Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures

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16. Abstract
Alkali-aggregate reaction (AAR) is only one of the many factors that might be fully or partly responsible for the deterioration and premature loss in serviceability of concrete infrastructure. Two types of AAR reaction are currently recognized depending on the nature of the reactive mineral; alkali-silica reaction (ASR) involves various types of reactive silica (SiO_2) minerals and alkali-carbonate reaction (ACR) involves certain types of dolomitic rocks ($CaMg(CO_3)_2$). Both types of reaction can result in expansion and cracking of concrete elements, leading to a reduction in the service life of concrete structures.

This document described an approach for the diagnosis and prognosis of alkali-aggregate reactivity in transportation structures. A preliminary investigation program is first proposed to allow for the early detection of ASR, followed by an assessment (diagnosis) of ASR completed by a sampling program and petrographic examination of a limited number of cores collected from selected structural members. In the case of structures showing evidence of ASR that justifies further investigations, this report also provides an integrated approach involving the quantification of the contribution of critical parameters with regards to ASR.

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1.0 Introduction

Alkali-aggregate reaction (AAR) is only one of the many factors that might be fully or partly responsible for the deterioration and premature loss in serviceability of concrete infrastructure. Two types of AAR reaction are currently recognized depending on the nature of the reactive mineral; alkali-silica reaction (ASR) involves various types of reactive silica (SiO_2) minerals and alkali-carbonate reaction (ACR) involves certain types of dolomitic rocks ($CaMg(CO_3)_2$). Both types of reaction can result in expansion and cracking of concrete elements, leading to a reduction in the service life of concrete structures (ACI 221.1R-98).

In many cases, several deleterious mechanisms will act simultaneously or consecutively, thus contributing to the damage observed; this is particularly the case in northern regions where freezing and thawing cycles will definitely contribute at increasing damage in concrete affected by other deleterious mechanisms such as AAR, sulfate attack, or others. It is consequently crucial, when assessing the cause of damage affecting a concrete structure, that every mechanism that may have contributed to the deterioration observed be considered. One should remember that an incorrect diagnosis may lead to the implementation of inappropriate/ineffective remedial actions.

Generally, it is only after a fairly extensive program of field and laboratory investigations that AAR can be confirmed as the main cause or a contributor to the deterioration observed. Such detailed investigations will likely include one or several of the following steps: 1) the survey of the presence/distribution and severity of the various defects affecting the concrete structure (especially those features diagnostic of AAR), 2) in-situ monitoring of deterioration (especially signs of expansion and deformation), and 3) a range of laboratory tests (including petrography, chemical, physical, and mechanical tests) on samples collected from one or several components of the affected concrete structure.

Visual symptoms on concrete structures affected by ASR and ACR are generally similar; i.e., evidences of expansion, relative movements between structural members showing different expansion rates, cracking. Petrographic examination generally allows differentiating ASR from ACR as deleterious expansion and cracking due to ASR relies on the formation of a secondary reaction product called *alkali-silica gel* that can generally be observed in concrete members affected by this mechanism. Since cases of ACR are generally limited and considering that the large majority if not all investigations to date related to the management of AAR-affected concrete structures have been carried out on structures affected by ASR, this document will focus and provide guidance for the early detection, the evaluation of the current condition, and the estimation of the future expansion and deterioration (prognosis) in concrete pavements and highway structures <u>in relation to ASR only</u>. The information thus generated through the series of investigations detailed in this report will lead to the selection of the most appropriate/effective remedial actions. The latter will be treated in Section 6.0. Readers interested in alkali-carbonate reactivity are invited to consult ACI 221.1R-98, which provides information on the manifestations of distress due to ACR, the mechanisms involved and the nature of the reactive rock types, testing for potential alkali-carbonate reactivity, and preventive measures against ACR.

Walker et al. (2006) also illustrates the features that provide evidence of alkali-carbonate reactions. If ACR is suspected in a particular concrete structure, it is highly recommended to contact someone with experience of this type of reaction.

2.0 General Approach

The global approach proposed for the diagnosis and prognosis of ASR in transportation structures is illustrated in the flow chart in Figure 1 and briefly described hereafter; Table 1 lists and provides an appreciation of the value of the various investigation tools/activities commonly performed in the field and in the laboratory for the diagnosis and prognosis of ASR in concrete structures. The global investigation program can be divided into three levels, as described hereafter.

2.1. ASR Investigation Program Level 1: Condition Survey

Signs of premature deterioration in concrete pavement and bridge structures that could be related to ASR can generally be detected during routine site inspections (*condition survey*) that are performed regularly by trained personnel of the State Highway Authorities (Van Dam et al. 2002). Visual symptoms of deterioration are noted and compared to those commonly observed on structures affected by ASR. If no visual signs suggestive of ASR are noted during the routine inspection program, further work is postponed until the next inspection. However, when the visual signs of deterioration observed on the structure(s) examined are such that AAR is a possibility, a "preliminary" investigation program (Level 2) is recommended to confirm the first diagnostic obtained from the visual survey.

2.2. ASR Investigation Program Level 2: Preliminary Studies for the Diagnosis of ASR

First, any documents relating to the structure and the materials used for the construction (e.g., construction files including results of AAR tests performed, reports from previous surveys/investigations on the structure, etc.), and reports on cases of ASR in the region (if any), should be gathered and reviewed. This "review of documentation" step could also be carried out in preparation for the condition survey (Investigation Level 1); as such information may assist in the appraisal of the structure.

Field activities at this *Level 2* consist in: 1) a measurement of the extent of cracking (*Cracking Index (CI) method*) on the most severely exposed/cracked sections of concrete; and 2) a "preliminary" sampling program on a selected number of elements from the concrete structure(s) examined. The quantitative assessment of the extent of cracking through the *Cracking Index*, along with the *Petrographic Examination* of the cores taken from the same affected element, is used as tools for the early detection of ASR in the concrete.

Cores are generally collected in concrete members showing visual signs of deterioration subjective of ASR and are then subjected to *petrographic examination* in the laboratory. If petrography does not confirm the presence of ASR in the concrete member examined, further investigations for other mechanisms of deterioration could be initiated, if necessary. On the other hand, when petrographic evidence of ASR is confirmed, a decision on the further steps to follow is then taken on the basis of the severity/extent of the cracking observed as follows:

- If the extent of cracking is limited (i.e., cracking index < criteria selected) and only limited to mild petrographic evidences of ASR are observed in the concrete, no immediate action is required; progress in deterioration will be monitored through cracking index measurements to be carried out as part of the next routine inspection survey.
- If the extent of cracking is considered "important" (i.e., the CI is > selected criteria) and definite petrographic signs of ASR are noticed, additional work may be required (i.e., ASR Investigation Program Level 3) and/or immediate remedial actions can be applied.

The decision regarding the nature and the magnitude of further actions to be taken at this stage will likely depend on factors such as the "criticality" of the structure and the extent of the damage observed. In some cases, it may be decided to limit further "technical" investigations and proceed immediately with some remedial actions such as the application of sealers and/or lithium-based products, corrections to drainage systems, etc. More details on "early-stage" remedial actions are discussed in Section 6.0. However, in the case of "critical" structures (e.g., large size highway bridges, Interstate/State concrete highway pavements) or when the extent of deterioration is judged significant enough to warrant further investigations, a detailed laboratory and/or in-situ investigation program may be necessary before selecting the best remedial measure to apply (ASR Investigation Program Level 3).

2.3. ASR Investigation Program Level 3: Detailed Studies for the Diagnosis/Prognosis of ASR

The ASR Investigation Program Level 3 deals with the assessment of the current condition, i.e., determination of the degree of expansion/damage reached to date, and of the trend for future deterioration of the concrete undergoing ASR expansion. Such investigations will provide critical information for the selection of the appropriate remedial actions to implement in ASR-affected concrete members/structures.

An in-situ investigation program which includes monitoring of expansion and deformation generally provides the most reliable "prognostic" for ASR-affected structural members. Considering the seasonal variations in climatic conditions that affect the progress of ASR and the differences in the reactivity levels of aggregates and other mix designs considerations (alkali contents, etc.), it is generally considered that a minimum of 2 years and ideally 3 years are required for reliable decisions on the implementation of remedial actions to be drawn from in-situ monitoring programs. A reasonable estimate of the potential for further expansion/deterioration can also be obtained through a detailed laboratory testing program. Such a program involves a series of tests on cores extracted from the concrete member / structure investigated, as listed in Table 1. In most severe cases of deterioration, an assessment of structural integrity may be required. The above investigations will provide further critical information in the selection of repair and/or mitigation strategies.

