

# Report on Alkali-Aggregate Reactivity

Reported by ACI Committee 221

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*Information that is currently available on alkali-aggregate reactivity (AAR), including alkali-silica reactivity (ASR) and alkali-carbonate reactivity (ACR) is summarized in the report. Chapters are included that provide an overview of the nature of ASR and ACR reactions, means to avoid the deleterious effects of each reaction, methods of testing for potential expansion of aggregates and cement-aggregate combinations, measures to prevent deleterious reactions, and recommendations for evaluation and repair of existing structures.*

**Keywords:** aggregates; alkali-aggregate reactivity; alkali-carbonate reactivity; alkali-silica reactivity; concrete; concrete distress; concrete durability.

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## **CHAPTER 1—INTRODUCTION**

In many parts of the world, precautions must be taken to avoid excessive expansion due to alkali-aggregate reactivity (AAR) in many types of concrete construction. AAR may involve siliceous aggregates (alkali-silica reactivity, ASR) or carbonate aggregates (alkali-carbonate reactivity, ACR), and failure to take precautions may result in progressive deterioration, requiring costly repair and rehabilitation of concrete structures to maintain their intended function.

Extensive knowledge is available regarding the mechanisms of the reactions, the aggregate constituents that may react deleteriously, and precautions that can be taken to avoid resulting distress. However, deficiencies still exist in our knowledge of both ASR and ACR. This is particularly true with respect to the applicability of test methods to identify the potential for reactivity, methods to repair affected concrete, and means to control the consequences of the reactions in existing structures.

Intensive research has been conducted to develop this needed information. As a result, concrete structures can now be designed and built with a high degree of assurance that excessive expansion due to AAR will not occur and cause progressive degradation of the concrete.

This report provides information for those involved with the design and construction of concrete, to make them aware of the factors involved in AAR and the means that are available to control it.

### **1.1—Historical perspective**

**1.1.1 Alkali-silica reactivity**—Alkali-silica reactivity (ASR) was first recognized in concrete pavement in California by Stanton (1940, 1942) of the California State Division of Highways. Stanton's early laboratory work demonstrated that expansion and cracking resulted when certain combinations of high-alkali cement and aggregate were combined in mortar bars stored in containers at very high relative humidity. Two important conclusions were drawn from this work: First, expansions resulting from ASR in damp mortar bars were negligible when alkali levels in cement were less than 0.60 percent, expressed as equivalent sodium oxide (percent  $\text{Na}_2\text{O}_e = \text{percent Na}_2\text{O} + 0.658 \times \text{percent K}_2\text{O}$ ). A second conclusion was that the partial replacement of high-alkali cement with a suitable pozzolanic material prevented excessive expansions. Thus, foundations for the engineering control of the reaction were developed. This work also formed the basis for ASTM C 227, the mortar-bar test procedure.

Based on Stanton's work, the U.S. Bureau of Reclamation (Meissner, 1941) conducted investigations of abnormal cracking in concrete dams. Meissner's findings generally corroborated those of Stanton, and lent further credence to the importance of cement alkali level, aggregate composition, and environmental requirements in the development of expansion due to ASR. One outcome of this work was the development of the quick chemical test, ASTM C 289 (Mielenz et al., 1948).

In the 1940s, other agencies both in the U.S. and other countries conducted further studies on ASR. These agencies included the Army Corps of Engineers, the Bureau of

Public Roads, and the Portland Cement Association in the U.S., the Australian Council for Scientific and Industrial Research (Alderman et al., 1947) and the Danish National Committee for Alkali Aggregate Research. They furthered the understanding of relationships among cement composition, aggregate types, mixture proportions of mortar and concrete, and expansion.

Other workers during this period and in the early 1950s concentrated on clarifying mechanisms expansive and non-expansive reactions. At the Portland Cement Association, Hansen (1944) proposed that osmotic pressures generated during swelling of gel reaction products were responsible for the observed expansion. Powers and Steinour (1955) proposed a variant of this hypothesis, while later researchers attempted to refine these ideas of expansion mechanisms. As with other aspects of the reaction, gaps still exist, particularly in the quantitative aspects of reactivity.

Mather (1993) reviewed the use of admixtures to prevent excessive expansion due to alkali-silica reaction. Stanton (1940, 1942) reported that 25 percent pumicite, a pozzolan, "seems to be effective" in reducing "the expansion to a negligible amount at early periods." The proposal to use pozzolan to prevent excessive expansion due to ASR apparently was first advanced by Hanna (1947). The 1963 report of ACI Committee 212 indicated that there had been "a few instances" where a mineral admixture was used to provide protection with high-alkali cement and reactive aggregate. In spite of this statement, Mather (1993) reported that he could find no documented evidence of such use. However, Rogers (1987) had written, "At the Lower Notch Dam on the Montreal River, 20 percent fly ash replacement was used successfully to prevent cracking of concrete containing argillite and graywacke." This appears to have been the first documented case where a pozzolan was used with cement known to have high-alkali content and with aggregate known to be potentially deleteriously reactive. A similar case was reported from Wales (Blackwell and Pettifer, 1992).

Test methods currently in use to determine potential for expansive reactivity, particularly in the United States, derive primarily from work carried out in the 1940s. However, research efforts in several countries today indicate a promise of newer, more reliable tests to identify potentially deleteriously reactive cement-aggregate combinations.

**1.1.2 Alkali-carbonate reactivity**—Alkali-carbonate reactivity (ACR) was identified as causing a type of progressive deterioration of concrete by Swenson (1957) of the National Research Council of Canada. He found that an alkali-sensitive reaction had developed in concrete containing argillaceous calcitic dolomite aggregate that appeared to be different than the alkali-silica reaction. Subsequent work by Swenson (1957), Swenson and Gillott (1960), and Gillott (1963) in Canada, and by various other agencies in Canada and the United States, further elucidated factors that affected the magnitude of expansion resulting from the reaction. Noteworthy among researchers in the United States were Newlon and Sherwood (1962), Newlon et al. (1972a,b), and Hadley (1961, 1964). Two hypotheses on the mechanism of ACR were developed, both of which still are cited.

Because rock susceptible to this type of reaction is relatively rare, and is often unacceptable for use as concrete aggregate for other reasons, reported occurrences of deleterious ACR in actual structures are relatively few. The only area where it appears to have developed to any great extent is in southern Ontario, Canada, in the vicinities of Kingston and Cornwall. Isolated occurrences in concrete structures have been found in the United States in Indiana, Kentucky, Tennessee, and Virginia. So-called "alkali-dolomite reactions" involving dolomitic limestones and dolostones have also been recognized in China (Tang et al., 1996).

## 1.2—Scope of report

This report is intended to provide information on ASR and ACR. Accordingly, chapters in this report provide an overview of the nature of both ASR and ACR reactions, the means of avoiding the deleterious effects of each reaction, methods of testing for potential expansion of cement-aggregate combinations, measures to prevent deleterious reactions, and recommendations for evaluation and repair of existing structures.

## CHAPTER 2—MANIFESTATIONS OF DISTRESS DUE TO ALKALI-SILICA REACTIVITY

### 2.1—Introduction

The most evident manifestations of deleterious ASR in a concrete structure are concrete cracking, displacement of structural members due to internal expansion of the concrete, and popouts. However, these features should not be used as the only indicators in the diagnosis of ASR in a concrete structure. Cracking in concrete is essentially the result of the presence of excessive tensile stress within the concrete, which can be caused by external forces such as load, or by development of a differential volume change within the concrete. Early contraction, too large thermal gradients during curing of the concrete, corrosion of embedded reinforcement, freezing and thawing, and internal and external sulfate attack are some of the mechanisms that also can lead to the formation of cracks in concrete.

Diagnosing ASR-related cracking requires the additional identification of ASR reaction product in the concrete and, most importantly, requires positive indications that this product has led to the generation of tensile stresses sufficiently large that the tensile strength of the concrete was exceeded.

### 2.2—Cracking mechanisms

Little is usually known about the time necessary for development of cracks in ASR-affected concrete in the field. This is partly due to the heterogeneous nature of concrete as a material, and to the fact that the reaction kinetics of ASR are practically unexplored. For example:

1. Does the reaction product swell at the place it forms, or at a different place where it migrates after formation?
2. How rapidly are expansive pressures generated from the swelling reaction product?
3. How do these mechanisms produce cracks in the concrete?

However, some inferences can be made based on observing ASR-affected concrete in the field and in the laboratory. For example, in an unreinforced and unconfined concrete element, such as a concrete slab or beam, the largest

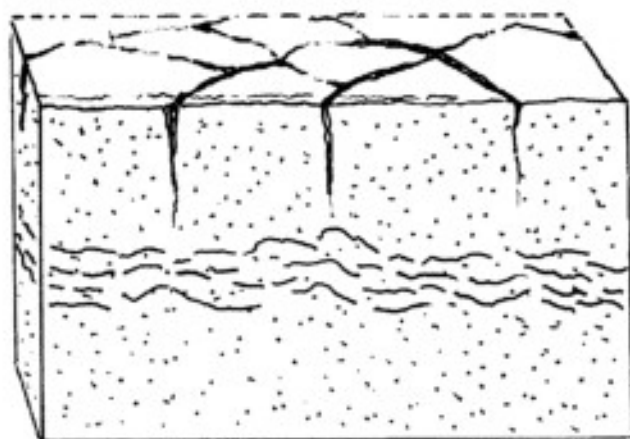


Fig. 1—Sketch showing typical features of surface map-cracking and subparallel cracks in concrete with ASR. Stresses due to ASR grades from horizontal tension near the surface to horizontal compression and vertical tension with depth.



Fig. 2—Photograph of a parapet wall showing typical map-cracking at the surface.

degree of deformation of the concrete will occur in the direction of least restraint.

Fig. 1 is a sketch of the surface and a cross section of a concrete slab undergoing ASR. Swelling due to the uptake of water by alkali-silica reaction product generates tensile stresses that lead to the local formation of fine cracks in the concrete slab. Since the least restraint occurs in a direction perpendicular to the surface, the cracks tend to align themselves subparallel to the surface. The expansion occurring within the concrete causes tension to occur in the concrete near the surface of the slab, where less expansion is taking place due to a lower rate of reaction. These tensile stresses are relieved by the formation of relatively wider cracks perpendicular to the surface. Viewed from above, these cracks tend to occur in a polygonal pattern that is the basis for the term "map-cracking." Fig. 2 shows the typical appearance of a concrete surface which has developed map-cracking due to ASR. Fig. 3 is a bridge deck core showing both the vertical and horizontal cracking due to ASR.

The relaxation of tension in the surface concrete allows further cracking subparallel to the surface to occur further

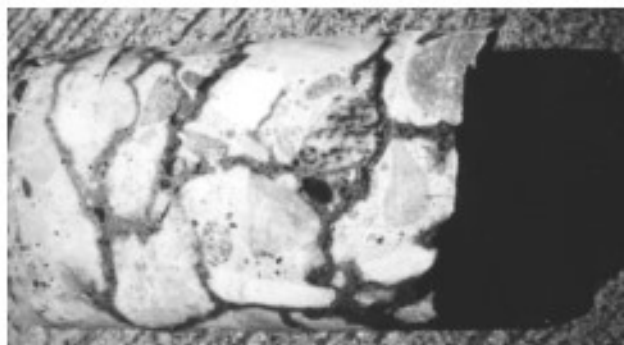


Fig. 3—Bridge deck core showing both vertical and horizontal cracking due to ASR (top of core to the right). Cracks are emphasized by retained moisture.

inward from the surface. With an excessive supply of external alkali and sufficient amounts of reactive silica in the aggregates, this subparallel cracking could theoretically continue to occur throughout the concrete. However, field experience shows that the subparallel cracking seldom goes deeper than 300 to 400 mm in unreinforced structures. In reinforced concrete, the cracking rarely progresses below the level of the reinforcement. It appears reasonable to assume that any reacting particle lying within concrete restrained by the reinforcement experiences confining pressures that exceed the expansive forces generated by the uptake of water by the reaction product. Cracking usually will not occur and the expansive pressures will most likely be accommodated by creep of the surrounding concrete. When evaluating specific structures, the type, location, and amount of reinforcement must be taken into account when considering the potential for cracking due to ASR.

The external appearance of the crack pattern in a concrete member is closely related to the stress distribution within the concrete. The distribution of strain is, among other things, controlled by the location and type of reinforcement, and the structural load imposed upon the concrete. Expansion of a concrete element will tend to occur in the direction of least restraint. Cracks caused by the expansion due to ASR tend to align parallel to the direction of maximum restraint.

Fig. 4 shows a pavement affected by ASR. In this case, stress distribution has caused the cracks to orient parallel to the slab free edges (longitudinally) over most of the slab surface, with additional cracks parallel to the transverse joints (also a free edge) in the areas near these joints.

### 2.3—Expansion and other indicators of alkali-silica reactivity

The development of cracks in a concrete structure due to ASR is caused by a volume increase that can be observed directly, either as a closure of expansion joints or by the misalignment of one structural element with respect to another. Also, the volume increase can be inferred from, for instance, an increasing difficulty in the operation of machinery attached to the concrete (for example, spillway gates in a dam). Fig. 5 depicts the closure of a joint and extrusion of the joint-filling material due to ASR expansion.





Fig. 4—Photograph of pavement affected by ASR. Typical longitudinal cracking is parallel to the slab free edges, with additional transverse cracks in the areas near the transverse joints.

Monitoring the amount and rate of expansion of a structure often is necessary to assess its structural integrity. Several ways of monitoring the rate of expansion exist. For example, the long-term change of length between reference points mounted on the concrete surface can be measured.

The method most suitable for monitoring the expansion must be considered in each specific case. However, it must be remembered that such observations should cover entire structural units. Measurements and summations of individual crack widths in a concrete structure are too uncertain for this purpose, because shrinkage of the concrete between the cracks will contribute to the opening of the cracks. Measurements of crack widths may thus give a false indication of the expansion in the concrete. Likewise, gathering sufficient data to be able to correct for the effects of variations in ambient temperature and humidity is important. As these variations are often seasonal or more frequent, at least several years of measurements are normally necessary before definite conclusions can be reached about the rate of ASR-induced expansion in the structure.

Popouts and exudation of gel onto the concrete surface also may indicate ASR but it does not, by itself, indicate excessive expansion of the concrete. Although the presence of alkali-silica gel on the surface of the concrete indicates the presence of ASR, it does not mean that the cracks were formed by the gel on its way to the surface. Discoloration often borders the crack in ASR-affected concrete, but discoloration may also occur for several other reasons (for example, leaching or algae growth).

"Popouts" refer to the breaking away of small conical fragments from the surface of the concrete, and can, in climates where freezing takes place, be the result of freezing of water-saturated, porous aggregate particles lying near the surface. Examining the popouts for the presence of gel is important; it can indicate whether ASR has taken place. As reactive particles are often porous and may be susceptible to



Fig. 5—Photograph showing closure of a joint and extrusion of joint-filling material due to ASR expansion.

both frost damage and ASR, unambiguously identifying the reason for the popouts is often difficult.

#### 2.4—Alkali-silica reactivity reaction factors

The distribution of ASR in a concrete structure is often highly variable, both with regard to appearance and intensity.

ASR involves a chemical reaction, and for the reaction to occur, the following components must be present: water, reactive silica, and a high concentration of hydroxyl ions (high pH). Likewise, the concentration and distribution of these components and the ambient temperature have a significant influence on the rate and deleterious effect of the reaction. A concrete structure with ASR commonly exhibits widely differing signs of deterioration in different places. Concrete exposed to dry, interior environments without water normally does not develop cracking from ASR, even though reactive silica and alkalis are present in the concrete.

The most vulnerable parts of a concrete structure are those exposed to a warm and humid environment. Field experience and laboratory work also indicate that concrete exposed to repeated drying and wetting cycles is more likely to develop excessive expansion due to ASR than concrete stored at a uniform moisture content.

