANSION DUE TO ALKALI-CARBONATE REACTION

Regardless of the true mechanism of expansion, it is extremely difficult to control expansion of alkali-carbonate reactive aggregates that have the characteristic composition and texture described in section 9.2 and undergo the dedolomitization reaction described in section 9.3. Consequently, it is strongly recommended that such aggregates are not used in the production of concrete.

The reactive material may be avoided by selective quarrying or, alternatively, the reactive phase can either be diluted until the reactive material represents less than 20% of the coarse aggregate (or 15% of the total aggregate if both fine and coarse aggregate contain reactive material), or crushed to a smaller particle size until deleterious expansion is eliminated. However, avoidance of the reactive material appears to be the most prudent solution in the authors' opinion.

It has been suggested that expansion can be reduced to an acceptable level using very low alkali cement, e.g. no greater than 0.40% Na_2Oe (Newlon et al. 1964). However, expansion has been observed with cements with alkali contents only marginally above this level such as 0.43% Na_2Oe (Deng Min and Tang Mingshu 1993). Williams and Rogers (1991) observed cracking in sidewalk slabs after just 24 months of field exposure in Kingston, Ontario, Canada, even though the alkali content of the concrete was just 1.74 kg/m^3 (2.9 lb/yd^3) Na_2Oe. A section of sidewalk, curb, and gutter in the same vicinity also exhibited cracking, and it is believed (Williams and Rogers 1991) that the alkali content of the cement was just 0.31% Na_2Oe. Figure 9.4 (from Shehata et al. 2009) shows the effect of cement alkalis on the expansion of concrete prisms produced with an argillaceous dolomitic limestone (Pittsburg aggregate) compared with a siliceous limestone (Spratt aggregate). The results provide a good illustration of the different behavior of ACR versus ASR reactive limestones.

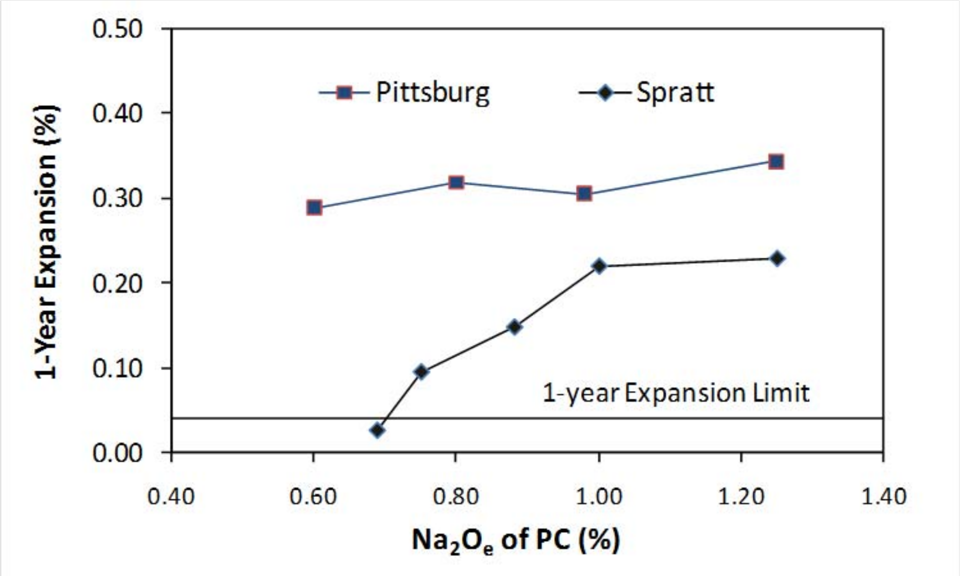

Figure 9.4. Effect of Cement Alkali Content on Expansion of Concrete Prisms with Argillaceous Dolomitic Limestone versus Siliceous (Spratt) Limestone (Shehata et al. 2009)