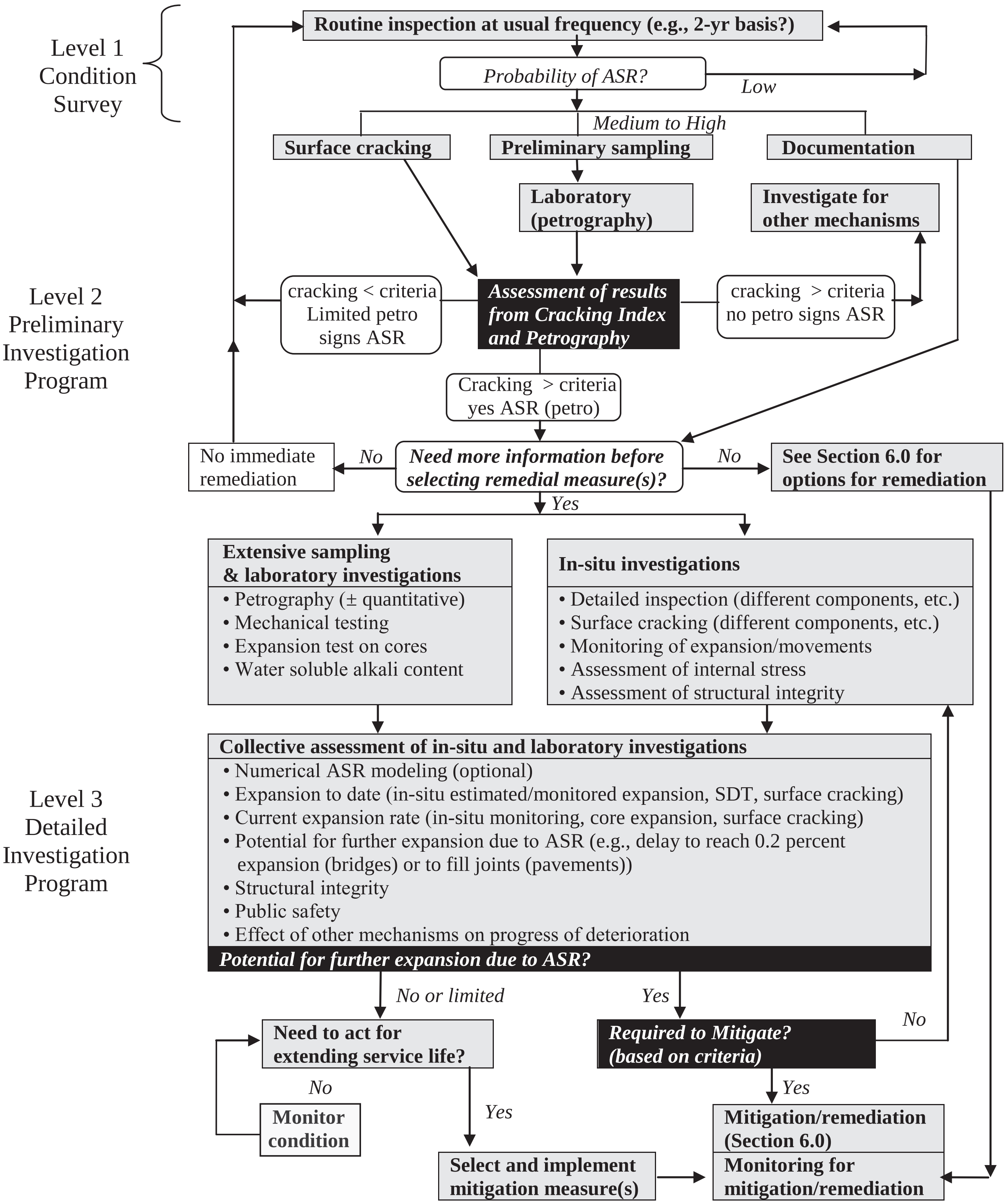

Figure 1. Global flow chart for the evaluation and management of concrete structures for ASR.

Table 1. Investigation tools for the diagnosis and prognosis of ASR in concrete structures (Fournier et al. 2004, adapted from BCA 1992).

Test / investigation	Main Objective	Diagnosis	Prognosis
Site investigation (condition survey)	• Assess the nature and extent of distresses and deterioration, and the risks relative to structural integrity and public safety • Assess the exposure conditions • Select sites for sampling and cracking measurements	XXX	X
Documentary evidence on concrete structures investigated	Collect and review available documents relating to the design, construction, survey and maintenance of the structure(s)	XXX	X
Initial and periodic measurement of cracks (Cracking Index)	• Assess deterioration and expansion level reached to date • Assess current rate of expansion rate • Assess potential for future expansion	XXX	XX
Petrographic examination			
• Macroscopic description	• Describe general condition of concrete cores • Identify macroscopic features of ASR	XXX	X
• Microscopic examination using polished slabs, thin sections (impregnated or not), broken pieces of concrete (possible uranyl acetate treatment)	• Identify reactive rock types and distribution • Identify presence and distribution of reaction products • Identify sites of expansive reaction • Identify pattern of internal cracking	XXX	X
• Quantitative petrographic analysis on polished slabs	Quantify extent of ASR damage due to ASR (e.g. cracking, gel) and possibly progression with time	XX	X
Mechanical testing			
• Compression and splitting tensile testing	• Assess general condition of concrete • Assess structural properties of members	XX	
• Direct tensile strength, flexural strength, and Young modulus	• Assess possible ASR • Assess structural properties of members	XX	
• Stiffness Damage Test	• Assess internal damage due to ASR • Assess the expansion level reached to date	XXX	
Expansion test on concrete cores			
• Cores at $38^{o}C$, R.H. > 95 percent	• Confirmation of deleterious expansion • Assess current rate of expansion • Assess potential for future expansion	XX	XXX
• Cores in 1N NaOH at $38^{o}C$	• Identification of reactive aggregates • Assess residual reactivity of aggregates • Assess potential for future expansion	X	XX
Determination of the water soluble alkali content of concrete	• Assess potential sources of alkalis • Assess potential for future expansion	XX	XX
Monitoring of expansion and movements	• Confirmation of deleterious expansion • Assess current rate of expansion • Assess potential for future expansion	XX	XXX
In-situ assessment of internal stresses and structural integrity	• Stresses in concrete and reinforcements • Assessment of structural damage and integrity	XX	
Numerical AAR modeling	• Confirmation of deleterious expansion • Assessment of structural damage and integrity • Forecasting future expansion and stability • Predict structural responses to remedial actions	XX	XX

X: Results could be useful if test can be done; XX: Do when possible; XXX: Important test

3.0 ASR Investigation Program Level 1: Condition Survey

3.1. General

The condition survey is generally carried out to provide data on: 1) the nature, the extent, and the progress (when damage ratings are established and repeated as part of the survey program) of any distresses and deterioration affecting the concrete structure; and 2) identify areas that may need further investigations and/or immediate action (i.e., repair).

With the special purpose of diagnosing for AAR in the concrete structure, special attention should also be given at assessing the exposure conditions to which the structure (or parts of it) is (are) subjected. ASR typically develops or sustains in concrete elements with internal relative humidity > 80-85 percent. ASR is not expected to develop to a significant extent in a dry environment, which corresponds to an average ambient relative humidity lower than 60 percent, normally only found in buildings. For intermediate conditions; i.e., between 60 and 80 percent, the extent of AAR will depend on factors such as the nature and reactivity level of the aggregate; however, the rate of expansion will be reduced compared to higher humidity conditions.

Expansion and cracking due to ASR is generally most severe in concrete elements subjected to an external and constantly renewable supply of moisture. Surfaces of concrete elements affected by ASR and exposed to sun, wetting and drying cycles (e.g., splash zones on bridge parapet/abutment walls), frost action (freezing and thawing cycles), saline water (e.g., tidal zones for structures exposed to sea water, splashing zone on the abutment walls of bridges or jersey barriers exposed to deicing chemicals), usually show more extensive/severe cracking and deterioration; although the above conditions are not necessarily promoting expansion due to AAR, they are exacerbating its effects and the damage it generates.

Common visual symptoms of ASR have been described in numerous documents since Stanton identified and reported the first case of ASR in concrete structures in the late 1930s (Stanton 1940)[1]. Although not necessarily exclusive to ASR, they generally consist of:

- Expansion causing deformation, relative movement, and displacement.
- Cracking.
- Surface discoloration.
- Gel exudations.
- Occasional pop-outs.

Detailed information and photographs illustrating the above defects in highway structures affected by ASR are given in **Appendix A** of this document.

[1] The following publications provide additional information on the topic: Stark (1991), BCA (1992), Farni and Kosmatka (1997), ACI (1998), LCPC (1999), CSA (2000), Fournier and Bérubé (2000), Van Dam et al. (2002), and Folliard et al. (2006).

As part of the condition survey, each component of the structure should be examined separately and observations on the type, extent (severity), and location of the defects, recorded in a consistent manner (e.g., using a condition survey form). Typical examples of the distress(es) observed should be photographed (including an indication of scale); this will help compare severity ratings of the damage between various parts of a single component, various components of the structure, as well as between conditions surveys. Sketch(es) and/or picture(s) of the structural members should be used to locate the areas of low, medium, and high damage severity, as well as any evidence of a potential relation between the damage observed and features such as the presence of physical restraints and the availability of moisture (exposure to rain, poor or defective drainage systems, etc.).

A field test to detect the presence of ASR silica gel by using uranyl acetate fluorescence was developed under the Strategic Highway Research Program (SHRP) in the United States (D. Stark 1991; Natesaiyer, et al.). Care should be taken in interpreting the results (see ASTM C 856 and section C2.2 – Appendix C).

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

3.2. Interpretation of the Findings from the Condition Survey

Table 2 classifies the occurrence of the features obtained from the condition survey as indicative of low, medium, and high potential of ASR contribution in the deterioration observed. It is often difficult to determine from field observations only whether ASR is the only/main factor responsible for the observed distresses since some of the visual signs of deterioration generally associated with ASR may have been caused by other processes such as internal sulphate attack, or plastic or drying shrinkage.

Table 2. Classification system for the condition survey (modified from CSA A864-00)

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

The assessment of the exposure conditions should also contribute to support the observations of the symptoms of distress listed in Table 2, as follows:

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

As indicated in Figure 1, if the potential for ASR contribution is low (i.e., no conclusive evidence of AAR-related distress is noted), further work is postponed until the next condition survey. However, when the potential for ASR contribution is medium to high, further work is required, which will be carried out as part of a preliminary investigation program for the diagnosis or ASR (Figure 1).

4.0 ASR Investigation Program Level 2: Preliminary Studies for the Diagnosis of ASR

The objective of this part of the program is to confirm the results of the condition survey, i.e., determine whether or not ASR is a contributing factor in the deterioration observed, and to generate preliminary quantitative assessment of the extent of damage. It consists in a limited number of field and desk/laboratory activities that will generate information leading to the selection of immediate remedial actions or that will open the door to a more detailed investigation program (*ASR Investigation Program Level 3*).

4.1. Documentation

Any documents (i.e., testing of materials, construction, and inspection reports) related to the structure examined should be gathered and examined as they may provide valuable information in the appraisal process. This activity could also be carried out either in preparation for the condition survey or following it (as in Figure 1), i.e., for structures where some signs of deterioration potentially indicative of ASR have been noticed. Useful information could include the following (CSA A864, FHWA-HRT-04-113):

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

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

4.2. Measurement of the Cracking Index (CI)

The development and the extent of surface cracking on concrete structures or members exposed to the elements is a function of many factors. In the case of concrete members undergoing internal expansion due to ASR and subjected to wetting and drying cycles (cyclic exposure to sun, rain, wind, or portions of concrete piles in tidal zones, etc.), the concrete often shows surface cracking because of induced tension cracking in the "less expansive" surface layer (because of variable humidity conditions and leaching of alkalis) under the expansive thrust of the inner concrete core (with more constant humidity and pH conditions). The extent of surface cracking on those elements is thus somewhat related to the overall amount of expansion reached by the affected concrete member.