Where it is possible for water to accumulate, such as from rain or snow, a rapid progression of ASR is often observed. This applies to every free-standing concrete surface that has not been protected. Cracking in free standing walls, exposed beams, or parapets is commonly observed. Degradation of these exposed concrete elements also is enhanced where freezing and thawing occurs in conjunction with ASR.

Cracking also tends to occur in concrete embedded in moist soil, such as in bases and foundations. The largest amount of cracking tends to occur at or near the soil surface where the concrete experiences the largest fluctuations in wetting and drying.

Sodium chloride also has been reported to promote ASR due to the external supply of sodium under conditions where the chloride ion reacts to form, for example, chloroaluminates. Salt concentrations, as found in ordinary sea water, do not seem to provoke ASR, but when the sodium chloride concentration exceeds approximately 5 percent (Chatterji et

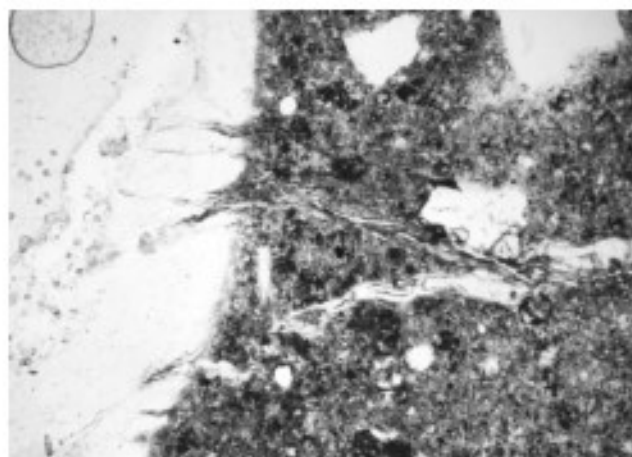


Fig. 6—Photomicrograph of gel-filled cracks extending from within a chert aggregate particle into the adjacent cement paste.

al., 1987), for instance due to evaporation, the rate of ASR may rapidly increase. The accelerating effect of sodium chloride on ASR has been reported to be a serious problem in areas where sodium chloride is used as a deicing agent on pavements, sidewalks, and in parking structures. However, recently reported research does not support this hypothesis (Duchesne and Berube, 1996).

The physical characteristics of concrete also can be decisive in determining the degree and rate of deterioration due to ASR. Air entrainment has been reported to reduce the degree of expansion due to ASR. However, air entrainment by itself should not be regarded as an effective means of preventing excessive expansion due to ASR.

The effect of water-cement ratio on ASR is more difficult to determine since a low water-cement ratio may reduce the availability of water for imbibition by the reaction product, but at the same time raising the alkali concentration of the pore fluid.

Field experience shows that initial cracking in a concrete element, such as thermal or drying shrinkage cracks, can have an accelerating effect on the development of excessive expansion due to ASR. This is probably due to both the capillary effect of the cracks promoting ingress of water into the concrete, and the reaction-product swelling that widens existing cracks instead of initiating new ones.

## 2.5—Microscopic evidence of alkali-silica reactivity

In most cases, absolutely diagnosing distress caused by ASR in a structure based solely on a visual examination of the concrete is difficult. In the final assessment of the causes of deterioration it is necessary to obtain samples for examination and testing. Petrographic examination should then be conducted in accordance with ASTM C 856.

In some cases, deposits of the reaction product, a transparent alkali-silica gel, are found. Fig. 6 shows a close-up of gel-filled cracks extending from within a chert aggregate particle into the adjacent cement paste. The appearance of the gel may vary depending on whether it is within an aggregate particle or in the paste. Inside an aggregate particle, the

gel may appear grainy, while it often appears more glassy within the paste.

In some cases, the amount of gel appears to be limited, while the amount of concrete cracking due to ASR can be rather high. In other cases, this behavior of the concrete is reversed, where alkali-silica gel is seen to replace practically the entire aggregate particle, apparently without causing any significant cracking.

The presence of discolored rims in reactive aggregate particles in the concrete is an indicator of ASR (Dolar-Mantuani, 1983). The presence of such reaction rims should be approached with some caution, as the formation of rims in aggregate particles can also be due to other mechanisms. Weathered outer layers of the individual particles are often seen in natural gravels, and even crushed rock can develop weathering rims if it has been stockpiled for some time. These weathering rims are often indistinguishable from reaction rims formed in concrete. Caution must also be used in identifying ASR based on deposits surrounding aggregate particles on fractured concrete surfaces. The fractured surface may have occurred along an old crack which could contain a variety of deposits (Thaulow et al., 1989). Fig. 7 shows a crushed aggregate with reaction rims (since the aggregate is crushed, it indicates the rims have formed after crushing, i.e., in the concrete) in a concrete with ASR (note cracking and ASR gel).

When observed in thin section, the disseminated calcium hydroxide in the cement paste often is depleted in the vicinity of reactive-aggregate particles. This phenomenon often occurs before other signs of ASR, such as cracking and gel formation, and is therefore helpful in detecting reacted particles in the concrete.

## CHAPTER 3—ALKALI-SILICA REACTIVITY MECHANISMS

### 3.1—Factors influencing the reaction

Three basic conditions must exist for ASR to proceed in concrete. These conditions include high pH, moisture, and reactive silica. The rate of the reaction is influenced by temperature.

**3.1.1 Cement alkali levels**—Early investigators recognized that the alkali content of portland cement had a direct influence on potential expansion (Stanton, 1940, 1942). The two alkali constituents are reported from chemical analysis as sodium oxide and potassium oxide. The total equivalent alkali is calculated as percent  $\text{Na}_2\text{O}$  plus  $0.658 \times$  percent  $\text{K}_2\text{O}$ , and the resulting percentage is described as equivalent  $\text{Na}_2\text{O}(\text{Na}_2\text{O}_2)$ . The concept has proven useful in the study of ASR. Diamond (1989) showed the relationship between the cement-alkali content and the OH-ion concentration (pH) of the concrete pore fluid. The latter is the driving factor in the chemical process of AAR.

A limit of 0.60 percent on the  $\text{Na}_2\text{O}$  equivalent alkali content of portland cement (low-alkali cement) has often been used in specifications to minimize deterioration of concrete when reactive aggregates are used. However, there have been cases where significant damage resulted despite the use of low-alkali cement (Hadley, 1968; Lerch, 1959; Stark, 1978, 1980; Tuthill, 1980, 1982; Ozol and Dusenberry, 1992). Based on his experience, Tuthill (1980) suggested that

a limit of 0.40 percent on the equivalent alkali content was more appropriate. An ASTM Committee C1 working group (Blanks, 1946) reported on laboratory tests of mortars containing natural aggregates, finding that excessive expansions were encountered with cements having alkali contents of 0.58 percent or greater. With alkali contents of 0.40 percent or less, excessive expansion in the mortars did not occur.

Several factors may be responsible for the problems encountered with low-alkali cements:

1. Concretes made using portland cement alone are relatively more permeable than similar concretes made with blends of portland cement and slag or pozzolan. Cyclic wetting and drying, freezing and thawing, as well as electrical currents can cause alkali migration and concentration in concrete (Xu and Hooton, 1993). Consequently, a given supply of alkali that might be tolerated if uniformly distributed throughout the concrete can become concentrated in certain areas in amounts high enough to cause distress. Lerch (1959) and Hadley (1968) reported on a pavement where damaging ASR was linked to wetting and drying; Ozol (1990) reported on the exacerbating effect of electrical currents on ASR of concrete in piers at a power substation. Moore (1978) had previously reported laboratory results indicating that passage of direct electric current through a mortar specimen containing reactive siliceous aggregate appeared to accelerate the disruption due to ASR.

2. The relative permeability of concrete also permits the migration within the concrete of alkalis from other concrete materials as well as the ingress of alkalis from external sources such as deicing salt. Studies (Grattan-Bellew, 1994, 1995; Berube et al., 1996; and Stark and Bhatti, 1986) have shown that significant amounts of alkalis can be leached from certain types of aggregates by concrete pore solutions.

3. Because the relevant issue with respect to ASR is the concentration of hydroxyl ion (pH) in the concrete, the cement factor plays an important role that is disregarded in the traditional consideration of cement-alkali content ( $\text{Na}_2\text{O}_e$ ). Various limits on the mass of alkali-per-unit volume of concrete have been suggested as a more appropriate method to prevent damaging ASR. A maximum value of  $3 \text{ kg/m}^3$  is often cited as sufficient to prevent damage in the presence of reactive aggregates (Concrete Society Working Party 1987; and Portland Cement Association 1994). The limit includes alkalis contributed from pozzolans or slag, as well as the cement. The Canadian Standards Association (CSA A23.1) places a limit of  $3 \text{ kg/m}^3$  on the alkali contribution from the cement alone. Ozol (1990) reported on field occurrences of ASR where chemical analyses suggested concrete alkali contents of  $1.8 \text{ kg/m}^3$ , and Ozol and Dusenberry (1992) reported ASR in concrete with an alkali content of  $2.3 \text{ kg/m}^3$ . Based on laboratory tests, Johnston (1986) suggests that concrete alkali contents less than 0.05 percent ( $1.2 \text{ kg/m}^3$ , for concrete with a density of  $2320 \text{ kg/m}^3$ ) were clearly safe, whereas alkali contents greater than 0.10 percent ( $2.3 \text{ kg/m}^3$ ) would clearly cause problems when used with reactive aggregate.

4. The alkali content that can be tolerated may be related to the inherent reactivity of the aggregate. Woolf (1952)

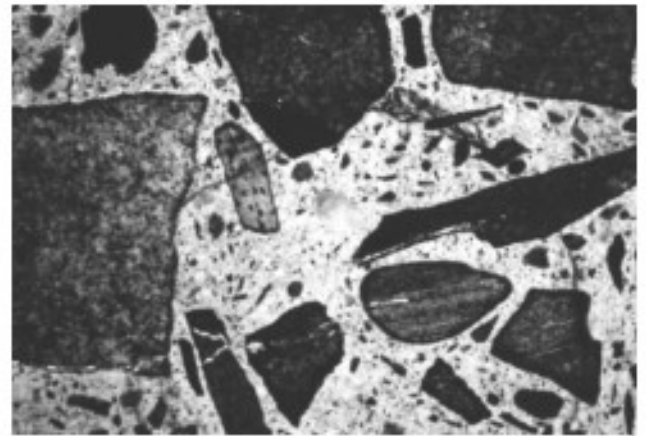


Fig. 7—Polished section showing crushed aggregate with reaction rims in concrete with ASR (note cracking and ASR gel).

reported on laboratory tests where the alkali content at which maximum expansion of mortar bars occurred varied with the percentage of highly reactive material in the aggregate. Stark et al. (1993) investigated the concept of using the accelerated mortar bar expansion test to determine a "safe alkali content" for a particular aggregate.

Although ASR problems can be minimized by limiting the alkali content of the cement or concrete, consideration must be given to the potential for alkali migration and concentration within the concrete to determine an appropriate limit. The advantages of using pozzolans or slag to produce ASR-resistant concretes with low permeability also should be considered.

**3.1.2 Moisture**—Moisture must be available for ASR to proceed, and below about 80 percent internal relative humidity the reaction will cease. For ordinary concretes, some portion of the original mixing water is usually available for a long period even in dry service conditions. However, for low water to cementitious ratio mixtures the water may be used up by hydration of cement. In service (such as slabs on grade) where the concrete has an external source of water, the reaction will continue until one of the reactive constituents is used up.

**3.1.3 Reactive silica**—Researchers first believed there was a limited group of susceptible aggregate constituents such as opal, chert containing chalcedony, and some glassy volcanic rocks. It is now recognized that ASR can occur with a wider range of siliceous aggregate constituents. Various other metastable forms of silica can be involved. Reactivity depends not only on the mineralogy but also on the mechanics of formation of the aggregate material, and the degree of deformation of quartz. Chapter 4 discusses reactive silica in detail.

**3.1.4 Temperature**—As temperatures increase, the rate of ASR increases. With given concrete materials and proportions, the reaction will take place more rapidly under warmer conditions. While this factor has not been quantified, it should be kept in mind when considering approaches to prevent ASR.

### 3.2—Basic mechanisms of reaction and expansion

The mechanisms of alkali-silica reaction and expansion have been under investigation since about 1940. The



hydroxyl ions present in the pore fluid in concrete react chemically with various forms of silica present in many aggregates. The sodium and potassium alkalis play two roles in the reaction. First, higher percentages of these alkalis in the concrete result in higher concentrations of hydroxyl ions in the concrete pore fluid (higher pH). The more alkaline (higher pH) the pore fluid, the more readily it attacks (reacts with) the reactive silica. Once in solution, the silica reacts with the alkalis forming alkali silica gel. This alkali-silica gel then imbibes water and swells so that its volume is greater than that of the individual reacted materials, and expansive stress is exerted on the concrete.

Where the reactive ingredients are present in the fresh concrete the reaction begins at the contact surface of the cement paste and the aggregate particle. Often the earliest indication is a discolored reaction rim within the surface of the aggregate particles. Increasing gel formation results in progressive cracking within the aggregate particles and in the matrix around the particles. Often a near-surface peripheral crack is evident in the aggregate.

Where the source of the alkali is external to the concrete, gel formation will advance on a front from the exposed faces. Alkalis may become available from such sources as deicing salts, seawater, and industrial solutions.

Rates of reactions are often low, and evidence of exudation of gel, pop-outs, cracking, and mass expansion may not be seen for years.

In a few cases, gel formation has been detected but has not caused disruption because of a relatively volume-stable replacement of aggregate material by gel. In most instances of ASR, however, disruptive expansive forces are generated.

## CHAPTER 4—PETROGRAPHY OF ALKALI-SILICA REACTIVE AGGREGATE

### 4.1—Introduction

The petrography of alkali-silica reactive aggregates is discussed here, using petrographic terms which may not be familiar to engineers. These terms will be explained where first used, to the extent possible. Alkali-silica reactive aggregate constituents can be classified in two broad categories: 1) naturally occurring forms of essentially pure silica, into which minerals, mineraloids, and volcanic glasses are grouped; and 2) synthetic or artificial siliceous materials. The reactivity of an aggregate: that is whether it reacts quickly or slowly, and also the amount of sodium equivalent alkalis in the concrete necessary to cause it to react, depends on the composition, geologic origin and textural characteristics of the rock(s) from which the aggregate is derived. For further discussion of these aspects, see Stark, Morgan et al. (1993), Dolar-Mantuani (1983), and Grattan-Bellew (1983).

### 4.2—Potentially reactive natural silica constituents

**4.2.1 Opal**—Opal, either alone or as a component in a rock is probably the most alkali-silica reactive natural material (Stanton, 1940, 1942). As described in ASTM C 294, opal is "a hydrous form of silica ( $\text{SiO}_2 \cdot n\text{H}_2\text{O}$ ) that occurs without

characteristic form or internal crystalline arrangement as determined by ordinary visible light methods."

Optically, opal is colorless to pale gray or brown. Its index of refraction ranges from 1.40 to 1.46, and is variable based on water content (Kerr, 1959; Mather, 1945). Its form is colloform (in rounded masses) crusts, cavity fillings, or linings in seams as replacement of wood, other organic materials, or feldspars. More often it is massive without any particular structure although opals fall into several crystallographic categories. Some opals appear completely amorphous while others are composed of poorly to moderately well crystallized cristobalite, disordered cristobalite-tridymite intergrowths, or disordered tridymite (Diamond, 1976).

**4.2.2 Chalcedony**—ASTM C 294 describes chalcedony as a fibrous, microcrystalline form of silica. Chalcedony has been considered both as a distinct mineral and a variety of quartz. It occurs in massive form, as cavity fillings, as cementing material, and as replacement material for fossils and for opal in diatomite. It is often a major constituent of chert.

Indices of refraction range between 1.534 and 1.538; that is, lower than the lower index of refraction of quartz. Chalcedony is colorless-to-pale brown in thin section and often bluish-white in reflected light. Extinction (the optical orientation for certain minerals at which no light is transmitted when viewed through crossed polarizing lenses) is parallel to the length of the fibers (Kerr, 1959).