Swenson and Gillott (1964) reported that a variety of pozzolans found to be effective with the alkali-silica reaction when used at replacement levels of 25% were not effective in mitigating ACR. Subsequent research has reinforced this finding. Ground granulated blast-furnace slag used at replacements of 25 and 50% with high alkali cement of 1.04% Na_2Oe was also not effective in controlling expansion of laboratory-exposed concrete prisms and field-exposed concrete slabs produced with the Pittsburg Quarry dolomitic limestone from Kingston, Ontario (Rogers and Hooton 1992). Thomas and Innis (1998) tested the same aggregate with slag in concrete prisms (CSA A23.2-14A; ASTM C 1293) and found that neither 65% slag combined with high-alkali cement (1.25% Na_2Oe) nor 50% slag combined with low-alkali cement (0.50% Na_2Oe) was sufficient to control expansion; the data are shown in Figure 9.5. The data in Figure 9.5 serve again to accentuate the different behavior of ACR aggregate (Pittsburg) and ASR aggregate (Spratt). Tang et al. (1994) have shown that very high levels of fly ash (70%) or slag (90%) may be effective in controlling expansion when combined with low-alkali cement (0.43% Na_2Oe). Perry and Gillott (1985) showed that 20% silica fume was not effective in suppressing the expansion due to ACR.

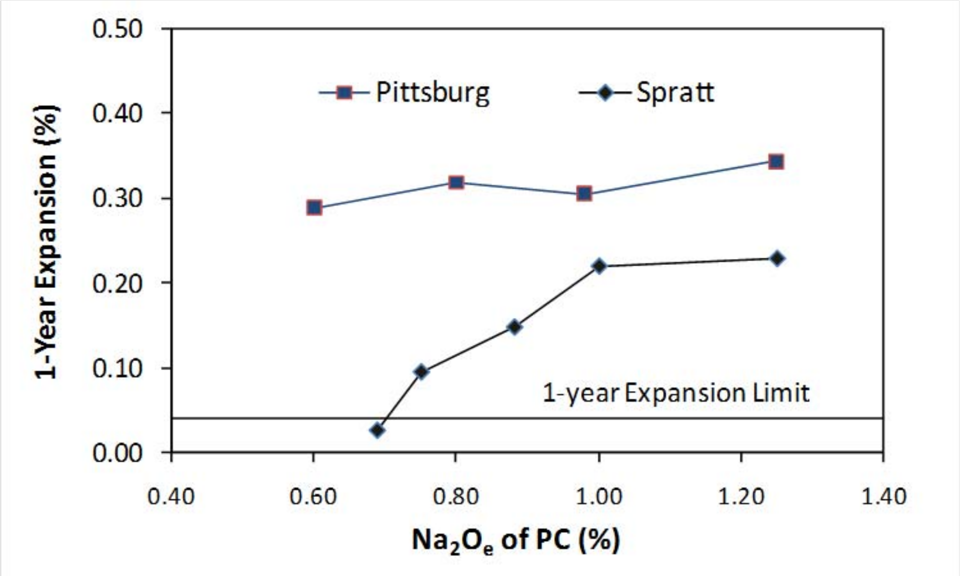

Figure 9.5. Effect of Slag Content and Cement Alkalis on Expansion of Concrete with Argillaceous Dolomitic Limestone versus Siliceous (Spratt) Limestone (Thomas and Innis 1998)

Testing has also shown that incorporating lithium-compounds (e.g., LiCl, LiOH, and Li_2CO_3) in concrete was not effective in controlling ACR expansion (Swenson and Gillott 1964; Wang et al. 1994). Such compounds have been shown to be effective with many (but not all) alkali-silica reactive aggregates, including siliceous limestones (Feng et al. 2010).

9.8 SUMMARY OF ALKALI-CARBONATE REACTION

The alkali-carbonate reaction was first documented in 1957 (Swenson 1957). Since this time, damage due to ACR has been observed in a limited number of states in the USA and a few other countries. ACR is much less widespread than ASR. Rocks that are alkali-carbonate reactive have a characteristic composition and microscopic texture, and expansion of concrete containing these rocks appears to be accompanied by dedolomitization of the rock. It is not been established unequivocally whether expansion results directly from the dedolomitization reaction or whether other mechanisms create the expansive forces. One school of thought is that reactive silica is present in these rock types and that the expansion is due to alkali-silica reaction. Regardless of whether the expansion is due to ACR or ASR, reactive rocks that have the characteristic composition and texture and that undergo dedolomitization clearly behave differently compared with reactive aggregates that are universally accepted as being alkali-silica reactive. From a practical standpoint the main differences are that these rocks cause expansion of concrete at very low alkali contents and even when relatively high replacement levels of pozzolans or slag are used. As such, it is very important that appropriate test procedures are adopted to identify these rock types and that they are excluded from use for the production of concrete.

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