The Cracking Index (CI) is a crack mapping process that consists in the measurement and summation of crack widths along a set of lines drawn perpendicularly (i.e., parallel and perpendicular to the main restraint(s)) on the surface of the concrete element investigated. The method gives a quantitative assessment of the extent of cracking in structural members.

The CI is introduced here, in combination with petrographic examination of cores (see Section 4.4) to provide decisionmaking criteria for the early detection of ASR and the selection of further actions right in this early stage of the global investigation program. Details on the CI method, the type of and conditions for readings, and reporting of the data are given in **Appendix B** and are summarized below.

4.2.1. *Number and location of the CI reference grids*

In order to generate a statistically representative assessment of the extent of cracking through the CI method, a minimum of two CI reference grids, 0.5m (20 in) in size, should be drawn on the surface of the most severely cracked structural components (see **Appendix B**). Those components generally correspond to those exposed to moisture and severe environmental conditions, as well as those where ASR should normally have developed to the largest extent.

4.2.2. *Timing of the readings*

Because of the significant effect of temperature and humidity on crack widths, CI readings should be carried out and repeated under very similar conditions of sun exposure, outdoor temperature, and outdoor humidity conditions (see **Appendix B** for details).

4.2.3. *Measurements and calculation of the CI*

The width of each crack crossing the four lines drawn on the surface of the element investigated is measured using a magnifying lens with internal gradations (e.g., 0.05 mm (0.002 in)) or a plastic crack comparator. As described in **Appendix B**, the CI is calculated and expressed in mm/m (in/yd).

4.2.4. *Criteria*

The following *cracking criteria*, which are obtained from the crack mapping survey performed as part of the Cracking Index method, are proposed to identify an extent of cracking that should justify more detailed investigations.

CI > 0.5 mm/m (0.018 in/yd), and/or Cracks of width > 0.15mm (0.006 in)

The estimate of the expansion attained to date by the structural member is a critical parameter in the evaluation of its current condition in view of selecting appropriate remedial actions. This is, however, not a parameter easy to determine in most cases. Section 5.2.2 indicates that the *Cracking Index* measured on the most exposed concrete component (which is most of the time the one showing the most severe signs of deterioration due to ASR), can give a rough estimate of the expansion reached to date by the element under investigation. The Institution of Structural Engineer (ISE 1992) suggests that expansions in structural members in excess of 0.5mm/m (0.018 in/yd) should warrant further investigations and that the potential consequences of such expansions should be assessed. In addition, cracks in excess of 0.15mm (0.006 in) in width could start to be a source of concerns, especially in the case of prestressed concrete members, and should also justify further investigations (ISE 1992). Table 3 gives ACI 224 committee guide to reasonable crack width for structural concrete members under service loads (ACI 224R-01). The limits range from 0.41 mm (0.016 in) in the case of members exposed to dry air or protected by a membrane to 0.10 mm (0.004 in) for water-retaining structures. In the case of highway bridge structures and pavements, cracks of 0.15mm (0.006 in) in size are large enough to raise attention and justify some investigations aiming at identifying their cause, thus potentially allowing an early detection of ASR cracking and some early remedial actions.

Table 3. Guide to reasonable crack widths, reinforced concrete under service loads (From ACI 224R-01).

Exposure conditions	Crack widths	
	mm	in
Dry air or protective membrane	0.41	0.016
Humidity, moist air	0.30	0.012
Deicing chemicals	0.18	0.007
Seawater and seawater sprays, wetting and drying	0.15	0.006
Water-retaining structures	0.004	0.10

4.3. Preliminary Sampling Program

4.3.1. *Nature and extent of sampling*

Sampling is carried out from a limited number of components of the structures, essentially to determine whether or not the concrete contains petrographic evidence of ASR. A minimum of two cores will thus be collected from each of those components showing typical to more severe signs suggestive of ASR (see **Appendix A**), which will or should most often be structural components exposed to a constant or renewable supply of moisture, with/without cycles of wetting and drying. For comparison purposes, it will also be appropriate to collect a few cores from structures that are less deteriorated than the structure in question, not deteriorated, or not exposed to the environment (i.e., to environmental elements).

4.3.2. *Type and size of samples*

Core samples, typically 100 mm (4 in) in diameter, are most suitable; however, special circumstances, i.e., where larger size aggregates or closely spaced reinforcement, may require cutting larger or smaller cores. Smaller cores are more susceptible to cracking during coring, which may be misleading regarding the severity of internal cracking. Also, the use of larger size cores (e.g., 150 mm (6 in) in diameter) will be beneficial in the case of expansion testing on cores at 38^{o}C (100°F) and R.H. > 95 percent as it will contribute at reducing leaching of alkalis during the test, thus generating more reliable test data for estimating the potential for future expansion (see Section 5.3.4).

As petrographic symptoms of ASR are known to vary from the surface to the interior part of the affected element, cores should be as long as possible to provide a profile of the element sampled. In the case of massive concrete elements such as abutment walls, reinforced concrete columns, and beams, the cores should be at least 30 cm (12 in) long. In the case of thinner concrete elements such as pavements, bridge decks, and parapet walls, cores should pass through the whole element.

4.3.3. *Treatment of samples and information collected*

A detailed record of all sampling operations should be made on site. The use of a sampling form or “Site Core Record” accompanied by pictures showing the characteristics of the components sampled is most appropriate. The information recorded on site should include the following:

- Sketch showing location of core.
- Photograph of core location.
- Size (diameter and total length) and orientation.
- Record of any features that may be indicative of ASR, such as damp patches on core surfaces, gel in cracks and voids, or reaction rims around aggregate particles.

The samples collected should be labeled carefully, photographed and, immediately after their preliminary examination, wrapped in a plastic film and sealed in a plastic bag to prevent alkali-silica gel and surfaces to carbonate, become contaminated, or dry out during subsequent transport and storage.

4.4. Petrographic Examination

As mentioned before, this *ASR Investigation Program Level 2* consists in identifying the presence (or not) of petrographic signs of ASR in the cores sampled from those components showing visual signs of deterioration most suggestive of ASR. Although not necessarily exclusive to ASR, petrographic signs of ASR generally consist of:

- microcracking in aggregates and/or cement paste;
- reaction product “gel”;
- reaction rims; and

- loss of the cement paste-aggregate bond.

Detailed information and photographs illustrating petrographic signs of ASR are given in **Appendix C** of this document, including a discussion on the use of the uranyl acetate test for the identification of alkali-silica gel in concrete[2]. **Appendix C** also includes a table that lists features to look for from petrography as a function of the method of examination used.

4.4.1. *Macroscopic description of the core (as received)*

The cores are first examined and photographed in an 'as-received' condition. If the surfaces of the cores are dry, they should be dampened and replaced in a plastic bag for an additional 24 hours before examination. The "macroscopic" description of the core is generally performed with the naked-eye and with a magnifying lens (7-10x) or a stereo-binocular (generally up to 60x). Certain features may be highlighted by rewetting the core surfaces and making observations as the core dries. In addition to observations normally made on core samples (e.g., size and distribution of aggregate, compaction, void content, and presence and condition of reinforcement), observations regarding the following features may assist in the diagnostic process, and their presence should be noted:

- Cracking location (e.g., around or through aggregate particles, etc.), associated gel exudation, crack width, depth of surface cracking, etc.
- Presence of gel (and other secondary reaction products – e.g., ettringite) in voids, cracks, around aggregate particles, or exuding from the core.
- Damp patches on the concrete surface.
- Reaction/weathering rims around aggregate particles.
- Any signs of concrete disintegration.
- Presence, size, position, and condition of reinforcement.

4.4.2. *Microscopic description of the core*

Following the macroscopic description of the cores, various types of specimens may be prepared from the drilled cores. These mainly consist of **polished sections** or slices, **broken (fresh) surfaces**, and **thin sections**. The examination of polished surfaces with the naked eye and low-powered (stereo-binocular; up to 60x magnification) microscopy is an efficient method for studying large areas of concrete and determining the presence, distribution, and extent of macroscopical features of AAR. The examination of thin sections (magnifications of up to 250x) will allow to further positively identify diagnostic features of ASR (e.g., sites of expansive reactions, reaction products). To maximize information generated through petrographic examination, polished slabs and thin sections can be prepared from various depths along the core sample.

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[2] ASTM C 856 outlines procedures for the petrographic examination of samples of hardened concrete, while *The Petrographic Manual* (Walker 1992, Walker et al. 2006, FHWA-06) is a valuable source of procedures for petrographic examinations related to ASR.

As mentioned before, **Appendix C** provides a description (with photographs) of the various petrographic features on ASR that can be obtained from the examination of polished slabs and thin sections. In summary, the following information may be obtained from the above examination (CSA 2000):

- Description of aggregates (for further information on the identification of rock types and potentially reactive components, see ASTM C 295).
- General characterization of microcracking, including intensity, size range of cracks, apparent association with particular aggregate type, cracking in or around aggregate particles, and presence of gel or any other deposits in cracks.
- Presence of reaction and/or alteration rims around aggregate particles ("reaction" rim types to be distinguished from "weathering" rims that are sometimes observed surrounding weathered gravel particles).
- Presence of gel or other deposits in voids.
- Sites of expansive reaction—occurrences of features that provide evidence of reaction and emanation of expansive forces, e.g., reactive aggregate particles showing cracking internally or at the cement/aggregate interface, with cracks propagating into the surrounding matrix and cracks filled or partially filled with gel.

4.4.3. *Interpretation of the findings from petrographic examination*

Petrographic examination on polished and thin sections, when conducted by a qualified petrographer experienced in the examination of concrete affected by ASR, is the most powerful tool to confirm the occurrence of that deleterious phenomenon. Table 4 classifies the occurrence of the features obtained from the petrographic examination as indicative of low, medium, and high probability of ASR.

As indicated in Figure 1, if the probability of ASR from petrographic examination is low, it may be advisable to investigate for other mechanisms to explain the deterioration observed. However, when the probability of ASR is medium to high, further work may be required.