**4.2.3 Quartz**—Coarse megascopically-crystalline (visible with the unaided eye) quartz is normally not reactive. However, there have been indications that megascopic unstrained (undeformed) quartz may, with certain irregularities or inclusions present, be slowly reactive and expansive given sufficient time and exposure to alkaline conditions (Diamond, 1976; Dolar-Mantuani, 1975).

Microcrystalline to cryptocrystalline (so finely crystalline that the crystals can not be seen with a hand lens) quartz, components of some cherts, have been found extremely susceptible to reaction.

Highly fractured quartz in quartzites and gneisses, and strained quartz are also alkali reactive. Studies relating optical properties of strained quartz to mortar-bar expansion indicate an apparent correlation based on measured undulatory extinction angles (extinction, see above, occurs over a range of crystal orientation angles) of the strained quartz (Mather, 1973; Dolar-Mantuani, 1975). However, Grattan-Bellew (1992) suggests that this apparent correlation may be due to the presence of microcrystalline quartz in rocks containing strained macrocrystalline quartz grains. Characterization of reactive aggregates containing strained quartz has also been investigated by scanning electron microscope and infrared spectroscopy (Mullick et al., 1985).

**4.2.4 Cristobalite**—Cristobalite is found in minute square crystals or aggregates in the cavities of obsidian, rhyolite, andesite, and basalt. It also occurs as a constituent of some specimens of opal. Cristobalite has been reported as a constituent in some blast-furnace slags (McCaffery et al., 1927), and therefore the composition of slags being considered for use as aggregate should be checked. Colorless in thin section, it is pseudoisometric (has the appearance, but



not the optical properties of the isometric crystal class) with principal indices of refraction of 1.484 and 1.487 (Kerr, 1959).

**4.2.5 Tridymite**—Tridymite occurs in minute, euhedral (well formed) crystals as cavity linings in volcanic igneous rocks such as obsidian, rhyolite, andesite, and as a porous crystalline aggregate. The crystals are six-sided, orthorhombic, thin, and tabular with characteristic wedge-shaped twins (crystal intergrowths). In the absence of twinned crystals, tridymite very closely resembles cristobalite, however the index of refraction of individual crystals is diagnostic: for tridymite,  $n < 1.480$ ; for cristobalite,  $n > 1.480$  (Kerr, 1959). The principal indices for tridymite are 1.469, 1.469, and 1.473.

**4.2.6 Volcanic glasses**—Volcanic glasses occur in virtually all volcanic rocks. Igneous rocks are described as acid if they contain more than 66 percent silica, intermediate when silica contents range from 52 to 66 percent, and basic when silica contents are less than 52 percent. This corresponds to index of refraction ranges of  $n < 1.57$  for acidic and intermediate glasses, and  $n > 1.57$  for basic glasses (Williams et al., 1954 and Mather, 1948). Acid and intermediate glasses tend to be alkali reactive, with reactivity decreasing as the amount of silica decreases. Thus, the high-silica glasses of rhyolites, dacites, and andesites (pumice and obsidian) are more reactive, while basaltic glasses are less reactive.

**4.2.7 Chert**—Chert is a general term applied to variously colored, fine-grained siliceous rocks composed of microcrystalline or cryptocrystalline quartz, chalcedony, opal, or mixtures of these constituents. Cherts can be dense or porous and chalky. The dense cherts are tough, with a waxy-to-greasy luster and conchoidal fracture. Chert particles may be gray, brown, white, red, green, blue, or black. ASTM C 294 and Mather (1948) delineate the chert varieties flint, jasper, agate, and novaculite primarily based on color. The porous varieties are usually chalky, lighter in color, and have a splintery fracture. In addition to potential reactivity with cement alkalis, porous cherts may cause cracking or popouts in concrete if frozen and thawed while critically saturated (Mielenz, 1956).

Chert occurs as nodules, lenses, or beds in calcareous and noncalcareous sedimentary rocks, and as discrete particles in sand and gravel. Impure cherts commonly grade into siliceous limestones (Diamond, 1976).

Most cherts are alkali-silica reactive. The degree of reactivity is dependent on several factors, including the mineralogical composition and internal structure of the chert, the amount of reactive chert relative to that of the total aggregate, and the particle size distribution.

**4.2.8 Volcanic rocks**—Acidic and intermediate volcanic rocks that are alkali-silica reactive include some rhyolites, dacites, latites, and andesites. The related porphyries (rocks with larger crystals in a fine grained matrix) and tuffs (rock composed of compacted volcanic fragments) of these rock types also may be alkali reactive. The reactivity of these rocks can be attributed to the texture and composition of glassy or partially glassy groundmass (matrix of the rock).

Some basic volcanic glasses and rocks are also alkali-silica reactive (Gudmundsson, 1971). Basalts containing highly siliceous interstitial glasses are slowly alkali-silica

reactive, and produce the expansion and map cracking typical of ASR in concrete.

**4.2.9 Argillites, meta-graywackes, phyllites, and slates**—These metamorphosed sedimentary rocks can react with cement alkalis to cause expansion and cracking. The mineralogy of these rock types is mainly quartz, feldspars, and phyllosilicates ("platey" silicates, such as mica). Associated minerals include magnetite, hematite, pyrite, graphite, and tourmaline (Gillott et al., 1973). Carbonate minerals also may be present in phyllites and slates (Regourd et al., 1981).

The reactive component in these rocks is finely divided or optically strained quartz, sometimes exhibiting inclusions (Dolar-Mantuani, 1983). Others believe the reactive component in these rocks to be finely divided quartz (microcrystalline quartz) exhibiting undulatory extinction, and sometimes fluid inclusions (Thompson et al., 1994; Langley et al., 1993; DeMerchant et al., 1995).

#### 4.3—Potentially reactive synthetic materials

**4.3.1 Silica brick**—The principal constituent of silica brick is tridymite, with cristobalite also present (Kerr, 1959). Silica brick is made by using finely ground quartzites of low iron content.

**4.3.2 Synthetic glasses**—Many synthetic glasses are alkali-silica reactive (Mukherjee and Bickley, 1987). The aggregate used as a standard reactive aggregate in ASTM C 441 is Pyrex manufactured by Corning Glass Works. This glass contains about 80 percent  $\text{SiO}_2$ .

The synthetic glasses generally are optically isotropic except for minor inclusions and occasional anisotropic grains (due to incomplete fusion or recrystallization). The index of refraction ranges from 1.510 to 1.555 (Meissner et al., 1942).

**4.3.3 Coatings**—Aggregates that are inherently innocuous may become deleterious because of surface coatings. Coatings may contain materials susceptible to reaction with cement alkalis, such as opal. The coatings may also contain salts of potassium or sodium which, if dissolved, can contribute to deleterious chemical reactions with alkali-reactive aggregate (Stanton, 1942).

## CHAPTER 5—MEASURES TO PREVENT ALKALI-SILICA REACTIVITY

### 5.1—Overview

A distinction is made between ASR reaction and the expansion resulting from the reaction. ASR gel can form as a result of the reaction, but it is not always the direct cause of distress observed in concrete, as outlined in Section 3.1. ASR and the subsequent expansion of concrete occur only when the following conditions are present (as documented by Stark et al., 1993; Kosmatka and Fiorata, 1991; Mid-Atlantic Regional Technical Committee, 1993a, 1993b; Swamy, 1992; Helmuth, 1993; Mather, 1995):

1. Concrete is sufficiently moist in service.
2. Concrete contains aggregates with siliceous constituents that are alkali-silica reactive. These constituents may include inter-layer silicate minerals, which may cause expansion in some cases, but typically react at a slower rate. The amount of reactive aggregate required for the reaction to occur may vary widely

according to aggregate type and other factors not fully understood. Some reactive forms of silica have a pessimum concentration, above and below which the reaction is less severe.

3. A source of sufficient alkalis, that is, sodium and potassium, is available that can: 1) raise the pH of the pore fluid by allowing more hydroxyl ions to remain in solution (this higher pH of the pore solution increases the solubility of the reactive silica); and 2) React with the dissolved silica to form alkali silica gel.

Strategies to prevent ASR expansion focus on controlling one or more of the three preceding conditions, that is:

1. Control the available moisture.
2. Control the type and amount of potentially reactive siliceous constituents in the aggregate, or in the concrete.
3. Lower the pH of the concrete pore fluid, in order to decrease the solubility of the silica in the pore fluid. This is done by lowering the amount of available  $\text{Na}_2\text{O}_e$ , since this will lower the pH, as noted above.

### 5.2—Limiting moisture

Concrete structures exposed to the environment or in contact with the ground will generally be sufficiently moist internally to promote ASR reaction and the resulting expansion (Stark, 1991a). Water in the concrete pores transports the alkali and hydroxyl ions to sites of reactive aggregates. Subsequently, the ASR gel reaction product formed as a result of the reaction imbibes water and expands, thereby causing most of the expansion of the concrete mass. Keeping concrete dry will reduce the potential for ASR gel to swell and cause distress. As a practical matter, this is possible only for interior concrete in buildings, or above-ground concrete in dry climates.

A measure of available moisture is the internal relative humidity of concrete. Sufficient moisture will be available for expansion if the internal relative humidity of concrete exceeds 80 percent, referenced to a temperature in the range of 21 to 24 °C (Stark, 1991a). Concrete structures such as highway pavements and bridges, parking garages, and water-retaining and underwater structures are most susceptible to expansion. In arid regions, for concrete in contact with the ground, about 50 mm of the outer surface may dry out to less than the critical relative humidity (Stark, 1991a). However, this may increase the concentration of alkalis at the surface and initiate the reaction (Swamy, 1992).

Reducing the permeability of concrete to external moisture and salt solutions can reduce the potential for expansion. This can be accomplished by using a concrete mixture with a low water-cementitious ratio that will result in concrete with a low permeability, and by assuring adequate curing.

Concrete with a low permeability will reduce ion mobility and delay the reaction (Durand and Chen, 1991). There are negative effects of low permeability, however. The lower water content will result in a higher alkali concentration of the concrete pore solution. Also, the reduced pore space of a low water-cement ratio paste may not be able to accommodate as much gel expansion without distress. In these situations, increased expansions may be observed (Durand and Chen, 1991; Berube, Chouinard et al., 1996). In general, a better approach to reducing the permeability of concrete is by using

pozzolans or ground slag in the mix, which does not have the negative effect of simply reducing the water content.

Applying a coating or sealant to the concrete surface may be a viable option to reduce expansion if the concrete is not in contact with moist subgrade or other moisture source (Stark et al., 1993; Durand and Chen, 1991). Sealants will limit the ingress of moisture and minimize swelling of ASR gel. The effectiveness of a sealant will be reduced when applied to cracked concrete. Typically, the sealant should be applied after the concrete has had time to dry to a moisture level below that required for reaction and expansion to occur since sealing moisture inside the concrete can increase expansion. Breathable sealants that permit water vapor to escape or enter concrete, but prevent the ingress of moisture have been developed and may be useful. Evaluation of sealants that rely on a range of mechanisms, including methacrylate (Kamimoto and Wakasugi, 1992), silanes and siloxanes, have been conducted, with limited success reported in the laboratory and in field applications (Durand and Chen, 1991; Berube, Chouinard et al., 1996). In general, the cost of these materials limit their use.

### 5.3—Aggregate selection

Not all aggregates are susceptible to deleterious ASR, and therefore the seriousness of the problem often depends on the aggregate available. However, avoiding aggregates that contain reactive minerals or rocks is not an economical option in many regions. Reactive siliceous constituents are discussed in Section 4.2 of Chapter 4. The service record of an aggregate source is extremely useful in determining whether a potential problem exists. Evaluating existing concrete structures with similar material composition (including cement alkali levels), mixture proportions, and service conditions is necessary to establish the field service record of an aggregate. A petrographic examination, (see ASTM C 856), of field concrete that contains the aggregate in question should be a part of the evaluation. The concrete evaluated should have been in service for at least ten years.

When a new source of aggregate is being evaluated, a petrographic examination, according to ASTM C 295, of a representative sample of aggregate is useful in determining its potential for causing deleterious reactions in concrete and for planning remedial procedures, if it is reactive. The aggregate petrographic examination should identify any potentially reactive constituents and estimate their amount.

Depending on the procedures used, a petrographic examination may not detect small amounts of reactive material, such as opal or chert grains in limestone or coatings on aggregate particles. Recommendations for maximum limits of reactive constituents in an aggregate sample have been published (U.S. Army Corps of Engineers, 1994; Mid-Atlantic Regional Technical Committee, 1993b). The conclusions of a petrographic examination should be confirmed by one or more expansion tests, as discussed in Chapter 6.

If an aggregate has potential for causing ASR distress, several beneficiation strategies could be employed (Kosmatka and Fiorato, 1991; Dolar-Mantuani, 1983):

1. Diluting the reactive silica concentration by blending reactive and non-reactive constituents may be useful. For example, "limestone sweetening" has been a successful approach in some areas of the United States, where a potentially reactive gravel is blended with innocuous limestone. However, for some rapidly reactive constituents, such as opal, blending may produce a "pessimum" concentration of reactive constituents that makes the situation worse.

2. Selective quarrying, although in many cases difficult to accomplish in the field, can be employed to avoid strata of rock that are identified as potentially reactive.

3. Heavy media separation or rising-current classification has been used successfully in cases where reactive material has a low density, such as weathered opaline cherts. Such beneficiation techniques can significantly increase aggregate processing costs.

4. Washing and scrubbing will remove some of the reactive coatings, and possibly some of the reactive fines if this operation follows final crushing. Washing is particularly effective, and in some cases necessary, to remove sodium or potassium salts (alkali ion source) when aggregate is dredged from marine environments. Some reactive fines can act as a pozzolan, and reduce the likelihood of excessive expansion due to ASR later in the life of the concrete. This potential benefit must be evaluated, however, by conducting tests to determine the role the reactive fines will play.

5. Chemical treatment of aggregate may reduce its potential for reactivity. This could be accomplished by a coating technique or chemically neutralizing the reactive surface. Literature on aggregate treatments of this sort is sparse. This appears to be a new area of research. For example, wetting reactive aggregate in alkaline calcium phosphate solution and then drying is reported to result in reduced expansions (Hudec and Larbi, 1989). The Committee is not aware of any report that chemical treatment has been proven in the field.

Beneficiation methods need to be chosen based on the type of reactive material, operating conditions, and economics. The chosen strategy may be unique to a particular region or a particular type of aggregate deposit.

#### 5.4—Minimizing alkalis

The commonly employed procedure to minimize the potential for deleterious ASR deterioration is to control the alkali content of concrete ingredients in order to reduce the hydroxyl ion concentration (and therefore the pH) of the concrete pore solution. Because some forms of silica are more susceptible to ASR than others, the actual hydroxyl ion concentration required will vary.

The principal concrete ingredient contributing alkalis is portland cement (Stark et al., 1993; Kosmatka and Fiorato, 1991; Mid-Atlantic Regional Technical Committee, 1993a; Swamy, 1992; Helmuth, 1993). Smaller amounts of alkalis are contributed by pozzolans or slag. However, fly ashes with alkali contents above 5 percent may contribute significant quantities of alkali to the concrete pore solution. Mixing water (particularly if sea or brackish water is used), some chemical admixtures (like high-range, water-reducing admixtures (containing sodium) used at high dosage rates of

greater than 1300 mL/100 kg cement), perhaps some sodium or potassium feldspar in aggregates, and aggregates dredged from brackish marine environments (Mid-Atlantic Regional Technical Committee, 1993a) can contribute alkalis. Alkalis could also be leached into the concrete pore solution from certain types of aggregates (Grattan-Bellew, 1994; Kawamura et al., 1989; Stark and Bhatti, 1986; Berube et al., 1996).

External sources of alkalis for concrete that will be exposed to deicing salts and marine exposure in service should also be taken into consideration.