Some cases may, however, justify special attention. It would be the case, for instance, where some petrographic signs of ASR would be identified in a fairly young but critical structure. Such observations may not at that stage justify any immediate remedial actions; however, it would contribute to raising attention and justifing accelerating the frequency of investigations (condition survey and petrographic examination).

Table 4. Classification system for petrographic examination (modified from CSA A864-00, see Appendix C).

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

4.5. Assessment of the Results from CI and Petrographic Examination

As illustrated in Figure 1, the assessment of the results from the crack mapping measurements in the field and from the petrographic examination in the laboratory will lead to options described in Table 5.

Table 5. Collective assessment of the findings from the cracking index and petrographic examination.

Cracking	Probability of ASR	Findings	Recommended Actions
Cracking < agency specified criteria	Low probability of ASR (from petrography)	Although cracking is noted in the element examined, the extent of cracking is still limited; there is no conclusive evidence of ASR in the concrete (based on petrography).	Monitor the progress in cracking by repeating the crack mapping (i.e., CI) process as part of the next routine inspection survey. If evidence of progress in cracking is noted, further coring and petrographic examination is recommended to evaluate the progress in internal distress due to ASR.
	Medium to high probability of ASR (from petrography) (situation not in Figure 1 flow chart)	This a fairly unlikely situation as ASR, when present to a significant extent in concrete, generally leads to noticeable cracking at the surface of at least on the most severely exposed affected elements. It may however correspond to a relatively early stage of ASR. Also, some signs of ASR may be observed in the case of some reactive aggregates such as opal-bearing or cherty that may react close to the surface (thus producing pop outs) or that may dissolve in the concrete without necessarily inducing significant cracking in the concrete element as a whole.	Initiate further investigations on other members of the structure (e.g., assess effect of exposure conditions, look for signs of expansion, coring of other members for petrography).
Cracking > agency specified criteria	Low probability of ASR (from petrography)	Significant cracking is affecting the element investigated. On the other hand, there is no conclusive evidence of ASR in the concrete (based on petrography).	Initiate further investigations for other mechanisms of deterioration.
	Medium to high probability of ASR (from petrography)	Presence of significant to extensive signs of ASR, both in-situ (cracking) and internally (petrography).	Additional investigations may be required to establish the expansion reached to date and the potential for further expansion, leading to the selection of the most appropriate remedial action. Some immediate remedial actions (e.g., application of sealers) may already be a possibility at this stage (i.e., without any further investigations).

4.6. Necessity to Pursue Investigations or Not

A decision should now be made as to whether or not a remedial action could or should be selected/implemented at this stage, in other words whether or not additional investigations should be carried out before any remedial actions could be implemented.

Based on the results of the field (condition survey and cracking index) and laboratory (petrographic examination) investigations carried out so far, we should be in the presence of concrete structures showing noticeable signs of deterioration. The latter is likely to range from mild to severe, and be totally or at least partially associated with ASR. The review of documentation may also have contributed useful information supporting a low to high probability of ASR [e.g., concrete structure "sufficiently" old for ASR distress to develop (i.e., generally > 10 years), previous inspection reports suggesting ASR as a possible cause of distress, materials testing reports showing that potentially reactive aggregates have been used, mix design information indicating that fairly high concrete alkali loadings have been used, etc.].

The field and laboratory investigations performed so far may have identified cases where some remedial actions could/should already be implemented. Examples of such cases are given in Table 6.

Table 6. Examples of potential "early-stage" remedial actions.

Type of Structure (Table 7)	Damage	Signs of ASR	Rationale for Implementing Immediate Remedial Action
S1 and S2	Mild to moderate	Mild to moderate	• No requirement for detailed studies (limited deterioration/AAR, type of structures). • Prevent or slow down further damage. • Structural stability and integrity issues. • Note: some monitoring of repair needed (especially for S2).
S1	Severe	Mild to moderate	• No requirement for detailed studies (type of structures). • Prevent further damage. • Structural stability and integrity issues.
S3 and S4	Mild to moderate	Mild to moderate	• Correct some obvious issues identified during condition survey (e.g., modify drainage system to control moisture). • Some inexpensive early-action measures (e.g., application of sealers).
S2, S3, and S4	Severe	Mild to severe	• Need further investigations for selecting remedial actions.

Section 6.0 discusses some remedial measures that could be implemented in such cases, with the limitations involved.

Critical information leading to a more complete assessment of ASR in the concrete structure and the selection of most appropriate remedial measures is, however, still missing. Such information is related to the following questions:

- What is the current condition (or the degree of damage) of the concrete element?
- What stage of the ASR-deteriorating process or what level of expansion has been reached to date?
- Is the structural integrity in danger (e.g., relating to the stability of the reinforcing steel and of the steel-to-concrete bonding, development of extensive spalling at joints in pavements)?
- How much more expansion could be expected?

- How much additional deterioration can be expected (impact on long-term durability and service life)?

The *ASR Investigation Program Level 3* described in Section 5 aims at answering the above questions, which will be especially important in the case of critical structures such as Class S3 and S4 structures in Table 7.

Table 7. Structures classified on the basis of the severity of the consequences should ASR occur (modified from RILEM TC 191-ARP).

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

[†]Note: this table does not consider the consequences of damage due to ACR. This report does not permit the use of alkali-carbonate aggregates.

5.0 ASR Investigation Program Level 3: Detailed Studies for the Diagnosis/Prognosis of ASR

5.1. Introduction

The selection of appropriate remedial measures for ASR-affected structures requires a good knowledge of the current condition of the concrete and its potential for future deterioration. For instance, in the case of reinforced concrete members, determinations of the *expansion reached to date*, the *current rate of expansion* and the *potential for future expansion* of the concrete are particularly critical in order to determine whether or not the reinforcing steel in the structural element has reached or will at some point reach its plastic limit, thus creating risk of structural failure. This is particularly important in mildly reinforced structural members affected by ASR. Similarly, the above information will be crucial for highway concrete pavements where extensive cracking and spalling at joints would create serious serviceability issues. Also, as cracking develops due to the expansion in any ASR-affected unreinforced or reinforced transportation structure, durability issues related to freezing and thawing (where present), external sulfate attack, steel corrosion, etc. will likely develop and affect the service life of the affected member.

A framework for the selection of the time/frequency and magnitude of the remedial action(s) is proposed below, as a function of estimate/ measurements of the following parameters:

- The expansion reached to date.
- The current expansion rate.
- The potential for further expansion due to ASR (e.g., number of years before the steel reinforcement yield could occur (in the case of RCC members) or joints in concrete pavement will close).
- The stability (or structural integrity) of the structure.
- The safety of persons/vehicles.
- The potential for further deterioration due to other mechanisms (extended service life).

The third parameter above is a function of the first two; in other words, remedial actions will be required more rapidly in structures where steel reinforcement or joints in pavements may shortly become “at risk of failure/serviceability issues,” no matter what age it is.

In order to generate the information required for the selection of remedial actions, as mentioned above, a combination of field (in-situ testing) and laboratory activities/investigations is proposed in Figure 1. Table 8 summarizes the information required and the various approaches/ activities proposed to obtain it. The selection of the activities to be carried out will depend on the criticality of the structure, the amount of time available to generate the data, and the degree of precision expected. For instance, a more precise indication of the expansion rates will be obtained from the in-situ monitoring of expansion in affected structural members. However, considering the seasonal and thermal effects, it will generally take a minimum of 2 and preferably 3 years to obtain reliable data from that approach. Expansion tests on cores submitted to high temperature (38^{o}C)(100°F) and humidity (> 95 percent RH) conditions will generate data in 6 to 12 months. However, how representative of the behavior of the overall structure will that data be, is still under debate as this kind of approach/investigation program is still very much under development worldwide.

Table 8. Activities to be carried out as part of the detailed investigation program.

Information	In-situ testing	Laboratory investigation
Expansion reached to date	• Monitoring. • Crack widths on the affected concrete members.	• Modified Stiffness Damage Test performed on concrete cores (see section 5.3.3).
Current condition of the concrete	• Detailed visual inspection. • Non-destructive testing (pulse velocity, impact echo). • Stress conditions (overcoring (see Section 5.2.4) , strain gages on reinforcements which are cut). • Measurements of temperature and humidity.	• Petrographic examination. • Mechanical testing (compressive and direct tensile strengths, direct tensile-to-compressive strength ratio, modulus of elasticity).
Current expansion rate	• Instrumentation and monitoring of expansion (minimum 3 years).	• Expansion test on cores (1 yr). • Measurement of water-soluble alkalis in the concrete.
Potential for future expansion	• Determination from monitoring of expansion.	• Estimation from expansion test on cores (1 yr), measurement of water soluble alkalis in the concrete and some field considerations (humidity, temperature, and stress conditions).

5.2. In-situ Investigations

Detailed field investigations are carried out through site inspection and expansion, deformation and displacement measurements on the structure.

5.2.1. *Detailed site inspection and selection of sites for further in-situ investigations*

Detailed investigations are proposed that complement the observations made as part of the condition survey (Section 3). The necessity to carry out such investigations will depend on the criticality of the structure, the extent of damage and the extent of information required; special attention will be given to generate a quantitative assessment of the extent (severity) of deterioration between the different members of the structure. This will provide an overall condition rating of the structure (and of its members); the Cracking Index method can be used for that purpose (see Section 4.2 and **Appendix B**). The detailed inspection will also focus on locating the signs of expansion and relative movement in the structure (and measure the extent of the relative movement to date), the sources and availability of moisture and the effect of the restraint on expansion for the various members of the structure. It will also help in selecting locations for:

- Additional coring.
- Installation of probes for temperature and humidity measurements.

- Installation of demec points[3] and other devices for expansion and/or relative movement measurements.
- Stress measurements (if required) in the steel reinforcement (see Section 5.2.4) and/or in the concrete.
- Possibly where some remedial actions could be implemented (e.g., modification in the drainage system, application of surface treatments, etc.).

5.2.2. *Surface cracking*

Surface cracking measurements will be carried out using the Cracking Index method described in section 4.2 and **Appendix B**. This crack mapping process will however be carried out on members of the structures showing a range of deterioration, but always exposed to moisture. As mentioned before, this will provide a comparative, quantitative rating of the "surface" deterioration affecting the structure as a whole. The measurements of the Cracking Index will then be used to estimate the expansion reached to date by the concrete (see Section 5.5.2), as well as the current rate of expansion (see Section 5.5.3). The CI measuring grids will be well identified so that crack mapping could be repeated over a 2- to 3-year period to allow the monitoring of changes in the extent of surface cracking affecting the structural members selected.