#### 5.5—Cement selection

Studies have shown that the hydroxyl ion concentration, or alkalinity, of the pore solution of mature cement pastes is related to the alkali content of the portland cement (Diamond, 1989) and the water-cement ratio (Helmuth, 1993). Cements with higher alkali contents produce higher expansions with the same aggregate in mortar-bar or concrete prism tests. ASTM C 150 recommends the optional use of a low-alkali (an alkali content of less than 0.60 percent  $\text{Na}_2\text{O}_e$ ) cement with a potentially reactive aggregate. However, cases have been reported where the use of cements within this range of alkali content have produced ASR-related expansion in concrete (Hadley, 1968; Lerch, 1959; Stark, 1980; Tuthill, 1980; Ozol and Dusenberry, 1992; Grattan-Bellew, 1981a; Rogers, 1990; Morgan, 1990).

Based on a 1994 survey (Gebhardt, 1994), the average alkali content of portland cements marketed in the United States and Canada is about 0.55 percent  $\text{Na}_2\text{O}_e$ , and ranges from 0.05 to 1.2 percent. The alkali content of cement primarily depends on the nature of the available raw materials, and therefore the availability of low-alkali cements may be limited in some regions. Further, environmental regulations have required the cement industry to modify kiln systems and reincorporate instead of wasting the alkali-rich kiln dust, making it difficult to reduce the alkali content of cements (Johansen, 1989).

#### 5.6—Finely divided materials other than portland cement

Ever since the first reported occurrence of ASR (Stanton, 1940), research has indicated that deleterious expansions due to ASR could be reduced by using raw or calcined natural pozzolans in concrete (Stanton, 1940, 1950). More recent research has confirmed that the use of ground granulated blast-furnace slag and pozzolanic materials like raw or calcined natural pozzolans, fly ash, rice husk ash, silica fume, and metakaolin are effective in minimizing the potential for excessive expansion of concrete due to ASR (Stark et al., 1993; Swamy, 1992; Durand and Chen, 1991). Good performance of concrete structures that were at least 25 years old and made with reactive aggregates and 20 to 30 percent fly ash replacement of the cement has been documented (Thomas, 1995).

The effects of a pozzolan or slag will depend on the particular pozzolan or slag, the reactivity of the aggregate, and the alkali content of the portland cement. In general, aggregates containing more rapidly reactive forms of silica will require higher replacement amounts of slag or pozzolan. Therefore, the



effectiveness of a particular cement-pozzolan or cement-slag combination should be tested prior to use. Testing as described in Section 5.7 should verify whether the pozzolan or slag reduces the expansion potential, as well as establish the replacement level that will control expansion with the particular aggregate, cement, and cement content being used. Other characteristics of concrete, such as setting time and strength, should also be tested to verify that they are not adversely affected.

The mechanism by which a pozzolanic material or slag reduces the potential ASR distress varies with the type used and can be a combination of one or more of the following (as documented by Helmuth, 1993; Chatterji, 1989; Nixon and Page, 1987; Dunstan, 1981):

1. When cement is partly replaced by a pozzolan or slag with a low available alkali content, the total alkali contribution of the cementitious materials is reduced. The use of pozzolans or ground granulated blast-furnace slag with cements whose alkali contents are at or below the 0.60 percent value has been recommended or required by some organizations (Lane and Ozyildirim, 1995; Thomas, 1995).

2. The cement-pozzolan reaction product or slag hydration product has a lower CaO:SiO<sub>2</sub> (C/S) ratio than the reaction product of the calcium silicates of the portland cement alone. This calcium silicate hydrate (C-S-H) gel has a greater capacity to entrap alkalis and reduce the pH of the concrete pore fluid.

3. Pozzolanic reactions consume calcium hydroxide, an abundant hydration product in concrete, and ASR gel that forms in a paste with reduced amounts of calcium hydroxide may have lower swelling characteristics.

4. The pozzolanic reaction or the slag hydration produces a denser paste by reducing the amount of calcium hydroxide and producing additional C-S-H gel. This is particularly significant as it occurs at the paste-aggregate interface. This effect reduces the mobility of ions and possibly slows the reaction rate. It also makes the concrete less permeable to external moisture and alkalis.

**5.6.1 Fly ash**—Fly ash is a finely divided residue resulting from the combustion of powdered coal. Because of its physical characteristics and its pozzolanic properties, it imparts several beneficial properties to concrete.

Based on its composition, fly ash is classified as Class F and Class C by ASTM C 618. Class F fly ash is usually derived from the combustion of anthracite or bituminous coal and generally contains less than 5 percent CaO by mass. Class C fly ash is usually derived from the combustion of lignite or subbituminous coal. Class C ashes typically contain 10-to-40 percent CaO by mass. As explained below, Class F ashes are generally more effective in mitigating ASR than Class C ashes.

Some of the alkalis in fly ash are encapsulated in the glassy particles and are released as the fly ash reacts in concrete. The role of fly ash alkalis and their net contribution to the alkalinity of the pore solution in concrete have been widely debated (Nixon and Page, 1987; Hobbs, 1989; and Thomas, 1995). The Canadian Standards Association (CSA) recommends that fly ash used for reducing the risk of deleterious expansion due to ASR should have a total alkali content less than 4.5 percent Na<sub>2</sub>O<sub>e</sub>, and a maximum water-soluble alkali

content of 0.5 percent Na<sub>2</sub>O<sub>e</sub> (Appendix of CSA A23.1). ASTM C 618 recommends an optional requirement that the maximum available alkali content of fly ash used to reduce ASR expansion be limited to 1.5 percent, by mass.

Class F fly ashes are generally efficient in controlling expansions related to ASR when used as a replacement for a portion of cement (Dunstan, 1981; Farbiarz et al., 1986; Robert, 1986; Lee, 1989). Normal proportions of Class F fly ash vary from 15 to 30 percent, by mass, of the cementitious material (Malhotra and Fournier, 1995). The effective replacement amount of Class F ash for portland cement should be determined by testing, as it will vary significantly based on the physical and chemical characteristics of the fly ash.

Some Class C fly ashes may be less efficient in reducing ASR expansions. Lower replacement amounts can cause higher expansions than a mixture not containing fly ash (Farbiarz et al., 1986, 1989). Some Class C fly ashes have hydraulic properties and react to a greater extent than Class F ashes. Due to a greater degree of reaction, Class C ash may release a larger portion of its total alkalis in concrete (Lee, 1989). Effective amounts of Class C fly ash to control ASR expansion may exceed 30 percent, by mass, of cementitious materials. In some cases, this effective amount of Class C fly ash to prevent ASR expansion may not be appropriate, due to the effects on other concrete properties.

**5.6.2 Ground granulated iron blast-furnace slag**—Ground granulated blast-furnace slag is a by-product from the manufacturing of iron. Ground granulated blast furnace slag is a finely ground glassy siliceous material formed when molten slag is rapidly cooled and then ground. Slag for use in concrete should conform to ASTM C 989. Three grades of ground slag are specified in ASTM C 989; grades 100 and 120 are recommended for use in controlling ASR expansions (Mid-Atlantic Regional Technical Committee, 1993b). Slag for use in concrete should conform to ASTM C 989. Effective amounts of slag to reduce ASR expansions vary from 25 to 50 percent, or more, by mass of cementitious materials (Malhotra and Fournier, 1995). The alkalis in slag will contribute to the alkalinity of the concrete pore solution (Kawamura and Takemoto, 1984). The alkalis encapsulated in slag are released at a slower rate than those in portland cement, but at a higher rate than those in fly ash.

**5.6.3 Natural pozzolans**—Natural pozzolans include naturally occurring amorphous siliceous material, or material processed to obtain amorphous silica identified as Class N pozzolan in ASTM C 618. In the United States, the use of natural pozzolans has been relatively rare in recent times. Historically, one of the most commonly used natural pozzolans has been volcanic ash. Calcining some siliceous material to temperatures of 1000 °C can produce a pozzolanic material. Some of these include calcined shale, certain pumicites and tuffs, opal, rice husk ash, metakaolin, and diatomaceous earth. Finely pulverized materials containing volcanic glass, opal, kaolinite, and smectite clays, may be used without calcining to produce pozzolanic materials that can be effective in controlling ASR expansion (Mielenz et al., 1950). Recently, calcined kaolinite (metakaolinite) has been shown to be effective in minimizing expansion caused by ASR (Jones et al. 1992).

Natural pozzolans can have significantly variable characteristics, and recommendations for use cannot be made without testing.

**5.6.4 Silica fume**—Silica fume is a very fine powder typically containing 85 to 99 percent amorphous silica by mass. It is a by-product of the silicon and ferro-silicon metal industries. The standard specification for silica fume for use in concrete is ASTM C 1240. Silica fume actively removes alkalis from the pore solution and thereby reduces the pH (Diamond, 1989). There is some concern that at lower amounts, silica fume delays, rather than prevents, the onset of ASR due to possible later regeneration of alkalis in the pore solution. Replacing at least 10 percent of high-alkali cement with silica fume has been sufficient in some cases (Davies and Oberholster, 1987), while using a minimum of 20 percent (Hobbs, 1989) has also been suggested. The higher percentages of silica fume may cause other problems with the concrete (such as cracking) that are unrelated to ASR. In Iceland, concrete containing 5 to 10 percent silica fume has been used successfully since 1979 to control ASR expansions (Olafsson, 1989). Silica fumes with higher amorphous silica and lower total alkali contents are generally more effective (ACI 234R). The commercial form of silica fume can influence its effectiveness in preventing ASR expansion. One study indicates that if densified pellets of silica fume are not effectively dispersed while mixing, they may act like reactive aggregate particles and cause cracking due to ASR (Pettersson, 1992). A study in Iceland reports that better dispersion of silica fume may be achieved by intergrinding it with the cement (Gudmundsson and Olafsson, 1996).

**5.6.5 Blended hydraulic cements**—Use of blended cements, such as ASTM C 595 Type IP, where the fly ash is interground with cement, may be more effective in controlling expansion, presumably due to a greater fineness and better distribution of the fly ash (Farbiarz et al., 1989).

### 5.7—Testing for effectiveness of pozzolans or slags

ASTM C 441 is the test method that evaluates the effectiveness of a pozzolan or slag in reducing expansions due to ASR. In this test, Pyrex glass is used as a standard reactive aggregate. Test mortar bars are prepared with a high-alkali cement or the job cement with 25 percent pozzolan or 50 percent slag by mass. The tested pozzolan or slag qualifies as effective if the mortar-bar expansion meets certain criteria. While this method qualifies the type of pozzolan or slag, it does not establish minimum effective amounts.

ASTM C 311 provides a procedure for evaluating the effectiveness of fly ash or natural pozzolan in reducing ASR expansion that is a modification of ASTM C 441. Mortar bars are made with Pyrex glass. A test mixture is prepared with at least 15 percent fly ash or natural pozzolan by mass of cementitious materials. The admixture is considered effective if the expansion is reduced to the level produced by a control low-alkali cement mixture. This "effective" amount of admixture can then be used in concrete to control ASR with cements having alkali contents that do not exceed by more than 0.05 percent the alkali content of the cement

used in the test mixture. Additional guidance is provided in Appendix XI of ASTM C 311.

Pyrex glass is a very reactive material, and if the pozzolan-cement combination can control its expansion, it should work with natural aggregates. However, some have questioned the use of Pyrex glass since it contains alkalis that may be released into the pore solution and is sensitive to test conditions (Berube and Duchesne, 1992; and Thomas, 1995). The Strategic Highway Research Program (SHRP) and other research (Stark et al., 1993; Davies and Oberholster, 1987) have indicated that the rapid mortar-bar test (ASTM C 1260) may also be able to be used to establish minimum effective amounts of pozzolans or slag. Multiple runs of the test using various amounts of pozzolan or slag are conducted. The effective amount of pozzolan or slag is the amount that reduces the expansion to below a prescribed expansion limit. This approach could potentially qualify cementitious material combinations for use with a particular aggregate. Further evaluation of this approach remains to be done.

A problem cited with deriving conclusions on the effectiveness of pozzolans or slag based on the results of a two-week test, as in ASTM C 441, the procedure in ASTM C 311, or the proposed modification to ASTM C 1260, is the uncertain mechanism that causes a reduction in expansion. Within the test period, the pozzolans or slag are unlikely to react to an extent that replicates the actual mechanism that occurs in field concretes. Appendix B in CSA A23.1 recommends a two-year testing period with the concrete prism test, ASTM C 1293, to evaluate concrete containing fly ash or slag. Research is underway on correlating laboratory tests with the performance of concrete subject to field exposure (Fournier et al., 1995).

### 5.8—Alkali content of concrete

Using a low-alkali cement (less than 0.60 percent alkali as equivalent  $\text{Na}_2\text{O}$ ) does not guarantee that concrete containing reactive aggregates will not produce excessive expansion due to ASR. Increasing the cement content with a low-alkali cement may increase the alkali concentration of the concrete pore solution and may cause deleterious expansions (Johnston, 1986).

Some specifying agencies limit the alkali content of concrete. British specifications limit the alkali content to  $3 \text{ kg/m}^3$  (Concrete Society Working Party, 1987). The alkali contributions from cement, pozzolans, admixtures, some aggregates, and mixing water are considered. For pozzolans, the water-soluble alkalis are used in the calculation. Canadian Standards Association (CSA) A23.1 Appendix B 5.2 does not include the alkali contents of fly ash and slag when specified minimum replacement levels are maintained and the alkali contents of these materials are within the CSA specified limits of 4.5 percent for fly ash and 1 percent for granulated iron blast furnace slag. In South Africa, the limit on the alkali content of concrete varies depending on the type of reactive aggregate (Oberholster, 1983).

### 5.9—Chemical admixtures

McCoy and Caldwell (1951) proposed the use of lithium salts to prevent excessive expansion due to ASR. ACI 212.3R lists salts of lithium (1 percent by mass of cement)

and Hansen (1960) lists salts of barium (2 to 7 percent by mass of cement) to be effective in reducing ASR expansion. Sakaguchi et al. (1989) observed that lithium ion prevents the formation of additional gel. SHRP research recommends that a minimum molar ratio of lithium to sodium (plus potassium) of 0.60:1 is required to prevent ASR (Stark et al., 1993; Stokes, 1996). Other chemicals that have shown some success in laboratory studies include sodium silicofluoride and alkyl alkoxy silane (Ohama et al., 1989). Development of these last two admixtures is in the research stage, and their use in practice is not yet recommended.

Salts of protein materials and some water-reducing set-retarding admixtures are reported by ACI Committee 212 (ACI 212.3R) to have produced moderate reductions in ASR expansion. Salts of chlorides and sulfates can increase expansion. High-range water-reducing admixtures used with opal as the reactive aggregate have resulted in increased expansions (Wang and Gillott, 1989). Several organic compounds have been used to complex or chelate the alkali ions with varying degrees of success. These generally tend to be too expensive for practical applications.

The status of more recent research developments on admixtures for ASR has been reviewed (Mather, 1993). Lithium salts appear to be the most promising admixtures, although still somewhat expensive.

### 5.10—Other methods

Entraining air in concrete has been reported to reduce expansions. An additional 4 percent entrained air (beyond that needed for freeze-thaw protection) resulted in a 40 percent reduction in expansion (Jensen et al., 1984). The gel has been observed to fill air voids that provide relief zones for the expanding gel. Most air-entrained concrete is used for the purpose of resisting deterioration due to cycles of freezing and thawing. ASR gel filling a sufficient number of the air voids could reduce the resistance of the concrete to freezing-thawing. However, this phenomenon has not been reported in field concrete. Use of additional entrained air as a practical solution to ASR expansion has not been attempted in practice.