5.2.3. *Expansion and deformation measurements*

In-situ deformation measurements will be performed by installing demec points and/or metallic references/devices at the surface of selected concrete members; periodic length-change measurements are then taken using extensometers of various shapes and ranges, invar wires/rods or optical systems (leveling). Fiber-optic and vibrating wire systems can also be used, with deformation measurements being performed and the data transmitted automatically to central servers for further treatment. Further details on the setting-up for expansion measurements are given in **Appendix D**.

5.2.4. *Stress measurements*

Evidence of stress build-up in reinforcing steel and the surrounding concrete resulting from restrained ASR expansion can be obtained from the measurements of stress in reinforcing bars (which are exposed, strain gauged, then cut to release the tensile stress) (see **Appendix D**) and the overcoring[4] technique (Danay et al. 1993).

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[3] Strain measurements are taken between different points on a structure using a single instrument, the *demountable mechanical strain gauge (*DEMEC), which consists of a standard or a digital dial gauge attached to an Invar bar. The points between which those measurements are taken are pre-drilled stainless steel discs or bolts which are attached to the structure with adhesive or drilled and "cemented" into the concrete (see Appendix D).

[4] In the overcoring technique, a hole is drilled in a concrete element to a selected depth (as required for the investigations). The end-face of the hole is ground flat to allow fixing of a strain gage. Drilling is then performed at a smaller diameter around the gage. Unexpected/excessive increase in strain could be measured as a consequence of relaxation in compressive stress (due to confinement release) in concrete experiencing expansive pressures due to deleterious mechanisms such as ASR.

5.2.5. *Temperature and humidity measurements*

It is commonly accepted that ASR develops or sustains in concrete elements with internal relative humidity >80-85 percent [relative to 23 °C (76 °F)]. The relative humidity in a concrete structure can be measured over time with depth or laterally in different concrete elements using various techniques such as wooden stick, or portable or permanent probes (Stark and Depuy 1987, Stark 1990, Siemes and Gulikers 2000, Jensen 2000) (see **Appendix D**). Humidity and temperature readings can provide useful information in the treatment (e.g., applying any corrections required for length-change measurements) and interpretation of expansion and crack measurements as those are influenced by the above conditions.

5.2.6. *Non-destructive testing*

Periodic pulse velocity measurements can be made on specific members or parts of the affected structure (at the surface or in the bottom of drilling holes), and might permit assessment of the evolution and the extent of internal cracking or deterioration. Impact echo measurements have been used to monitor the performance of concrete pavements affected by ASR after topical treatments with a lithium nitrate solution (Johnston et al. 2000, Stokes et al. 2003).

5.2.7. *Structural evaluation*

Especially in the case of reinforced concrete bridges where the visual survey and the in-situ measurements indicate a severe level of deterioration (e.g., C.I. > 0.5 mm/m (0.018 in/yd), crack width > 0.15mm (0.006 in), expansion > 0.05 percent), it will likely be necessary to determine whether or not the stability of the structure is at risk. This should be carried out through a full-scale investigation performed by a competent structural engineer. Full-scale loading tests in the field will ultimately permit assessment of the real structural loss in performance (or serviceability) of the affected structure (see **Appendix D**). The criteria for load tests are usually based on some limiting deflection criteria and recovery of the deflection with time (CSA 2000).

The assessment of the structural stability of an ASR-affected structure could also be made using a macro-scale numerical ASR model provided this model is sophisticated/reliable enough and fed with the required/pertinent information gathered from all in-situ and laboratory investigations (see Section 5.4).

5.3. Detailed Laboratory Investigations

A series of tests are described hereafter that can be performed on samples cored from various members of the structure and that will provide useful data in the determination of the current condition of the concrete, the expansion reached to date, the current rate of expansion, and the potential for future expansion of the concrete.

5.3.1. *Sampling and treatment of samples*

Requirements for the sampling process and the treatment of the specimens at site are similar to those described in Section 4.3 (Preliminary sampling). However, a much larger number of cores will be required, because of: 1) the various laboratory tests to be performed; and 2) the interest of sampling various components of the structure to assess the effect of various parameters (such as exposure conditions and restraint) on the results of the tests. Such requirements will be discussed in more detail as part of the various sections hereafter.

5.3.2. *Detailed petrographic examination*

Special attention is given at quantifying the degree of ASR-related damage between the different members showing various visual degrees (severity) of deterioration and/or various degrees of exposure conditions. This will provide a better picture of the overall condition of the structures and of its members. In order to do so, a quantitative petrographic technique is proposed, the *Damage Rating Index (DRI)*. The method is described in **Appendix C**; it evaluates the condition of concrete by counting the number of typical petrographic features of ASR on polished concrete sections (16x magnification) (Grattan-Bellew 1992, Dunbar and Grattan-Bellew 1995). The *DRI* represents the normalized value (to 100 mm^2) of the presence of these features after the count of their abundance over the surface examined has been multiplied by weighing factors representing their relative importance in the overall deterioration process (**Appendix C**).

The DRI method is a useful tool for the quantitative assessment, based on petrography, of internal damage in concrete due to ASR or other mechanisms. However, as the results are very much related to the experience of the petrographer and since there is currently no standard test procedure available, the method is fairly subjective and the results can be quite variable from one operator to another. Consequently, it is highly recommended that the method be carried out by the same petrographer, at least on the sets of cores from a same structure.

5.3.3. *Mechanical testing*

Compressive and tensile strengths and elasticity modulus – All mechanical properties of concrete are negatively affected by ASR, however not at the same extent or at the same expansion levels. Reductions by up to 60 percent for the compressive and splitting strengths, 80 percent for the direct tensile strength, and 60 percent for the elastic modulus have been reported. In most cases, the properties the most rapidly affected are the modulus of elasticity and the direct tensile strength, even before significant levels of expansion are attained. The compressive and splitting strengths generally behave similarly, being significantly affected only at relatively high expansion levels. The following tests/parameters are thus proposed for the early detection of ASR:

- The compressive strength (which is normally not significantly affected), required for determining the direct-tensile strength-to-compressive strength ratio.
- The direct-tensile strength, which may be significantly affected.
- The direct-tensile strength-to-compressive strength ratio; this ratio lies normally between 0.07 and 0.11 for sound concrete while values below 0.06 would indicate internal damage due to AAR (Nixon and Bollinghaus 1985).
- The elastic modulus, which is rapidly affected even at relatively low expansion levels (even less than 0.08-0.10 percent), then appearing relatively low with respect to the

theoretical value calculated from the compressive strength; the elastic modulus, however, tends to stabilize at high expansion levels.

The diagnostic value of the above tests, however, lies in the capacity to establish the reduction in mechanical properties due to ASR, which may require some assumptions/tests to establish the properties of the original/unaffected concrete[5]. Cores can be extracted from various members showing mild to severe surface cracking (and other symptoms suggestive of ASR) to determine if any changes in mechanical properties of the concrete are observed and, if so, the extent of such a variation.

Modified Stiffness Damage Test (SDT) – The Stiffness Damage Test was originally proposed by Chrisp et al. (1989) and adopted by the Institution of Structural Engineers (ISE 1992). Recently, the method was slightly modified and used for estimating the expansion attained to date by ASR-affected concrete (Smaoui et al. 2004a, 2004b, 2004c, Bérubé et al. 2005). In the modified test procedure described in **Appendix E**, the test specimens, which consist in 100 mm (4 in) cores extracted from the concrete member to be evaluated, are subjected to five cycles of uniaxial loading/unloading up to a maximum of 10 MPa (1450 psi). The following parameters are then used for assessing the ASR expansion attained to date: (1) the energy dissipated during the first cycle (hysteresis loop), and (2) the accumulated plastic strain after the five load/unload cycles. Both parameters progressively increase with increasing internal microcracking and expansion in concrete affected by ASR (see Figure E1 in **Appendix E**); however, the energy dissipated parameter was found to best correlate with AAR expansion, and is consequently recommended for assessing the expansion attained to date in concrete affected by ASR. However, the results were found to be aggregate-dependent, which requires the use or the establishment of a calibration curve suitable to the particular reactive aggregate involved or a similar type (see Section E.2.9 in **Appendix E**).

Cores for stiffness damage testing can be extracted from various members showing mild to severe surface cracking (or other symptoms suggestive of ASR) to determine if any changes in properties are observed and, if so, the extent of such a variation. This testing is recommended because of the inherent variability in the internal damage of deteriorating concrete members, which a set of SDT specimens consists of a minimum of three cores. Moreover, when the number of cores available for mechanical testing is limited, the modulus of elasticity can be determined in the modified SDT and the compressive strength can be obtained by reloading the cores up to failure after the five load/unload cycles (see **Appendix E**).

5.3.4. *Expansion tests on cores*

The potential for further expansion due to ASR is a critical parameter to consider when selecting the most appropriate remedial action(s) for concrete affected by ASR. Current rates of expansion are best established from periodic or continuous in-situ monitoring of deformations, which can then be extrapolated for estimating the potential for future expansion. However, in-situ

[5] One should remember that any reduction in performance may be due to a combination of mechanisms, and that the test results were obtained on specimens that have been extracted from their restrained conditions; consequently, they may not represent the exact condition (or structural capacity) of in-situ concrete.

monitoring will generally take a minimum of 2 and preferably 3 years to yield useful information, i.e., where permanent and cumulative deformation due to ASR could "reliably" be differentiated from reversible and cyclic movements related to mechanical (loading, traffic, operation conditions, etc.), thermal and climatic (daily and seasonal) variations.

On the other hand, expansion tests on cores can yield results in a relatively short period of time, e.g., 6 months to 1 year, which makes them fairly popular techniques for assessing the potential for further expansion of ASR-affected concrete (prognosis), and for diagnosis as well. It is, however, important to mention that expansion tests on concrete cores extracted from deteriorating concrete structures will often lead to more pessimistic results than that obtained from in-situ monitoring, as the cores are not tested under the same environmental (temperature, humidity, wetting-drying, freezing-thawing, etc.) and stress conditions.