SHRP research (Stark et al., 1993) evaluated the effect of restraint on ASR expansion. Sufficient triaxial restraint can result in creep that will offset expansion due to ASR. Uniaxial restraint will promote cracking in a direction parallel to the restraint. Stark et al. (1993) also reported that if concrete is allowed to dry, the alkalis are chemically altered, and their recovery into the pore solution upon re-wetting is sufficiently slow that ASR expansion will be reduced. Practical approaches to incorporate these observations need development. Useful guidance is given in a report (Institution of Structural Engineers, 1992) on the effectiveness of reinforcement for controlling expansion due to ASR in concrete.

## CHAPTER 6—METHODS TO EVALUATE POTENTIAL FOR EXPANSIVE ALKALI-SILICA REACTIVITY

### 6.1—Introduction

Several informative papers have been written on methods to evaluate the potential for deleterious ASR. Diamond (1978), Grattan-Bellew (1981b, 1983, 1989), Sims (1981),

Kosmatka and Panarese (1988), and Berube and Fournier (1994) have described various test methods used to evaluate potential ASR of aggregates or cement-aggregate combinations. Standards organizations such as ASTM and CSA supply detailed methodologies for evaluating potential ASR of aggregates, concrete, and cement-aggregate combinations. Work funded by SHRP also provides some significant improvements in understanding alkali-silica reactions and in development of a rapid technique to evaluate potential ASR of aggregates (Stark et al., 1993). Tests continue to be modified and developed in an effort to attain a definitive rapid technique for ASR potential of aggregate.

### 6.2—Field service record

The most reliable means to determine potential ASR susceptibility of an aggregate is by verifying available field service records. Verification can be accomplished for existing sources through inspection of concrete structures, 10 years old or older, that were made with aggregate from the source in question, cements of similar alkali level, and other concrete components, all in similar proportions. Moist, damp, and wet-dry environments would be most conducive to deleterious reactivity; therefore, inspections should be geared toward such structures as wastewater treatment plants, dams, pavements, and bridges. Knowledge of the alkali level of the cement (from project records) used in the inspected concretes would be needed in establishing performance of the aggregate, particularly its performance with a high-alkali cement.

To establish the service record of an aggregate in concrete, the inspector should look for manifestations of distress due to ASR, or a lack thereof. Such manifestations may include pattern or map cracking, displacement or evidence of movement due to expansion, exudation or deposits of alkali-silica gel, and reaction rims around aggregate particles that may be present along spalled or scaled surfaces (Stark, 1991c). During the course of the field inspection, procuring concrete cores or other samples from a structure for petrographic examination to verify the occurrence of deleterious ASR is advisable.

Satisfactory field service of an aggregate may not be a guarantee of future performance, if concrete materials previously used (including aggregate composition, cement composition, as well as concrete mixture components and proportions) have changed. If this is the case, several methods of materials evaluation also should be used to ensure that deleterious ASR does not occur in the planned construction.

Aggregates having no service record should be tested by some of the methods described later in this chapter.

### 6.3—Common tests to evaluate potential alkali-silica reactivity of aggregates

Several tests are commonly used (often in combination) to evaluate whether an aggregate or cement-aggregate combination is potentially deleteriously alkali-reactive. These tests are usually done to pre-screen new aggregate sources before use as concrete aggregate. Additional information on testing is contained in Stark (1994).

**6.3.1 Petrographic examination of aggregate**—Potentially reactive components of an aggregate can be identified and



quantified through petrographic examination when performed by an experienced petrographer. The petrographic examination is generally done according to procedures outlined in ASTM C 295. A petrographic examination can be done on samples from undeveloped quarries (ledge rock or drilled rock core), operating quarries (drilled rock core, processed crushed stone, or manufactured sand), undeveloped sand and gravel deposits (bulk sand and gravel samples from either test pits or drilled test holes), and operating sand and gravel deposits (bulk samples from processed natural sand and gravel stockpiles/process streams).

ASTM C 295 specifically recommends that the petrographer identify and call attention to potentially alkali-silica reactive constituents. The examination, however, cannot predict if potentially reactive materials are indeed deleteriously expansive. Therefore, ASTM C 295 directs the petrographer to recommend appropriate additional tests to determine if the amount of potentially reactive material identified is capable of deleterious expansive reactivity. Thus, a petrographic examination is a useful screening procedure that can be done early in the development and testing of a new aggregate source and as a periodic check of operating deposits to verify consistency of composition. Great care is needed in making a petrographic examination, and in some instances small amounts of micro-crystalline quartz, which may not be visible even in thin section examination may be sufficient to cause expansion. In these cases, the presence of this micro-crystalline quartz can be determined by x-ray diffraction analysis. Chapter 4 of this report further describes the types of potentially alkali-silica reactive rocks and minerals.

**6.3.2 Mortar-bar expansion test**—One of the most commonly used tests to determine whether a cement-aggregate combination is potentially alkali-silica reactive is the mortar-bar expansion test, described in ASTM C 227. The test involves molding mortar bars containing either the fine aggregate or the coarse aggregate (which has been crushed and graded to sizes required by ASTM C 227) in question and either a job cement or a reference cement of known alkali level. Some gneisses and graywackes, which are more slowly expanding will only expand in the mortar-bar test if the alkali content of the cement is boosted by the addition of alkali to a level of 1.25 percent.

The mortar is placed in metal molds to fabricate a set of four mortar bars. After hardening, the four mortar bars are demolded and measured for initial length in a comparator meeting the requirements of ASTM C 490. The specimens are placed over water in containers, and the containers are sealed to maintain 100 percent relative humidity. Maintaining optimum moisture conditions in the storage containers presents a problem. If there is excessive moisture, leaching may reduce the alkali content of the mortar before expansion has surpassed the maximum allowable limits. A high moisture level may give maximum expansion with some types of aggregate (for example, opal, that causes the mortar bars to expand within a few weeks). However, this same moisture level may not be suitable with another aggregate type (for example, graywacke, for which the mortar bars may not start to expand for two or three months). For this reason, mortar

bars made with graywacke, gneiss or other slower reacting aggregates, should be stored in containers over water but without wicks (Rogers and Hooton, 1989).

The containers are stored at 38 °C to accelerate the effects of alkali-silica reaction. Periodically, the specimens are removed and length changes are determined. An average length change (for the four mortar bars) greater than 0.05 percent at three months and greater than 0.10 percent at six months test age is considered by ASTM C 33 to be excessive and indicative of potentially deleterious ASR. Specimens exhibiting expansions greater than 0.05 percent at three months but less than 0.10 percent at six months are not considered to be deleteriously expansive by ASTM C 33.

The distinct advantage of this test is that it is a direct evaluation of a particular cement-aggregate combination, which is somewhat closer to an actual service condition. However, a disadvantage of the test is that the performance of the test mortar may not be the same as the performance of a field concrete containing the same materials. Another difficulty is the six-month test duration requirement. In many cases, construction sequencing does not allow for the long lead time required of the mortar-bar test. Further, some investigators believe six months is not long enough to adequately evaluate some aggregate types (Stark, 1980). When slowly expanding aggregate is being evaluated, the trend of the expansion versus time graph at the end of the test should be considered when making the evaluation. If it is obvious that in time the mortar bars will exceed the 0.10 percent expansion limit, care is needed in the use of such potentially reactive aggregates. For example, a cement with an alkali content of 0.80 percent that meets the requirements of ASTM C 227 (that is, alkali content having  $\text{Na}_2\text{O}_e > 0.60$ ) and does not exceed the deleterious expansion limit during the mortar-bar test may cause expansion and deterioration of concrete in the field.

Despite these and other shortcomings (Diamond, 1978; Grattan-Bellew, 1981b, 1983, 1989; Sims, 1981), this test is considered an accurate indicator of a highly-reactive siliceous aggregate's potential for deleterious reactivity with alkalis in concrete.

**6.3.3 Accelerated mortar-bar expansion test**—There have been recent modifications and improvements to mortar-bar expansion tests that are currently in use, such as ASTM C 1260, which are based on the National Building Research Institute (NBRI) or South African mortar-bar test (Oberholster and Davies, 1986). This test is a modification of ASTM C 227, and aggregate and test specimen preparation are the same. However, the mortar bars are stored in a 1 Normal NaOH solution (to provide an immediate source of sodium and hydroxyl ions to the bars) and maintained at a temperature of 80 °C to accelerate the alkali-silica reaction. Comparator readings are taken over a period of 14 days. The test conditions are more severe than most field service environments.

On the 16th day after casting, if the average expansion is less than or equal to 0.10 percent, the cement-aggregate combination is considered non-reactive. If the average expansion is greater than 0.10 percent but below 0.20 percent, the aggregate may be slowly reactive and additional confirma-

tory tests should be performed. If average expansion exceeds 0.20 percent, the aggregate is considered deleteriously reactive.

After several years of extensive worldwide testing and evaluation of the NBRI method (Davies and Oberholster, 1987; Hooton and Rogers, 1989, 1992; Hooton, 1990; Fournier and Berube, 1991), the test has gained acceptance and has been adopted as ASTM C 1260. Investigators have found that the method can identify slowly reactive rock types that previously could not be detected by other reactivity tests (Stark et al., 1993; Davies and Oberholster, 1986, 1987; Hooton and Rogers, 1989, 1992; Hooton, 1990).

One shortcoming of the ASTM C 1260 test is in the interpretation of test data when the observed expansion falls between 0.10 and 0.20 percent. Work done on known reactive and non-reactive rock types by Stark et al. (1993) suggests modification of the test interpretation criteria for some slowly reactive aggregate types: to eliminate the gray area between 0.10 and 0.20 percent expansion, it has been suggested that expansions greater than 0.08 percent be interpreted as potentially deleterious and expansions of 0.08 percent or less as non-deleterious. These criteria are based on using a 1N NaOH immersion test solution. Stark et al. (1993) also investigated a linear regression equation that relates the concentration of the NaOH solution to cement alkali level for a given water-cement ratio. By adjusting the alkali level of the immersion test solution (prior to the start of the test), they sought to simulate a range of cement alkali levels. When using a lower normality (for example 0.60N) NaOH immersion solution, the test criterion for potentially deleterious reactivity must be progressively adjusted downward. Thus, the test was investigated for its potential to determine a safe alkali level at which a potentially reactive aggregate will not result in deleterious expansions. This option is currently under further investigation, and is not yet a recommended practice.

In general, the accelerated mortar-bar test is quick, reliable, and can characterize the potential reactivity of slowly as well as rapidly reactive rock types.

**6.3.4 Quick chemical method**—The ASTM C 289 quick chemical method determines the potential ASR of an aggregate. For this method, a representative sample of the aggregate in question is crushed to pass a 300  $\mu\text{m}$  sieve and be retained on a 150  $\mu\text{m}$  sieve. The crushed material is immersed in a hot ( $80 \pm 1.0$  °C) 1N NaOH solution and is sealed for 24 hours. The intent is to dissolve soluble silica that is capable of being reactive and to allow it to react with the NaOH solution.

After 24 hours, the solution is filtered and measured for dissolved silica ( $S_c$ ) and a reduction of the original hydroxide ion concentration ( $R_c$ ) due to the reaction. The quantity of dissolved silica and reduction in alkalinity are plotted on a graph presented in ASTM C 289. The graph basically divides plotted points, and hence aggregates, into three categories: aggregates considered innocuous, aggregates considered potentially reactive, and aggregates considered deleterious.

The quick chemical test has been extensively evaluated (Mielenz et al., 1948; Mielenz and Benton, 1958; Chaiken and Halstead, 1958; and Mielenz and Witte, 1988). Correla-

tions exist between the quick chemical test, and expansion of mortar bars made with high-alkali cement, petrographic examination of aggregates, and field performance. These correlations provided the basis for establishing the divisions on the graph used in ASTM C 289 to classify an aggregate with respect to potential for reactivity.

The test, although extensively studied, has limitations. For example, the category "aggregate considered potentially reactive" is vague. ASTM C 289 recommends that a series of mortar bar tests be done to determine the "innocuous character" of an aggregate that falls into this category, or that proof of innocuous character be established by satisfactory service history.

Operator techniques and judgement can influence the results of the quick-chemical method. Grinding the sample too fine can lead to erroneous results. Aggregates containing certain rocks such as carbonates, sulfates, and some magnesium silicates such as serpentine can yield erroneous results. Because of these limitations, ASTM C 289 recommends correlation of results obtained by the quick chemical test with other reactivity tests, such as mortar-bar expansion and petrographic examination.

Now there is less need for the quick chemical method (ASTM C 289) due to the development of other rapid methods such as the accelerated mortar bar method (ASTM C 1260) and autoclave tests, developed in Japan, China, France, and Canada (see Section 6.4.6 of this report). These new test methods are more reliable than the quick chemical method, and are possibly less susceptible to operator error. The quick chemical test has been shown to work well detecting rapidly expanding aggregates such as opal, chert, and volcanic glass. However, this test is more problematic when used to evaluate slowly expanding aggregates such as graywackes, gneisses, and argillites. The danger with the use of the quick chemical method as a screening test for aggregates is its possible failure to detect some potentially reactive aggregates.

A modification of the quick chemical test developed in France is reported to permit distinguishing between those aggregates that behave normally and those that exhibit the pessimum effect in the mortar-bar test (Sorrentino et al., 1992). However, evaluation of this proposed modified quick chemical test has been limited.

**6.3.5 Concrete prism test**—The concrete prism test is similar to the mortar bar expansion test, except that it uses concrete prisms rather than mortar bars, and alkali is added to the fresh concrete mixture to reach a certain level within the concrete. The prism test has been adopted as ASTM C 1293. The concrete prism test is used in Canada as CSA A23.2-14A. A cement content of 420 ( $\pm$ ) 10  $\text{kg}/\text{m}^3$  is used for the test procedure, with a  $w/c$  (water-cement) ratio ranging from 0.42 to 0.45. Sodium hydroxide is added to the mixture water to reach an alkali level 1.25 percent  $\text{Na}_2\text{O}_e$  by mass of cement. Once de-molded, the specimens are stored in a sealed container with a water reservoir in the bottom. The container is kept at 38 °C.

Evaluation of aggregates in concrete instead of in mortar provides a more realistic assessment of the potential for reactivity. The length of time (a year or more) to obtain definitive

results for this test method will reduce its use in cases where results are required quickly.

The concrete-prism method can be used to assess both coarse and fine aggregate. When assessing a coarse aggregate, an innocuous fine aggregate is used; and when assessing a fine aggregate an innocuous coarse aggregate is used. The concrete-prism test is generally better suited to assess coarse aggregate, and in particular, aggregate types that react very slowly with alkalis (those that may pass the mortar-bar test). Length-change measurements are taken periodically to at least a 365-day specimen age.

Criteria for excessive expansions are set forth in Appendix B of CSA A23.1. Expansions of 0.040 percent, or more, at one year are considered to indicate the potential for deleterious reactivity. A note to Table B2 of the CSA document specifies that "in critical structures such as those used for nuclear containment or large dams, a lower expansion limit may be required."

CSA A23.1—Appendix B, Alkali-Aggregate Reaction, suggests continuing the test until expansion has stopped. Grattan-Bellew (1983) recommended performing petrographic examination in conjunction with the concrete-prism test to determine the type of reactivity that may occur (fast reaction or slow reaction). This information helps in determining how long the test should be continued. CSA A23.1—Appendix B 2.4 states that this test can be used to evaluate the effectiveness of supplementary cementing materials on expansion of concrete containing slowly reactive materials. When the test is used for this purpose, care should be taken to prevent alkali leaching, and expansion should be measured for at least two years.

Expansions in the concrete prism test are affected by the storage environment. In particular, maintaining adequate humidity in the containers is of paramount importance.

#### **6.4—Less common tests to evaluate potential alkali-silica reactivity of aggregates**

Several standardized and non-standardized test methods (that are not as commonly used as the methods previously described) are available to evaluate potential ASR of aggregates. These tests are generally used to pre-screen the potential ASR of new aggregate sources or used for research purposes.

**6.4.1 Potential volume change of cement-aggregate combinations**—This test method, is described in ASTM C 342, is essentially a modified mortar-bar expansion test to determine potential expansion of a particular cement-aggregate combination due to a number of mechanisms, including ASR. It has been used primarily for research on aggregates in Oklahoma, Kansas, Nebraska, and Iowa (Conrow, 1952 and ASTM, 1954).