The procedures used for testing cores from ASR-affected structures vary greatly from one study to another (Bérubé et al. 2004). Based on several studies carried out over the past decade, two following test procedures are recommended, which are described in **Appendix F**:

- Expansion test in air at > 95 percent R.H. and 38°C (100°F).
- Expansion test in 1N NaOH solution at 38°C (100°F).

The first test is the most realistic for evaluating the potential for further expansion of ASR-affected concrete. The concrete is tested with its own alkali content and the test conditions used are similar to those used in the Concrete Prism Test (ASTM C 1293). The second test in alkaline solution is recommended for determining the "residual reactivity" of the aggregates present in the concrete under study.

Because of the inherent variability in the extent of ASR in deteriorating concrete members, it is recommended that a set of specimens for expansion testing should consist of a minimum of three cores for cores stored in humid air and two cores for testing in the NaOH solution), minimum 100 mm (4 in) in diameter and 200 mm (8 in) long. One of the main issues with the expansion test on cores at 38°C (100°C) and R.H. > 95 percent is that cores suffer from significant alkali leaching under those test conditions; consequently, the expansion tends to level off after a certain time, which is not really due to consumption of reactive mineral phases or alkalis. The use of larger cores, e.g., 150 mm (6 in) in diameter, could contribute to reducing that effect, as they would be less susceptible to alkali leaching during the test and would likely lead to more reliable results (see **Appendix F**).

5.3.5. *Expansion tests on concrete made with coarse aggregates extracted from cores*

A method of determining the "residual aggregate reactivity" when the reactive phases belong to the coarse aggregates is to extract those aggregates from cores and test them through the standard Concrete Prism Test ASTM C 1293. The procedure for aggregate extraction has been recently revisited and is described in **Appendix G**.

5.3.6. *Water-soluble alkali content of concrete*

The higher the alkali concentration in the pore solution, the higher the OH^- concentration (thus the pH), and the higher the risk for ASR. Therefore, the determination of the above concentrations could be useful for assessing whether or not it is sufficiently high for ASR to be maintained, and may thus provide valuable insights in the process of selecting remedial actions.

As it is difficult to extract pore solution from old concrete, an indirect way of measuring the chemical composition of this solution consists in determining the *water-soluble alkali content* of the concrete by subjecting a representative sample of concrete to hot-water extraction. In the method described by Bérubé et al. (2002a) and presented in **Appendix H**, two 10g subsamples of ground concrete (< # 100-sieve) are placed in boiling water for 10 minutes and the solution left to stand overnight. The solution is then filtered, the volume of solution completed to 100 ml with distilled water and the Na and K concentrations measured through chemical analysis. The alkali and OH^- concentrations in the original pore solution could be calculated from the above values (see **Appendix H**), and expressed on a kg/m^3 Na_2O_{eq} basis. It is important to mention that about 40 percent of the cement alkalis are incorporated in the cement hydrates, thus not extractable through the hot water extraction method. For example, a concrete with a total alkali content of 3 kg/m^3 (5 lb/yd^3) Na_2O_{eq} (e.g., concrete incorporating 300 kg (660 lb) of a cement with 1 percent Na_2O_{eq}) would yield about 1.8 kg/m^3 (3 lb/yd^3) of water-soluble Na_2O_{eq} (i.e., 60 percent x 3 kg/m^3) (i.e., 60 percent x 5 lb/yd^3).

The measurements of the water-soluble-alkali content in concrete can yield interesting information in assessing whether or not the concrete tested contains sufficient alkalis for ASR (diagnosis) or to sustain this reaction (prognosis). However, the results should be used with care as the experimental variability of the method is relatively high, the estimated coefficient of variation being about 20 percent (between 10 and 15 percent when a control concrete is tested in parallel, as recommended).

5.4. Numerical AAR Simulation Modeling

A large number of more or less sophisticated numerical models have been developed for the diagnosis of ASR, the assessment and structural integrity, forecasting of further expansion and long-term stability (prognosis), and prediction of structural responses to various mitigation measures (Léger et al. 1995, CSA 2000). Due to the complexity of the ASR expansion process, any simulation model, as sophisticated as it might be, must be calibrated based on the monitoring data and pertinent information obtained from in-situ and laboratory investigations; for instance:

- Geometry of the concrete member or structure under study (drawings).
- Aggregate reactivity and concrete expansivity (in-situ monitoring, petrography, expansion tests on cores, water-soluble alkali content, etc.).
- Environmental conditions (humidity, temperature) (which can be monitored or assessed).
- Mechanical, physical, and thermal properties of concrete (laboratory testing).
- Internal stress conditions (reinforcement, prestressing, postensioning, loading, confinement, etc.; stresses in concrete and reinforcing steel) (which can be measured in-situ or assessed).

- Location of discontinuities (important cracks) (site inspection).
- Results from in-situ structural investigations (loading capacity, concrete/steel bond, etc.).
- Current rates of dimensional changes and buildup of internal stresses (movements, deformations, crack width, stresses in concrete and reinforcing steel, etc.) (from in-situ monitoring or using indirect procedures).

5.5. Collective Assessment of In-situ and Laboratory Investigations

5.5.1. *Scope*

The collective assessment of all in-situ and laboratory investigations, and from modeling as well, addresses the following subjects:

- ASR expansion attained to date.
- Current expansion rate.
- Potential for further expansion due to ASR.
- Risks relative to structural integrity.
- Risks relative to security of persons and properties.
- Potential for further deterioration due to other mechanisms.
- Decision criteria for application of mitigation/remediation measures.

In the case of reinforced concrete members (e.g., bridges), the potential for further expansion due to ASR will be expressed by the number of years before the reinforcing steel yields (in the direction of lower or lack of restraint) could occur, which requires data on the ASR *expansion attained to date* and the *current expansion rate*. In the case of concrete pavements, the potential for further expansion due to ASR will be expressed by the number of years before the joints could close, which requires information on the current expansion rate and widths of joints.

Decision criteria for application of mitigation measures are proposed in the following sections, which are based on the above time limits and on other critical considerations, such as the risks relative to structural stability and security of persons and properties, and the potential for further deterioration due to other mechanisms. The selection of the particular measures to apply in the particular case under study is discussed in Section 6.0.

5.5.2. *ASR expansion attained to date*

Three methods can be used for estimating the expansion attained to date in ASR-affected concrete members; those are listed hereafter from the most to the least recommended method:

- In-situ monitoring of deformations and movements since construction.
- Modified Stiffness Damage Test (SDT) (see Section 5.3.3 and **Appendix E**).
- Surface Cracking Index (see Section 4.2 and **Appendix B**).

Monitoring since construction – This is the best method for determining the expansion attained to date in ASR-affected concrete members. This is often possible in the case of large concrete dams or civil structures that have been instrumented during, or shortly after construction; however, this was not a common practice and is consequently rarely possible in the case of transportation structures.

<u>Modified Stiffness Damage Test (SDT)</u> – The modified SDT can be used for determining the expansion attained to date by the concrete under study. The test procedure and the determination of the expansion attained to date are described in **Appendix E**.

The results obtained appear more reliable and relatively less variable than those obtained from surface cracking measurements, while not being affected by the outdoor exposure conditions. However, as mentioned in Section 5.3.3, the results were found to be aggregate-dependent, which requires the establishment of a calibration curve suitable to the particular reactive aggregate involved or a similar type. A number of such curves are presented in **Appendix E** for different types of reactive aggregates, and could be used in the absence of more pertinent data. **Appendix E** also describes a methodology for the establishment of calibration curves for the SDT.

It must be mentioned that the information gathered from the modified SDT strictly applies along the axis of the cores tested, which direction, however, is generally the most critical (i.e., the direction along which the expansion is the most important since cores are usually taken perpendicular to the direction of the main reinforcement or restraint). However, the test seems to underestimate expansions in directions perpendicular to prestressing cables or rods (Bérubé et al. 2005).

<u>Cracking Index</u> – As mentioned in Section 4.2, the cracking at the surface of a concrete member reflects differential deformations (expansion or contraction) between the surface and the inner concrete due to various mechanisms such as ASR, sulphate attack, freezing/thawing, and shrinkage.

In the absence of any information from monitoring since construction or from modified SDT's, the expansion attained to date in concrete members where ASR is the main contributing factor to cracking, can be <u>roughly</u> estimated from the measurement of the Cracking Index. In the method presented in Section 4.2 and described in **Appendix B**, measurements along the three or two accessible directions are recommended in order to separately assess the expansion attained to date along each direction. For each direction of measurement, the expansion due to ASR (in the absence of any other mechanism causing surface cracking) is considered equal to the Cracking Index discussed in Section 4.2 and **Appendix B**, i.e., to the sum of the openings of all cracks intersected along the measured lines divided by the total length of these lines.

(Eq.1)

$$\text{ASR expansion to date (\%)} = 100 \times \sum \text{crack widths / total length of measurement lines}$$

It must be emphasized that the measurements must be made on the most exposed concrete sections, i.e., where the conditions (variations in humidity conditions, alkali leaching) are largely contributing to limit ASR-expansion in the near-surface concrete layer. It is thus considered that the expansion obtained from the measurements of surface crack widths, resulting from that differential expansion (inner expansion – surface expansion) is then closer to the actual expansion encountered within the whole mass of the concrete member under investigation.

5.5.3. *Current rate of expansion due to AAR*

Three methods can be used for estimating the current rate of ASR expansion in the concrete member under study; those are listed hereafter from the most to the least recommended method:

- In-situ monitoring of deformation for at least 2 and preferably 3 years (see Section 5.2.3 and **Appendix D**).
- Expansion tests on cores (see Sections 5.3.4 & 5.3.5 and **Appendix F**) combined with determination of the water-soluble alkali content (see Section 5.3.6 and **Appendix H**) and the humidity, temperature, and stress conditions prevailing in the field (Figure 2 and **Appendix I**).
- In-situ monitoring of surface cracking for at least 2 and preferably 3 years (see Section 4.2 and **Appendix B**).

<u>In-situ monitoring</u> – As discussed previously, monitoring the movements and deformations within an AAR-affected concrete member is the best method of determining the current rate of expansion in this member, which can be then extrapolated to the next 5 to 10 years. Because of the variability on the temperature and moisture conditions in the field, in-situ monitoring of expansion and deformation should generally be carried out for a minimum of 2 years, preferably 3 years, in States where large annual temperature variations are observed.

<u>Expansion tests on cores combined or not with the water-soluble alkali content (and considering the humidity, temperature, and stress conditions in the field)</u> – In the absence of reliable monitoring data, the approach proposed by Bérubé et al. (2002b), illustrated in Figure 2 and described in **Appendix I**, can be used for estimating the current rate of expansion in ASR-affected concrete in the field. The method uses the following parameters:

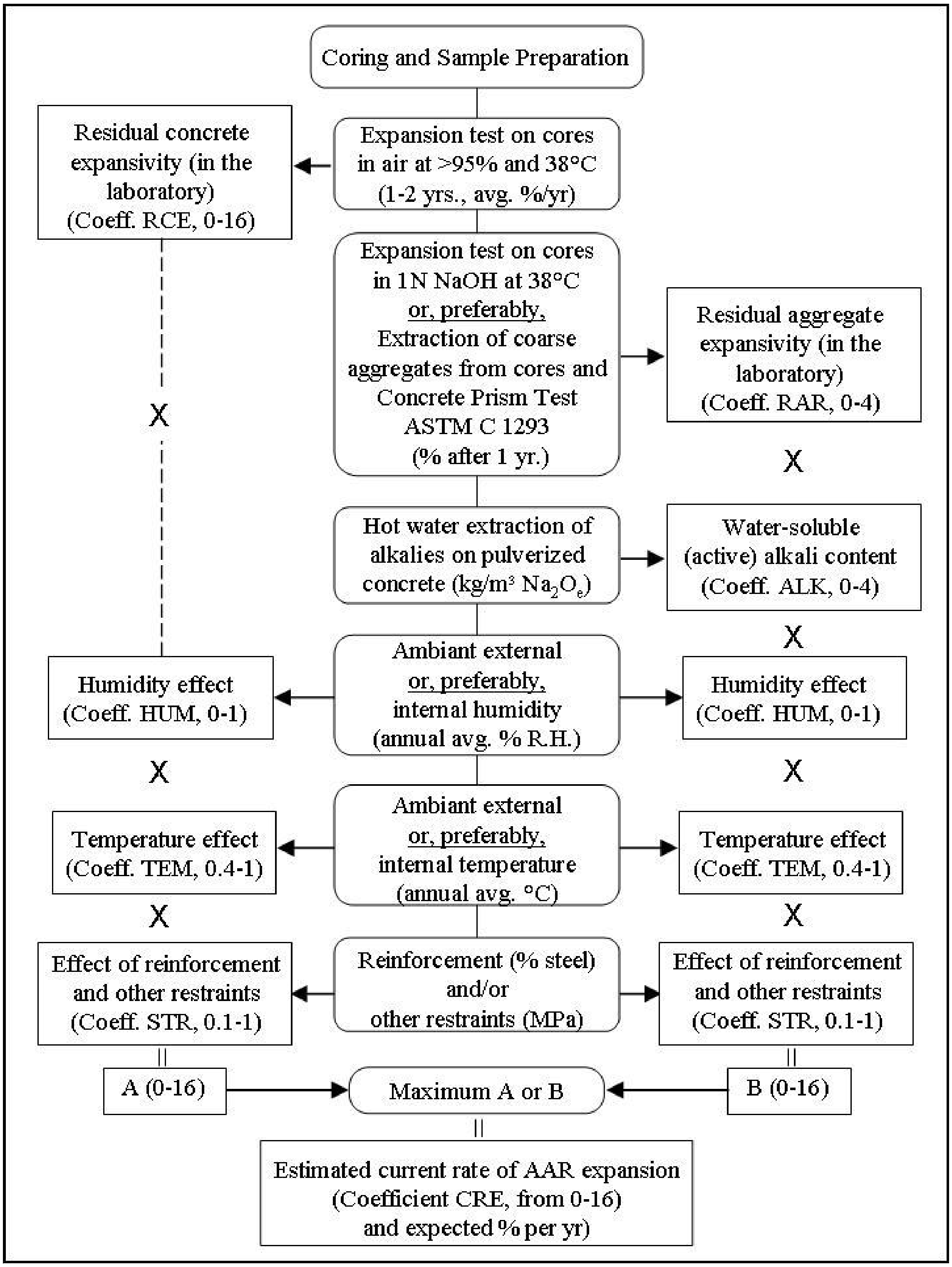

Figure 2. Laboratory assessment of the current rate of AAR expansion in concrete members in-service either already affected by AAR or not. (After Bérubé et al. 2002b).

- The "residual concrete expansivity," which is determined by testing core samples in air at > 95 percent R.H. and 38°C (100°F) (see Section 5.3.4 and **Appendix F**).

- The "residual aggregate reactivity," which is determined by testing core samples in 1N NaOH solution at 38°C (100°F) (see Section 5.3.4 and **Appendix F**) or, when the reactive aggregates belong to the coarse fraction, by testing coarse aggregates extracted from cores through the Concrete Prism Test ASTM C 1293 (see Section 5.3.5 and **Appendix G**).
- The amount of (water-soluble) "active" alkalis in the concrete, which is estimated through the hot-water extraction method described in Section 5.3.6 and **Appendix H.**
- Estimates of the humidity, temperature, and compressive stress conditions (confinement, reinforcement, prestressing, postensioning, loading, etc.) to which the concrete member under investigation is subjected in the field.

The individual risk indices corresponding to each of the above parameters are combined to estimate the current rate of ASR expansion in this member in the field, either already or not yet affected by ASR.

Cracking Index – In the absence of any information from in situ monitoring or from expansion testing on cores, the current rate of expansion in ASR-affected concrete members can be roughly estimated from the periodic measurement of the Cracking Index (see Section 4.2 and **Appendix B**).

5.5.4. *Potential for further expansion due to AAR*

Ultimate ASR expansion in the field – According to the Institution of Structural Engineers (ISE 1992), the maximum ASR expansion that a concrete member may suffer in the field can be obtained from the determination of the maximal expansion of concrete cores tested in humid air (> 95 percent R.H.) at 38°C (100°F). However, such an assumption does not take into account the alkali leaching during the test and the consequent flattening-off of the expansion curve (see Section 5.3.4).

Reinforced concrete members of bridges (delay before steel yielding) – According to the Institution of Structural Engineers (ISE 1992), the most important criterion behind the decision of applying some remedial measures for an ASR-affected reinforced concrete member is its *expansion attained to date*. Indeed, if the steel/concrete bond remains good, which would suggest that concrete and steel reinforcement expansion is similar, one can estimate the condition of the steel with respect to its elastic limit. This limit is considered to be about 0.20 percent expansion in absence of any more relevant information.

Moreover, based on the current rate of expansion, one can estimate the number of years before the reinforcing steel could reach/exceed its elastic limit, thus before the structural integrity could be at risk:

(Eq. 2)

$$\text{Delay before steel yielding (yrs)} = \left[0.20 - \text{Expansion to date (\%)}\right] / \text{Current expansion rate (\%/yr)}$$

The expansion to date and the current expansion rate can be determined using one of the methods proposed in Sections 5.5.2 and 5.5.3, respectively. However, results from cracking measurements on concrete surfaces where the steel/concrete bond may be defective could lead to misleading estimates of expansion to date and /or current expansion rate.

In the case of reinforced concrete members (bridges), the urgency of applying remedial actions will be partly based on criteria related to the delay before steel yielding. More rapid intervention is required in the case of a reinforced member whose reinforcing steel is near yielding conditions, whatever its age is. In any case, it is recommended that further action be taken when the delay before steel yielding is estimated to be less than 5 years, for example by starting an in-situ monitoring program of expansion, with measurements at least on a yearly basis, and/or by performing a structural assessment of the member/ structure in question; it would be appropriate to confirm an assessment that has been based essentially on expansion tests on cores rather than on in situ monitoring.

<u>Concrete pavements (delay before closure of joints</u>*)* – In the case of concrete pavements, the current rate of ASR expansion is a most critical parameter that can be used for estimating the number of years before the expansion joints could close, thus causing spalling at joints :

(Eq. 3)

$$\text{Delay before closure of joints (yrs)} = \frac{\text{Actual opening (lowest value during summer) (mm)}}{\left[\text{Current exp. rate (\%/yr)}/100\right] \times \left[1000 \times \text{length of the two adjacent pavement sections (m)}/2\right]}$$

The current expansion rate can be determined using one of the methods proposed in Section 5.5.3.

In the case of concrete pavements, the urgency of applying some mitigation measures will partly be based on criteria related to the delay before the closure of expansion joints occur. In any case, it is recommended that further action be taken when the delay before joint closure is estimated to be less than 5 years, for example by starting an in-situ monitoring program of expansion, if not done, with measurements at least on a yearly basis; it would be appropriate to confirm an assessment that has been based essentially on expansion tests on cores rather than on in situ monitoring.

5.5.5. *Risks relative to structural stability (reinforced members of bridges)*

The structural assessment of an ASR-affected concrete member/structure requires the knowledge of the current stress conditions and strength of the member/structure. The stress conditions may be determined from in-situ measurements (e.g., compressive stresses in concrete using overcoring techniques and tensile stress determination in reinforcing steel; see section 5.2.4), manual calculations, or numerical modeling. The strength of materials of the structure may be determined based on results from mechanical tests on cores that are adjusted to field conditions, usually using a numerical model. Full-scale load tests can also be used to assess the load carrying capacity of the structure (see Section 5.2.7). The evolution with time of the strength of materials of ASR-affected concrete structures can also be predicted using a numerical ASR model which is populated with the data resulting from all relevant in-situ and laboratory investigations.

ASR-affected concrete may present significant losses in strength and elastic modulus (see Section 5.3.3). However, for the whole reinforced concrete member from which the cores were taken, the structural strength (flexural, compressive, and shear strengths) may remain quite satisfactory if well-anchored three-dimensional reinforcement is present in sufficient amount. Such reinforcement reduces the expansion of the surrounding concrete which can then become "chemically" prestressed. However, the concrete expansion can also result in steel yielding, loss of concrete/steel bond, concrete delamination, with potential weakening of the structural integrity of the concrete member or structure.