The test involves subjecting mortar bars to varying moisture and temperature storage conditions, and periodically measuring length change for up to one year. ASTM C 342 states that no acceptance limits for expansion have been established for this procedure. However, ASTM subcommittee work has indicated that expansion equal to or greater than 0.020 percent after one year may indicate unacceptable expansions due to alkali-silica reaction (ASTM 1954).

Disadvantages of the test are: a lack of correlation for aggregates outside the Plains states, duration of the test, and labor requirements necessary to maintain the specimens in various storage conditions.

**6.4.2 U.S. Army Corps of Engineers modified mortar bar test**—This is a mortar-bar expansion test incorporating petrography as a means of selecting particular aggregates for evaluation (Buck and K. Mather, 1984). The test can be used to identify slowly reactive rock types, such as quartzite. If a petrographer identifies siliceous rock types that contain more than 20 percent strained quartz having an undulatory extinction angle (an optical property indicating deformed crystals) greater than 15 degrees during a petrographic examination, then the aggregate is considered potentially reactive and is to be tested by the modified mortar bar test. For coarse aggregate evaluation, mortars using standard sand (as specified in ASTM C 778) are made according to ASTM C 227. Five of the potentially reactive coarse aggregate particles passing the 19.0 mm sieve but retained on the 12.5 mm sieve (and identified by petrographic means to fit the above criteria) are embedded in each fresh mortar bar. After hardening, the mortar bars are stored over water in sealed containers and maintained at 60 °C. Length change measurements are taken periodically for a period of one year.

If fine aggregate is to be evaluated, it is used in the mortar rather than the standard sand. Storage of fine aggregate specimens is the same as for the coarse aggregate. An aggregate is considered potentially reactive by this method when expansion is 0.025 percent or greater in six months or 0.040 percent in twelve months.

The drawbacks to this test are the same as for ASTM C 227. However, the Army Corps of Engineers test appears to identify slowly reactive rock types that are not detected by ASTM C 227. This method has been incorporated in the Army Corps of Engineers Standard Practice for Concrete for Civil Works Structures (U.S. Army Corps of Engineers, 1994).

**6.4.3 Concrete cube test**—The concrete cube test, also known as the "Dahms cube test," was first developed and used in Germany (Bonzel and Dahms, 1973 and Dahms, 1977). The cube test is a qualitative method whereby 300 mm concrete cubes made with test aggregate are periodically inspected for cracking and gel exudations due to ASR. The cubes are stored in a moist room ( $\geq 95$  percent Rh) maintained at 40 °C. An alternative method is to cast 100 mm cubes and store them at 65 percent relative humidity and 20 °C, with partial immersion in water. British and South African investigators use variations of this test (Grattan-Bellew 1983). The cubes are examined visually at intervals to identify cracking and gel exudation due to ASR.

A major advantage of the concrete cube test is the qualitative nature of the method. An accurate length change apparatus is not necessary, so that aggregate and concrete producers could easily perform the test in-house. However, as with most other reactivity tests, the length of time for visible cracking due to ASR could be quite long. The test also does not quantify whether the reaction is indeed deleterious.

**6.4.4 Nordtest accelerated alkali-silica reactivity test**—This method was first developed by Chatterji (1978) and has



become Nordtest Building Method 295. The test is one of several modified, accelerated mortar-bar expansion tests. The method is intended to identify reactive fine aggregate that is problematic in Denmark. Triplicate mortar prisms having prescribed sizes, but slightly larger end area (or cross section) and shorter length than ASTM C 227 mortar bars, are cast using the test aggregate, a cement-sand proportion of 1:3, and water-cement ratio of 0.5. The molded specimens are moist cured for 24 hours.

After hardening and demolding, the specimens are cured by water immersion for 4 to 5 weeks. After curing, comparator readings are taken and the test specimens are immersed in a saturated sodium chloride (NaCl) solution maintained at 50 °C. Comparator readings are taken periodically for a period of eight weeks or more.

The method has been shown to detect ASR for Danish aggregates (Chatterji, 1978; Jensen et al., 1982). The test is noteworthy because of the shortened testing time compared with ASTM C 227 method. However, its use is not widespread, and experience with aggregates outside of Denmark is not known.

**6.4.5 Gel pat test**—The gel pat test was originated in England as a qualitative method to characterize ASR (Sims, 1981; National Building Studies, 1958). The smooth, sawn surface of a mortar specimen containing the test aggregate is immersed in alkali solution for a period of three days. If the aggregate is reactive, gel forms along the reacted particles, and the percentage of reactive constituents can be estimated.

The test can be a simple means of evaluating an aggregate's potential for ASR (Fournier and Berube, 1993), despite its lack of quantitative measurements.

**6.4.6 Autoclave mortar bar test**—Fournier et al. (1991) developed a rapid autoclave test for mortar bars that provides results in a few days. The mortar bars are made according to ASTM C 227 specifications, except that a fixed *w/c* of 0.5 is maintained, and the alkali content of the mortar is raised to 3.5 percent by the addition of NaOH to the mixing water. The bars are stored at 100 percent relative humidity at 23 °C for two days before autoclave treatment. The autoclave procedure follows ASTM C 151, except that the steam curing is five hours at 130 °C and 0.17 MPa. Only two length measurements are taken, one after two days of moist curing and the second when the bars have been cooled to 23 °C after autoclave treatment.

Good correlation has been found between expansions in the autoclave test and ASTM C 1260 (Berube et al., 1992; Salomon and Gallius, 1992). Correlation of autoclave expansions with the results of concrete prism tests was poorer, but it is not known if this is due to inadequacies of the concrete prism test rather than the autoclave test. The proposed autoclave test has not been standardized, although a similar test method is in use in Japan.

## 6.5—Tests to evaluate alkali-silica reactivity in hardened concrete

Two test procedures are used on hardened concrete to evaluate ASR. One method is used to identify in-situ ASR products in hardened concrete. The second method is used to evaluate the potential for continued expansion in a hardened concrete.

**6.5.1 Uranyl acetate gel fluorescence test**—A method to identify in-situ ASR products in hardened concrete for field and laboratory use has been developed (Natesaiyer and Hover, 1988, 1989). This rapid procedure involves treating a freshly fractured surface of hardened concrete with uranyl acetate solution and observing fluorescence under ultraviolet light. ASR products will fluoresce under ultraviolet light when treated with the uranyl acetate solution.

The method has been successfully used on field concretes (Stark, 1991c) and is included in an annex to ASTM C 856. The method has some limitations and should be used by trained specialists or done in conjunction with confirmatory methods, such as petrographic examination.

**6.5.2 Methods of evaluation of the state of alkali-silica reactivity in hardened concrete**—Stark (1985, 1991b) developed methods to determine whether the potential exists for expansive ASR in hardened concrete. Since ASR requires moisture, available alkalis, and reactive silica for expansion to take place, these variables are evaluated in hardened concrete.

Moisture availability can be measured through relative humidity measurement of concrete powders drilled from affected structures or in-place through the use of commercially available, portable capacitive relative humidity/temperature meters. Stark (1985) has established that a relative humidity of at least 80 percent, referenced to 21 to 24 °C, is necessary to sustain expansion due to ASR.

To ascertain whether alkalis and reactive silica are still available in hardened concrete, nominal 100-mm diameter cores are drilled from the structure in question and subjected to length change measurements under a variety of test conditions. Three companion cores are tested under the following conditions: one core stored continuously in water maintained at 38 °C, one core stored continuously in 1N NaOH solution maintained at 38 °C, and one core stored continuously over water in a sealed container maintained at 38 °C.

The core stored in water serves as a baseline for evaluating expansion developed by cores stored over water or in NaOH solution. The cores are fitted with gage pins and length change measurements are taken periodically for up to 12 months.

Whether or not potentially reactive but as yet unreacted silica is present in the aggregate in concrete is determined by storing the core in a NaOH solution. This storage could force expansive reaction to occur by providing additional alkali at elevated temperatures. If expansive reactivity develops, length changes develop steadily with time. Since testing is initially done using partially-dried concrete, early rapid expansion occurs due to uptake of moisture. This early expansion is subtracted from length increases due to further expansive ASR in cores stored in NaOH solution. This correction is applied using expansions of the cores stored in water, which normally terminates when equilibrium mass is reached, usually within about 7 to 14 days. Occasionally very slow expansion occurs in cores stored in water over extended periods of time. This is easily distinguished from early high rate of expansion due to uptake of moisture. The slow expansion is interpreted as evidence of swelling of pre-existing alkali-silica gel.

In determining that additional expansive ASR has occurred in the core stored in NaOH solution, expansion must exceed that of the companion core stored in water by at least 0.03 percentage points after equilibrium mass is reached by the core stored in water. The greater this difference, the greater the potential for further expansion, provided sufficient alkali is available. If differences in expansion between cores stored in water and NaOH solution is less than 0.03 percentage points, then sufficient unreacted but potentially reactive silica is not present in sufficient quantities to produce deleterious ASR.

The residual expansion of aggregate in concrete has also been estimated by extracting the coarse aggregate from cores by freezing and thawing, and then testing the reclaimed aggregate in the concrete prism test (Grattan-Bellew 1995). This approach will work only if the original concrete is not durable in freeze-thaw testing.

The expansion of cores stored over water at 38 °C provides an indication of whether available alkali and reactive silica are still present in the concrete. Expansion under this condition develops more slowly than in cores stored in NaOH solution. If no expansion occurs after the initial uptake of moisture, but high expansion levels are reached by cores stored in NaOH solution, insufficient alkali is available in the concrete for further expansive ASR.

The main limitations to this method of testing are: length of time to obtain test results, intensive use of several pieces of equipment and labor, and cost. However, these methods have been used successfully in testing field structures, and ultimately the results were used to formulate the repair schemes (Stark, 1985, 1991b).

## 6.6—Summary of testing

In the past fifteen years significant efforts have been made toward developing new or improved rapid test methods to evaluate the potential for ASR. This chapter highlights many of the methods in use and presented at the Sixth, Seventh, Eighth, Ninth and Tenth International Conferences on Alkali-Aggregate Reaction. Specific papers from the proceedings of these conferences are cited throughout this chapter.

## CHAPTER 7—MANIFESTATIONS OF DISTRESS DUE TO ALKALI-CARBONATE REACTIVITY

### 7.1—Overview

Expansive distress in field concrete due to alkali-carbonate reactivity (ACR) was first observed by Swenson (1957) in Ontario. The indications of this distress are the same as those resulting from ASR, and therefore, until the reactive aggregate is identified, the distress may initially be attributed to the more general category of AAR.

### 7.2—Field indicators

The most common indication of this type of distress is map or pattern cracking, so called because of the more or less uniform spacing of the cracks over the entire concrete surface (Fig. 8). From structure to structure, the spacing of the cracks may vary, depending on the extent of the reaction, environmental conditions, and other factors. As the reaction progresses, crack spacing usually decreases as additional

cracks develop between the initial cracks. With continued degradation, spalling of the surface begins at the cracks, allowing further moisture ingress and accelerated breakdown of the concrete.

Several other types of cracking may be confused with the cracking due to ACR. So called D-cracking is also an aggregate-related cracking, but is due to the use of aggregate not resistant to freeze-thaw cycling in saturated concrete. D-cracking develops first near and parallel to the joints, cracks, or free edges of the concrete, and progresses inward from these areas. D-cracking does not divide the surface into equidimensional blocks but rather into elongated pieces with the long dimension parallel to the edge. An additional distinction is that D-cracking typically occurs only in concrete slabs-on-grade.

Another type of cracking that has a surface manifestation similar to AAR is plastic-shrinkage cracking. It is not surprising that the two types of cracks present a similar appearance, since they are both a result of tensile forces at the surface of the concrete. Plastic-shrinkage cracking results from rapid drying of the freshly-placed concrete surface due to high temperature, low humidity, and wind. Plastic-shrinkage cracking is generally noticed soon after construction (a few hours to a few days). In contrast, cracking due to AAR will take much longer time (usually months, to years) to develop. Plastic-shrinkage cracks are usually open at the surface and quickly wedge down to no opening within one to two inches of the surface. Since these cracks occurred when the concrete was plastic, often "tails" of paste can be seen spanning the open portion of the cracks.

General indicators of expansion due to the ACR include closed joints, blow-ups or crushed concrete, and relative offsets of adjacent slabs or substructures (Stark, 1991c). The amount of concrete expansion can be estimated in a number of ways, but usually is done by comparison with some reference structure or feature that has remained stable. For pavements, the reference may be an adjacent unaffected slab or shoulder, or the total length reduction of an expansion joint at the approach to a bridge. For bridges, decks may be compared with railings or substructures. Various parts of a dam may also be compared or height changes determined, if survey data are available.

### 7.3—Microscopic indicators

A number of features visible under the microscope are indicative of alkali-carbonate-related distress. Although often not conclusive individually, when several of these features are present together, distress due to ACR can confidently be diagnosed.

Unlike ASR, there is no gel product formation related to ACR. Rather, the process involves dedolomitization (decomposition) of the dolomite present in the aggregate to form brucite and calcite. Unique evidence of this reaction, in the form of magnesium hydroxide ( $Mg(OH)_2$ , brucite) or more complex magnesium silicates should therefore be sought. Brucite is not easily seen, so its apparent absence should not be taken as evidence of the absence of ACR. Reaction rims may occur, as well as microcracking within

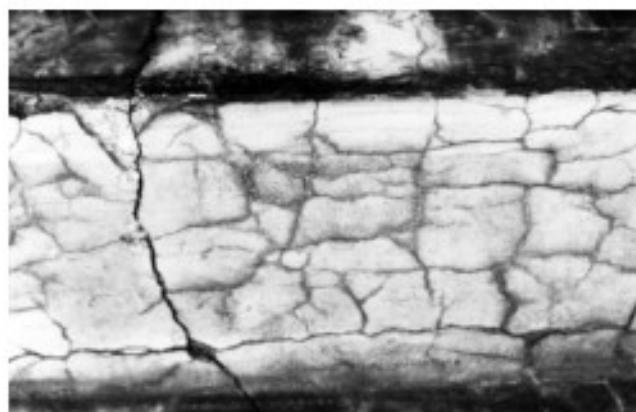


Fig. 8—Photograph of typical surface map-cracking due to ACR.

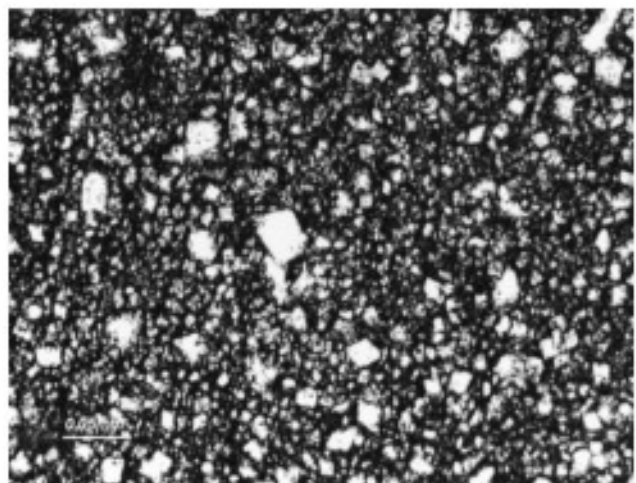


Fig. 9—Characteristic texture of alkali-carbonate reaction susceptible rock. Shown are dolomitic rhombs in a fine-grained matrix.

the aggregate particles and in the paste between the aggregate particles. Cracking within the aggregate particles may be peripheral or random. Rims on the aggregate are not positive indicators of deleterious reactivity.

#### 7.4—Role of environment, structure geometry, and restraint on distress development

A number of factors can influence the occurrence of ACR, and the severity of associated distress. Since the distress mechanism is not yet completely defined, the magnitude of these various influences is also somewhat uncertain. The environment influences the reaction and the distress developed in concrete through the amount of water made available to the concrete, and locations within the structure where that moisture is concentrated. Water is needed to provide the sodium and potassium hydroxides necessary for the dedolomitization (see Section 8.3) reaction, and also for absorption by the clay minerals exposed (by dedolomitization), thus creating the expansive forces. The cracking actually occurs in those zones of the concrete where expansion does not occur (normally, exposed surfaces that can dry out), since

those zones are put in tension by the adjacent expansion. Therefore, the rate and depth of drying of the concrete at the surface also influences distress development.

In addition to moisture, temperature plays a role in the reaction. At least early in the reaction process, higher temperatures increase the rate of reaction and presumably the subsequent expansion.

Finally, the environment can be the source of additional alkalis, resulting in distress even if the concrete was made with low-alkali cement. The ACR apparently may occur at alkali levels in the cement as low as 0.40 percent  $\text{Na}_2\text{O}_e$  (Dolar-Mantuani, 1983).

Geometry of the structure can influence the distress as it affects the ratio of exposed surface area (where distress occurs) to the volume of the concrete where expansion is occurring. In some structures (pavements, for example), the expansion itself may not be as damaging as the surface distress, while in other structures (dams, for example) the expansions may be of more concern.

Finally, in contrast to ASR, alkali-carbonate expansion lessens as the maximum aggregate size is decreased.

## CHAPTER 8—ALKALI-CARBONATE REACTIVITY MECHANISMS

### 8.1—Overview

Carbonate rocks participating in ACR have a particular mineralogical composition. Their fine-grained texture and structure are characterized by relatively larger, rhomb-shaped crystals of dolomite [ $\text{CaMg}(\text{CO}_3)_2$ ] set in a finer-grained matrix of calcite [ $\text{CaCO}_3$ ], clay, and (commonly) silt-sized quartz (Fig. 9). In the characteristic composition, the carbonate-mineral components of the rock consist of substantial amounts of both dolomite and calcite, and the dilute hydrochloric (HCl) acid insoluble residue usually contains a significant amount of clay.

### 8.2—Characteristics of alkali-carbonate reactive rocks

Alkali-carbonate reactive rock types have several defining characteristics.

**8.2.1 Texture and composition**—The rhombic crystals of dolomite occurring in the matrix of the reactive rock may be relatively sparsely distributed and appear to be floating in the background or may be crowded together with dolomite rhombs (equilateral parallelogram shape) adjacent or touching.

Many of the discrete floating dolomite rhombs set in the matrix are sharp-edged euhedral (crystal-shaped) sections, while others may occur with rounded corners and a more irregular, less pronounced, rhombic shape. The typical average size of the rhombs is approximately 25 microns maximum dimension, with the largest being approximately 50 to 75 microns. The smallest rhombic sections may be at the limit of optical resolution, because they are equal or less than the thickness of the concrete slice resulting from the preparation of the thin sections used for examination.

The grain sizes of the calcite and clay matrix (where present) in which the dolomite rhombs are set are typically 2 to 6 microns for the calcite, with the smaller clay particles disseminated throughout. Silt-sized quartz grains are often



also disseminated throughout the matrix where the clay particles are not present, and the matrix is often porous.

The typical texture described above is found in reactive carbonate rocks that have been identified as early expanders by ASTM C 586 (the rock prism test, see Section 10.4). These rocks have rock prism expansions of a few tenths of a percent in a matter of weeks, and field manifestations of expansion within perhaps one year of construction.

A modification of the typical texture is found in reactive carbonate rocks identified as late expanders, which may not show noticeable rock-prism expansions until approximately 25 weeks (rarely before one year) and may not show field manifestations until 5 years in service. In those rocks, the typical feature—dolomite rhombs in a clayey fine-grained matrix—is consistent. The matrix is coarser-grained, and is composed of interlocking dolomite grains together with calcite, clay, and, commonly, silica minerals. The differences between early and late expanders are reflected in the bulk compositions, as shown in Table 1.

Carbonate rocks displaying the characteristic reactive texture may comprise all, or a substantial portion of, a bed in a limestone quarry. Or, the reactive texture may occur as thin layers within a limestone bed that does not otherwise display the reactive texture.

The occurrence of the reactive carbonate rock texture is a reliable diagnostic guide since there appear to be no known exceptions to the observation that all limestones that possess the characteristic texture and composition will react, or dedolomitize (see Section 8.3), in an alkaline environment. Also, all limestones that dedolomitize in an alkaline environment possess the characteristic texture and composition.

Broadly speaking, the geological conditions for the formation of alkali-expansive carbonate rocks are more restricted than for other limestones, and they are not abundant in the volume of the world's limestone. They may occur in carbonate rocks of any age but, thus far, have been found most often in carbonate rocks from the Ordovician age on the geologic time scale.

To put reactive carbonate rocks in perspective with reactive siliceous rocks, the latter are far more abundant in the overall category of siliceous aggregates (crushed stone, sand, and gravel) that might be used in concrete than are reactive carbonate rocks relative to all carbonate rocks. Carbonate rocks can be susceptible to alkali-carbonate, alkali-silica, or both reactions. Some siliceous dolomitic limestones or siliceous dolostones are reported to be susceptible to both reactions (Tang et al., 1996).

Typical compositions of early and late expanding alkali-reactive expansive carbonate rocks are shown in Table 1 (Walker, 1978), with their geographic location and publication reference indicated.

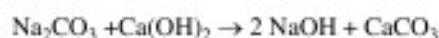
The expansion of concrete containing alkali-reactive carbonate rocks is promoted by large-size aggregate, availability of moisture, high temperature, high alkali content, and for those concretes containing early expanding carbonate rocks, a carbonate component in which the calcite and dolomite are present in approximately equal amounts.

### 8.3—Mechanism of reaction and expansion

Following the first recognition of ACR by Swenson (1957), work by Hadley (1961) and later investigators showed that the main chemical reaction in the rock was that the dolomite [ $\text{CaMg}(\text{CO}_3)_2$ ] decomposed, or dedolomitized, to calcite ( $\text{CaCO}_3$ ) and brucite  $\text{Mg}(\text{OH})_2$ , as represented by the following reaction in which M represents an alkali element, such as potassium, sodium, or lithium



A further reaction that occurs in concrete is that the alkali carbonate produced in the initial reaction may react with the  $\text{Ca}(\text{OH})_2$ , produced as a normal product of cement hydration, to regenerate alkali hydroxide. For example



The starting and end products of the reaction have been well documented, and they establish the overall bulk chemical and mineralogical changes that take place. However, the dedolomitization reaction does not obviously translate into a mechanical explanation for the expansion, because the reactions proceed as a volume-for-volume replacement without the direct production of some extra volume to generate expansion.

Tang and Lu (1987) proposed a mechanism of alkali-carbonate reaction based on before and after studies of reactive rock in KOH solution. They used a scanning-transmission electron microscope with energy dispersive x-rays to observe the locations of the reaction products in the expanded sample.

They found that the brucite produced by the reaction occurred in a 2-mm-thick ring of parallel-oriented, 2.5 nm crystals (with space between them) surrounding the euhedral dolomite rhombs of the typical reactive texture. The brucite crystals were, in turn, surrounded by the calcite and clay particles of the matrix. The calcite of the dedolomitized rhomb remained within the interior of the dedolomitized crystal.

The authors concluded that the expansion was caused by the combined processes of: (1) the migration of alkali ions and water molecules into the restricted space of the calcite/clay matrix surrounding the dolomite rhomb; (2) migration of those materials in the dolomite rhomb; and (3) the growth and rearrangement of the dedolomitization products, especially brucite, around the outsides of the rhombs. According to this explanation, the presence of clays is not required for expansion to take place.

The rhombs maintain their original dimensions, supported by the remaining calcite framework, but the  $\text{MgCO}_3$  component of the original dolomite rhomb is transformed into the brucite that accumulates in parallel layers around the periphery of the rhomb. The original rhomb, together with the brucite layer, needs more space and generates an expansive force that translates into cracking of the coarse aggregate particle.

Damage due to ACR is produced by cracking initiated within the reactive coarse aggregate particles, and extension of those cracks into the paste of the concrete. With ASR, the first generation, or direct damage to the concrete can also be produced by internal cracking of aggregate particles

**Table 1—Composition of early and late expansive carbonate rocks\***

|   | Acid insoluble residue, percent | Dolomite percent of total carbonate |
|---|---------------------------------|-------------------------------------|
| Kingston, Ontario, Early Expanders (Swenson, 1957; Swenson and Gillott, 1960; Dolan-Mantuani, 1964) | 5 to 15                         | About 50                            |
| Iowa, Illinois, and 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 (Dolan-Mantuani, 1964)  | 21 to 49                        | 75 to 87                            |
| Virginia Late Expander (Newlon, Ozol, and Sherwood, 1972)   | 33                              | > 90                                |

\*adapted from Walker, 1978

extending cracks into the paste. However, it also can be produced by extension of existing cracks and production of new cracks as a consequence of the migration and subsequent expansion of the alkali-silica gel. The gel is initially produced by the reacting particles, and migrates away from the particles and into cracks and voids in the concrete.

Although bulk expansions of concrete produced solely by the ACR may be lower than those produced by severe ASR, the ultimate damage to the concrete can be of the same order as produced by ASR if the concrete is exposed to cyclical freezing and thawing (Ozol and Newlon, 1974). The initial cracks produced by the ACR are exacerbated by saturation of the cracks and subsequent freezing.

## CHAPTER 9—MEASURES TO PREVENT ALKALI-CARBONATE REACTIVITY

### 9.1—Introduction

No known methods exist of adequately preserving existing concrete containing the elements that contribute to ACR chemical reactions. However, there are recommended approaches to prevent ACR in new construction. These approaches are described in the next several sections.

### 9.2—Aggregate selection

The aggregate source should be tested for reactivity by the rock cylinder test, ASTM C 586. The criterion for selection of non-reactive rock is that the expansion of rock cylinders stored at 23 °C, and immersed in 1N sodium hydroxide solution for 84 days is not to exceed 0.20 percent (Smith, 1974). A more conclusive way of identifying non-reactive rock is to measure not greater than 0.30 percent expansion in one year rather than not greater than 0.20 percent in 84 days (Ryell et al., 1974).

If the rock cylinder tests show reactivity, the best preventive measure is to avoid the rock by selective quarrying (Smith, 1964). If it is not economically feasible to avoid the use of reactive material, it should be diluted either naturally or by artificial means until the coarse aggregate contains less than 20 percent of potentially reactive material. If both the coarse and fine aggregates are reactive, then less than 15 percent of the total aggregate should be reactive material (Walker, 1978).

Another way to mitigate the effect of the ACR is to use a smaller aggregate size. Although a decrease in the aggregate

size does not slow the chemical reaction, the expansion appears to be in direct proportion to the diameter of the aggregate particles. Therefore, the smallest feasible size of the reactive aggregate should be used (Walker, 1978).

### 9.3—Cement

As with ASR, ACR may be prevented by the use of a cement with a low-alkali content. However, in the case of ACR, the alkali content must be even less than that typically specified for ASR. With highly reactive carbonate aggregates, the low-alkali cement should be limited to 0.40 percent alkali content (Newlon et al., 1964).

### 9.4—Pozzolans

Unlike the reduction in ASR, pozzolans have little or no effect in reducing by ACR (Swenson and Gillott, 1960). They may have an indirect effect by slowing the rate of the reactivity through reduction of concrete porosity, and thereby reduction in the rate of migration of the pore fluids.

### 9.5—Moisture

Water or moisture is partly involved in at least some of the ACR reactions. Therefore, any practical means of decreasing the exposure of ACR-susceptible concrete to water may extend its useful life.

## CHAPTER 10—METHODS TO EVALUATE POTENTIAL FOR EXPANSIVE ALKALI-CARBONATE REACTIVITY

### 10.1—Introduction

Investigations of concrete deterioration resulting from ACR have provided several techniques and criteria for evaluating the potential of aggregates to participate in this reaction (Swenson and Gillott, 1960; Smith, 1964, 1974; Newlon, Ozol and Sherwood, 1972; Ryell et al., 1974; Rogers, 1987). Through careful study of cases of deterioration, the lithologic characteristics of reactive rocks have been identified, and the cause of deterioration understood.

A series of screening techniques eliminate most non-reactive rocks in steps. The remaining suspected rocks are subjected to a more rigorous evaluation technique. The methodology culminates by testing the aggregate in concrete prisms, the most reliable method for determining its susceptibility to react with alkali and cause expansion of concrete. Concrete prism tests are more time consuming and involved than the other screening tests. Newlon, Ozol and Sherwood (1972), Ryell et al., (1974), and Rogers (1987) offer examples of the evaluation process for potentially reactive carbonate rocks. Additional information on testing is in Chapter 33 of ASTM STP 169 C (1994).

### 10.2—Field service record

The first step in the process is to evaluate the field service record of the aggregate, if one exists. Consideration must be given to the environmental conditions as they may contribute to ACR as well as the type of structure in which the concrete was used. The similarity of conditions and structure to the proposed construction must be evaluated.

Field service records, when available, can provide the direct response to the question other evaluation techniques attempt to answer indirectly: Will concrete containing this aggregate suffer distress from ACR in its intended service environment that will significantly reduce its service life? Unfortunately, this information is rarely complete enough to provide a definitive answer and must therefore be supplemented by other lines of evidence.

**10.2.1 Factors to consider**—In evaluating the performance of an aggregate in concrete with respect to ACR and its field service record, several factors are critical and must be addressed.

**10.2.1.1 Aggregate particle size**—The expansion caused by alkali-carbonate reactive aggregates is related to the particle size of the aggregate, with larger particles causing greater expansions. Consequently, an aggregate that performs poorly as large particles may perform satisfactorily when reduced to smaller particles. Conversely, an aggregate with a good service record when used in small particle sizes may cause deleterious expansion when used in larger particles.

**10.2.1.2 Aggregate proportion**—Expansion is directly related to the amount of reactive aggregate in the mixture. The more reactive the aggregate, the greater the expansion. An effort should be made to determine the amount of questionable aggregate in the concrete when establishing the field service record.

**10.2.1.3 Alkali content**—Expansion increases with increasing alkali content. Therefore, it is useful to ascertain the alkali content of structures inspected to establish the field service record. External sources of alkali, such as deicing salts, also contribute to the reaction and should be noted.

**10.2.1.4 Moisture content**—Moisture is necessary for expansion to occur. The environmental conditions relative to the availability of moisture should be noted as well as design features that promote or inhibit moisture ingress into the concrete.

### 10.3—Petrographic examination

Petrographic examination of the aggregates determines if their lithologic characteristics match those representative of reactive carbonate rocks (see Section 8.2). This examination is an important part of any evaluation of aggregates for concrete, and should be performed by an individual qualified by education and experience. Guidance on performing petrographic examinations is provided in ASTM C 295 and by Dolar-Mantuani (1983). Because of the variability of lithologic characteristics in rock units, the sampling of aggregates for evaluation purposes is quite important and should be performed with care (Walker, 1978). Good sampling procedures are discussed in ASTM C 295 (Newlon, Sherwood, and Ozol, 1972; Rogers, 1987).

The distinctive texture of ACR susceptible rocks is illustrated in Fig. 11 and can readily be identified in thin section (Walker, 1978; Dolar-Mantuani, 1983). When viewed with the unaided eye, the reactive aggregates are usually dark, fine-grained, and exhibit conchoidal fracture (Walker, 1978).

### 10.4—Rock cylinder test

This test is based on work by Hadley (1964) and indicates whether a rock will expand when exposed to an alkaline solution. As described in ASTM C 586, the test provides a relatively rapid indication of potential expansive reactivity and is an effective tool for screening aggregate sources. It does not, however, directly predict the expansion that will occur in concrete and should not be used as an acceptance test.

Cylinders or prisms of suspected reactive rock are immersed in 1N sodium hydroxide solution and measured for length change at specified ages. ASTM C 586 recommends the test specimens should be examined petrographically to be sure they are representative of the lithologic characteristics of the rock being evaluated. The problems of specimen selection that arise from the natural variability of rock characteristics are discussed by Walker (1978). Appropriate sampling techniques are described in Newlon, Sherwood, and Ozol (1972), ASTM C 295, and Rogers (1987).

ASTM C 586 interprets rock cylinder expansion of 0.10 percent, or more, at 28 days to be considered indicative of expansive reactivity. Some rocks have been found to initially contract before expanding at later ages (Dolar-Mantuani, 1964), and an alternate expansion limit of 0.20 percent at 16 weeks has been suggested (Newlon, Sherwood, and Ozol, 1972; Ryell et al., 1974; Rogers, 1987). There is some suspicion that the later-expanding rocks may not cause deleterious expansion of concrete (Swenson and Gillott, 1960; Newlon, Sherwood and Ozol, 1972) but the potential of these rocks to cause deterioration should not be totally disregarded (Walker, 1978). Rocks that expand in this test should be subjected to testing in concrete prisms to evaluate their potential for causing expansion.

### 10.5—Concrete prism tests

The expansion of concrete prisms has long been used in investigations of expansive-reactive carbonate rocks. Through the work of many investigators (Swenson and Gillott, 1960; Smith, 1964; Smith, 1974; Newlon, Ozol and Sherwood, 1972; Ryell et al., 1974; Buck, 1975; Rogers, 1987; Rogers and Hooton, 1992), the criteria used in this test for recognizing rock that will expand in concrete has been refined. It is generally recognized that this test provides the best indication of potential for deleterious expansion of a carbonate aggregate in concrete. Versions of the test have been standardized for some time by CSA A 23.2-14A and ASTM C 1105. The two methods use a similar size of test specimen. ASTM C 1105 specifies moist storage at 23 °C while CSA A 23.2-14A specifies moist storage at 38 °C. Work by Rogers and Hooton (1992) indicates that storage at 38 °C gives a better prediction of the long-term expansion of field concrete. The two methods also differ in how they approach the issues of alkali content of the mixture and the aggregate gradation. Because these two factors influence the amount of expansion that results from the ACR, care must be taken in evaluating and comparing the results and criteria of the two methods.

CSA A 23.2-14A uses a fixed grading of aggregate and specifies that the cement have an alkali content ( $\text{Na}_2\text{O}_e$ ) of 0.90 percent by mass. In addition, sodium hydroxide solution



must be added to the concrete mixture to attain an  $\text{Na}_2\text{O}_e$  alkali content of 1.25 percent, by mass of cement. The cement of the highest alkali content expected is used. The CSA criterion for potential deleterious reactivity is an expansion greater than 0.04 percent at one year. ASTM C 1293 is similar to this method, and although specifically adopted for evaluating aggregates for ASR, could be used to evaluate rocks for ACR as well.

ASTM C 1105 specifies the use of the aggregate grading proposed for the job with a maximum size of 19.0 mm. When the proposed mixture contains larger size aggregates suspected of reactivity, they are crushed to pass the 19.0-mm sieve. The cement used in the test is that proposed for the project. Reducing the size of aggregate for the test may obscure an expansive tendency of the larger particle sizes. Consequently, it is more appropriate to test the larger sizes, increasing the size of the test specimens if necessary, or limit the maximum size of the aggregate in field use to that of available test results. The suggested criteria for potential deleterious expansion, based on work by Newlon, Ozol and Sherwood (1972) and Buck (1975), are 0.015 percent at three months, 0.025 percent at six months, and 0.030 percent at one year. Limits for expansion in this test have not been adopted in ASTM C 33.

The suggested limits for expansion in concrete prisms in these tests have resulted from comparison of laboratory studies and concrete performance in the field. As with any type of acceptance limit, the biggest problem arises in how to assess those results falling near the limit. In borderline situations, consideration should be given to the relationship between the conditions of test, the conditions existing in field studies (if available), and the conditions of the intended service. Other aspects to consider are the economics of the aggregate choice, and how cracking might affect the service life of the concrete.

These two methods (CSA A 23.2-14A and ASTM C 1105) reflect different approaches to the problem of evaluating the potential for deleterious expansion. The CSA approach and that of Rogers (1987) is the more conservative, attempting to simulate in the test the most severe conditions the aggregate is likely to encounter in concrete, to minimize the possibility of failure to detect expansive-reactive rocks. It relieves the engineer/specifier of a certain amount of responsibility in making judgements about the materials to be used. The ASTM method takes a more pragmatic approach in testing the materials proposed for the job. In doing so, it places a great deal of responsibility on the engineer/specifier to understand the causes of alkali-carbonate reactivity, the materials, and their likely interaction in the test environment and the service environment.

### 10.6—Other procedures

Rogers (1987) has proposed a quick-chemical screening test based on the finding of researchers that most expansive rocks have a characteristic chemical composition with respect to  $\text{CaO}$ ,  $\text{MgO}$ , and clay content. By plotting the  $\text{CaO}:\text{MgO}$  ratio versus either the insoluble residue or  $\text{Al}_2\text{O}_3$  content, potentially expansive-reactive rocks are found to

fall into a distinct field and thus can be quickly separated from non-reactive rocks. This method (CSA A23.2-26A) is included in CSA A 23.2.

Gillott (1963) developed a method that measures the expansion of powdered rock in sodium-hydroxide solution. This method has not been used extensively due in part to concern about the changing of particle size and texture of the rock, which are known to have an effect on expansion, and the somewhat elaborate nature of the instrumentation needed (Dolar-Mantuani, 1983).

Tang and Lu (1987) showed that significant expansions due to ACR could be obtained in a modified rock prism test, in which 8 by 15 mm prisms were stored in 1N KOH at 80 °C. Expansions of up to 1 percent were obtained in 10 days, using the highly reactive aggregate from Kingston, Ontario.

### 10.7—Evaluation of new aggregate sources

When new areas or lifts of quarries are opened, the field service record of the established material should not solely be relied upon without determining whether the new lithologies are of similar character. When assessing the information provided by the field record, it is important to establish that the aggregates in the concrete are of the same lithologic character as the aggregates whose potential performance is being evaluated. Tests explained in this chapter are sufficient to aid the experienced concrete technologist in performing such an evaluation.

## CHAPTER 11—EVALUATION AND REPAIR OF STRUCTURES AFFECTED BY ALKALI-AGGREGATE REACTIVITY

### 11.1—Introduction

Many structures and pavements investigated at some period after construction have been found to be affected by ASR or ACR. In some cases, petrographic examination of extracted samples has shown the presence of reactivity but no microcracking or distress within the cement paste matrix surrounding the reactive particles. In other cases, serious microcracking of the particles and surrounding cement-paste matrix and exudation of expansive gel are apparent. In the latter cases, the surface of the concrete will most likely be seriously cracked in a random pattern from internal growth. In addition, the strength and modulus of elasticity within the concrete may have decreased. Investigators have found that the decrease in compressive strength is generally slight. Tensile strength tends to decrease more rapidly and to a greater extent than compressive strength.

Each situation where reactivity occurs or is suspect must be evaluated independently due to the original cement-aggregate combination used in the work, other type of structure or pavement involved, the environment, and potential problems that may occur as the concrete expands or grows. The most important consideration in establishing a repair and maintenance program is that reactivity requires moisture. Therefore, those methods that restrict or prevent moisture from entering the concrete will have the greatest success in extending the service life of that concrete.

## 11.2—Evaluation

Once AAR has been detected in a structure or pavement, the areas of affected concrete should be identified and the existing level of strength of the concrete should be established. The initial investigation will provide a basis for comparison with future study or continued monitoring. Guidance in making such investigations is given in ACI 201.1R, ACI 201.3R, and ACI 207.3R. Most investigators use nondestructive techniques (see ACI 228.1R) to establish concrete strength since the volume of concrete involved can be quite large. This work can be done quickly without damaging the structure. The results of nondestructive methods are usually correlated with results of tests on drilled core samples and sawed beams. This correlation should include cores from unaffected concrete from a dry part of the structure for comparison, if possible. Recently, test methods have been developed that can provide a reasonable prediction of the amount of expansion remaining within the concrete (Stark, 1985, 1991b; also see Section 6.5.2).

A program of periodic monitoring should be established. Affected structures or pavements are commonly examined on a yearly basis, at least initially, since the early age progress of reactivity distress is slow. Where results of yearly monitoring show little change, the frequency of monitoring can be decreased. In critical structural elements, it may be prudent to determine existing stress conditions by the over-coring technique. The installation of instrumentation such as stress meters, extensometers, and crack monitors can be useful in monitoring change.

Based on the results of periodic monitoring, the degree of effect of reactivity on various parts of the structure or pavement should be determined. For structural concrete, a structural analysis should be made with design strength parameters to evaluate structural integrity. Finite-element analysis is frequently used for this purpose since it enables the level of strength to be varied within each element (Moranville-Regourd, 1996). A cost analysis should then be made to determine whether to maintain and repair the concrete, or to replace it based on its predicted service life (Oberholster, 1996). The extent of deterioration and loss of strength will determine the most economical approach. Failure to provide at least some maintenance will usually result in an acceleration of surface deterioration from other durability factors such as freezing and thawing of moisture within cracked concrete (Thompson et al., 1995; Cavalcanti, 1987).

## 11.3—Repair methods and materials

Distress in a structure or pavement that has been affected by AAR may be classified as structural or non-structural in nature. Structural repair will most likely require considerably more study and analysis to ensure adequate safety.

**11.3.1 Structural distress**—Structural distress may be manifested in excessive loss of concrete strength, distortion of structural members, or loss of structural continuity due to cracking. Expansion of the concrete may also produce excessive pressure at joints or spalling of concrete along joints.

Repair of structural distress generally involves a combination of methods. Where the level of strength is adequate, repair

may be possible by removal and replacement of any deteriorated concrete, epoxy resin injection of cracks, and application of a moisture-repellent coating. Where moderate loss of strength occurs, it may be necessary to strengthen the members with additional concrete.

Prestressing within added concrete has been successfully used to repair bridge-column pile caps in South Africa (Vanderstraeten, 1986). However, where the concrete volume is more massive, the force generated by expansion may exceed that which can be restrained by prestressing. Concrete dams that are affected by ASR and ACR are good examples. The pressure generated by expansion may be as much as 4.0 MPa. When prestressing is used through reactive concrete, the tendons should be unbonded. Tendons should be run through pipes of polyvinyl chloride or other suitable material. Experience has shown that continued expansion from ASR and ACR may cause cracking in concrete surrounding bonded tendons, causing a complete loss of prestressing with time.

Where the pressure from expansion is great enough to cause spalling along joints or distortion of members, pressure relief can be obtained by cutting slots (Thompson, 1992). If expansion is continuing, additional slotting may be necessary.

Some research has shown that curing mortar bars of reactive aggregate in a saturated carbon dioxide atmosphere will stop expansion. In Brazil, carbon dioxide has been injected into mass concrete through drill holes with some success in inhibiting reactivity (International Committee on Large Dams, 1991).

Additional information on repair of cracks in structural concrete can be found in ACI 224.1R and ACI 503R. The effectiveness and longevity of the repairs will be variable, due to such factors as rate of continued expansion.

Structural distress in pavements may cause a significant loss of service life. Pavement deterioration from ASR and ACR has been reported in the U.S. and elsewhere. Expansion from AAR produces cracks and loss of slab stiffness.

Cracking in reinforced pavements has also been reported as a contributor to corrosion of the reinforcement. Cracking has been a problem at joints and has caused loss of load transfer. Studies have shown that reconstruction is not a practical approach to damaged pavement since these pavements retain some structural value. Extensive work in South Africa has shown that pavement overlay has been the most cost effective solution (Van der Walt et al., 1981). Overlay types include unbonded jointed concrete overlay, unbonded continuously reinforced concrete overlay, thick asphalt overlay, and crushed stone base with thin asphalt overlay.

Where the pavement is still structurally adequate, maintenance and rehabilitation measures may include undersealing where voids exist beneath the slab, joint and crack repair, joint and crack sealing, improvement of drainage, and improvement of load transfer. Information on repair of pavements and joints can be found in ACI 504R and several bulletins by the Portland Cement Association (1976a, 1976b, and 1981).

**11.3.2 Non-structural distress**—Non-structural distress in concrete damaged by ASR and ACR is mostly of cosmetic nature. Repair may include removal and replacement of only severely damaged concrete and application of a protective

moisture-repellent coating. The method selected will depend on the width of cracking. Hairline cracks can generally be coated if the coating can bridge the narrow cracks. Wider cracks will require sealing with epoxy before coating. Some success in maintaining good surface appearance has been achieved with latex-modified cement-based coatings. Information on surface preparation (including the need for drying) and application of moistureproof coatings is contained in ACI 515.1R.

Where appearance is not a concern, excellent moisture protection and reduced expansion have been obtained by treating the concrete surface with a solvent-based water repellent silane (Imai et al., 1987; Berube et al., 1996). A number of field trials sponsored by the SHRP and the Federal Highway Administration are currently underway in the United States to evaluate the beneficial effects of lithium in mitigating ASR deterioration in pavements. Lithium has also been used as part of an electrochemical process to remove chloride from concrete. Lithium was used in the electrolyte to prevent any worsening or development of expansion from the reaction (Stokes, 1996).

## CHAPTER 12—REFERENCES

### 12.1—Referenced standards and reports

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designation.

#### *American Concrete Institute*

- 201.1R Guide for Making a Condition Survey of Concrete in Service
- 201.3R Guide for Making a Condition Survey of Concrete Pavements
- 207.3R Practices for Evaluation of Concrete in Existing Massive Structure Structures for Service Conditions
- 212.3R Chemical Admixtures for Concrete
- 224.1R Causes, Evaluation, and Repair of Cracks in Concrete Structures
- 228.1R In-Place Methods to Estimate Concrete Strength
- 234R Guide for the Use of Silica Fume in Concrete
- 503R Use of Epoxy Compounds with Concrete
- 504R Guide to Sealing Joints in Concrete Structures
- 515.1R A Guide to the Use of Waterproofing, Damp-proofing, Protective, and Decorative Barrier Systems for Concrete

#### *ASTM International*

- C 33 Standard Specification for Concrete Aggregates
- C 150 Standard Specification for Portland Cement
- C 151 Standard Test Method for Autoclave Expansion of Portland Cement
- C 227 Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)
- C 289 Standard Test Method for Potential Reactivity of Aggregates (Chemical)
- C 294 Standard Descriptive Nomenclature for Constituents of Natural Mineral Aggregates

- C 295 Standard Guide for Petrographic Examination of Aggregates for Concrete
- C 311 Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use as a Mineral Admixture in Portland-Cement Concrete
- C 342 Standard Test Method for Potential Volume Change of Cement-Aggregate Combinations
- C 441 Standard Test Method for Effectiveness of Mineral Admixtures or Ground Blast-Furnace Slag in Preventing Excessive Expansion on Concrete Due to Alkali-Silica Reaction
- C 490 Standard Practice for Use of Apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar, and Concrete
- C 586 Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock-Cylinder Method)
- C 618 Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
- C 856 Standard Practice for Petrographic Examination of Hardened Concrete
- C 989 Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars
- C 1105 Standard Test Method for Length of Concrete Due to Alkali-Carbonate Rock Reaction
- C 1240 Standard Specification for Silica Fume for Use in Concrete
- C 1260 Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)
- C 1293 Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction

#### *Canadian Standards Association*

- A23.1 Concrete Materials and Methods of Concrete Construction
- A23.2 Methods of Test for Concrete

The above publications may be obtained from the following organizations:

American Concrete Institute  
P.O. Box 9094  
Farmington Hills, MI 48333-9094

ASTM International  
100 Barr Harbor Dr.  
West Conshohocken, PA 19428

Canadian Standards Association  
178 Rexdale Blvd.  
Rexdale, ON  
Canada M9W 1R3

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