The structural assessment of an AAR-affected member or structure must focus on the following aspects:

- Presence or potential for formation of major cracks that could affect the stability of the structure.
- Potential for sliding along horizontal construction joints.
- Potential for bearing or crushing failure of concrete where sharp changes in structural geometry occurs.
- Potential for steel yielding, concrete/steel bond reduction, and concrete delamination.

The decision should be made concerning the application of appropriate remedial measures based on the presence of one or more of the above features, which all constitute a risk for the integrity/stability of the structure and for the security of peoples and vehicles as well.

5.5.6. *Additional risks relative to security of persons and vehicles*

The stability/integrity of an ASR-affected concrete member or structure may be totally safe; however, concrete fragments of various sizes can be detached from portions of the deteriorating pavement or bridge structure, due to various deterioration mechanisms causing spalling, and overall degradation (ASR, corrosion, freezing/thawing, sulfate attack, excessive loading, etc.). The detachment of such fragments would represent a threat to the public and should thus be avoided by taking immediate remedial actions.

5.5.7. *Potential for further deterioration due to other mechanisms*

Concrete bridges and pavements are most often exposed to aggressive conditions which will eventually impair their durability or serviceability. In particular, exposure to chloride (deicing and/or anti-icing chemical, sea water, etc.) may cause concrete degradation, corrosion of reinforcing or prestressing steel and/or concrete spalling. Wetting/drying and freezing/thawing cycles may cause/enhance surface cracking, and deterioration. Freezing/thawing may significantly increase the expansion and the deterioration in ASR-affected concrete due to the presence of preexisting ASR microcracking, despite the presence of an appropriate air void system and even when ASR is almost completed.

When exposed to such aggressive conditions, the potential for further deterioration of ASR-affected concrete members shall be investigated, even if they do not present any significant potential for further expansion due to ASR. A decision shall then be made about the necessity or not to immediately apply some mitigation measures (e.g., penetrating sealer, etc.). According to CSA A23.3[6] ("Design of Concrete Structures"), the crack widths in reinforced concrete for durability should not exceed 0.1 to 0.4 mm. The ISE (1992) considers that cracks in excess of 0.15 mm (0.006 in) in width should justify further investigations; ACI committee 224 also suggests a 0.15 mm (0.006 in) limit criterion for structures exposed to wetting and drying conditions. The decision criterion proposed in this document corresponds to a maximum of 0.15 mm (0.006 in) for the crack width.

5.5.8. *Summary of decision criteria for application of mitigation/remediation measures*

Based on the above discussion, the decision to immediately apply some appropriate mitigation/remediation measures could be taken on the basis of the criteria given in Table 9.

[6]Canadian Standards Association (CSA), "Design of Concrete Structures," CSA A23.3-94, Canadian Standards Association, Mississauga, Ontario, Canada, 2000.

Table 9. Decision criteria for additional investigation or immediate application of appropriate mitigation/remediation measures.

Decision criterion	Based on following investigations	Immediate action(s) recommended
Risk of steel yielding (expansion > 0.2 percent) in 5 years or less (reinforced members of bridges).	• Expansion to date (in-situ monitoring, modified SDT or Cracking Index). • Current rate of expansion (monitoring or expansion tests on cores). • Stress conditions (in-situ measurements in concrete and reinforcing steel).	• Implement monitoring for better assessing the current rate of expansion (minimum of 3 yrs); measure the stress conditions. • Apply appropriate measures of mitigation/remediation (see Section 6.0).
Risk of closure at joints by 5 years or less (unreinforced members of bridges, pavements).	• Current widths of joints. (maximum opening in summer) • Current rate of expansion (monitoring or expansion tests on cores).	• Implement monitoring for better assessing the current rate of expansion. • Apply appropriate measures of mitigation/remediation (see Section 6.0).
Risk relative to structural stability, excluding imminent steel yielding, considered above (reinforced members of bridges).	• Site inspection. • Mechanical tests on cores. • In-situ measurement of stresses in concrete and reinforcing steel. • Structural assessment. • Numerical modeling. • Full-scale loading tests.	• Confirm data, if possible (e.g. measurement of the stress conditions, structural assessment, full-scale tests). • Apply appropriate measures of mitigation/remediation (see Section 6.0).
Additional risk relative to security of persons and vehicles due to detached concrete fragments (bridges and pavements).	• Site inspection.	• Apply appropriate measures of mitigation/remediation (see Section 6.0)
Low potential of further expansion due to AAR but risk for rapid further deterioration due to other mechanisms (corrosion, sulphate attack, spalling, scaling, degradation,…) (bridges and pavements).	• Presence of cracks > 0.15 mm in width. • Aggressive exposure conditions (de-icing salt, sea water, external sources of sulfates, freezing/thawing, etc.).	• Apply appropriate measures of mitigation/remediation (e.g., penetrating sealer etc.) (see Section 6.0).

6.0 Mitigation Measures for AAR-Affected Structures

Report on Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction (Thomas, et al., 2008) is the first of this two-part series of reports, and summarizes the various options for preventing AAR in new construction. The options are numerous, and the end result, namely avoidance of deleterious AAR reaction, has been borne out by years of laboratory and field experience. With this knowledge in hand, there is no reason that concrete structures constructed in the future should suffer from AAR. However, there are literally thousands of structures already in service that were built before this knowledge was applied in practice, and unfortunately, there are still new structures being built that will ultimately suffer from AAR-induced expansion and distress because guidance, such as that contained in the aforementioned report by Thomas, et al. 2008, was not followed. As such, AAR-affected structures will continue to plague our infrastructure for years to come. Unfortunately, the mitigation options available for existing structures suffering from AAR are no where near as numerous as those available for preventing AAR in new concrete, and there is a major shortage of information/data documenting the effectiveness of the mitigation measures that have been implemented in the field. The majority of the work to date on treating existing structures has focused on ASR-affected structures, as opposed to ACR-affected structures, and there are by far many more ASR-affected structures worldwide. As such, the focus of this portion of this report is aimed at ASR-affected structures. However, some of the mitigation measures, particularly those aimed at drying the concrete, would be helpful whether it is ASR or ACR that is impacting the structure.

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

6.1. Decision Factors When Considering Mitigation Options

Once the decision has been made that mitigation or remediation measures are necessary, the owner/agency who has jurisdiction over the structure must carefully consider all available options before deciding upon and implementing the selected measure(s). This section identifies some of the critical decision factors that must be considered during this process. After discussing the decision factors, presented as a series of questions to an owner/agency, the various mitigation options that are available to an owner/agency are described in Section 6.2, and lastly, Section 6.3 describes how the decision factors can be coupled with the available mitigation measures to select the mitigation measure(s) for a given structure.

Figure 3 summarizes the decision factors that should be considered (or questions that should be answered, for that matter) for a given structure that has been deemed in need of immediate mitigation or remediation due to ASR-affected expansion and distress. The relevance of these factors, or questions, may not be obvious at this point, but Sections 6.2 and 6.3 of this report will describe the importance of these factors as they relate to potential mitigation measures. For example, if a reinforced concrete structure is exposed to both chlorides and freeze-thaw cycles, the ingress of water and chlorides must be restricted and maximum crack widths must be closely scrutinized – this situation would thus trigger specific measures aimed at these concerns, such as caulking of cracks and improvements in drainage to limit moisture ingress.

<u>What is the criticality of the structure?</u>

- Can ASR be tolerated?
- What are the consequences should ASR occur?
- What is the intended service life of the structure?

↓

<u>What are the exposure conditions for the structure?</u>

- Is the structure directly exposed to moisture, and if so, is it constant exposure or cyclic exposure?
- Is the structure exposed to freezing and thawing cycles?
- Is the structure reinforced with steel and is it exposed to chlorides?
- Is the structure exposed to an external source of alkalies (deicing salts, etc.)?
- Is the structure exposed to external sulfates or other aggressive ions?

↓

<u>How well-reinforced (or confined) is the structure?</u>

- Is the affected structure internally reinforced and detailed such that the ASR is sufficiently confined in the "cage" (see ISE (1992) for classification of structures based on level of reinforcement detailing)?
- Is the affected structure confined externally (i.e., by adjacent structures, etc.)?
- What specific type of reinforcement is used in the structure (black steel, epoxy-coated steel, prestressed/post-tensioned, etc.)

↓

<u>What are the geometric configuration and accessibility of structure or structural element?</u>

- What are the dimensions, shape, aspect ratio of the structure or structural element (relevant for strengthening/confining by FRP, etc?)
- How accessible is the structure or structural element (relevant for certain mitigation options that need a certain degree of access)
- Is the structure on grade or is there backfill on one more sides?

↓

<u>Are there any serviceability issues due to current or projected ASR damage?</u>

- Is the serviceability affected by ASR (i.e., ride quality for bridge deck, watertightness for containment structure, etc?)
- Is the intended use of the structure jeopardized by ASR-induced damage?
- Are adjacent structures/elements being adversely affected?

↓

<u>Is future expansion anticipated for the structure?</u>

- What is the estimated magnitude (and direction) of future expansion (especially relevant if external confinement is being considered as mitigation option)?

↓

<u>How important is the aesthetics of the structure?</u>

- Is it a "historic" structure?
- Can mitigation options that impact aesthetic nature of structure be tolerated?

Figure 3. Decision factors when considering mitigation option(s) for structure that has been deemed in need of immediate mitigation due to ASR-affected expansion and distress.

6.2. Overview of Mitigation Measures for ASR-Affected Structures

Figure 4 summarizes the various mitigation options that have been used or proposed for use in field structures. These are grouped according to whether they are intended to treat the *causes* of ASR or the *symptoms* of the deleterious reaction. This section will briefly discuss each of the options shown in Figure 4 and will then focus on those that have the greatest potential for effectively treating ASR-affected structures. For each of these options, the merits will be discussed, as well as inherent shortcomings, both in terms of general applicability to field structures and specific application to certain structures.

When attempting to treat the causes of ASR, it is worth noting that three conditions must be satisfied for ASR to occur: 1) Sufficient alkalies must be present within the concrete; 2) sufficient reactive silica within the aggregates must be present in the concrete; and 3) sufficient moisture must be present within the concrete to sustain the reactions. As such, when attempting to mitigate active ASR by treating the underlying causes, actions should be taken to reduce or eliminate the above factors.

TREAT THE CAUSE

Chemical Treatment/Injection
- CO_2
- Lithium Compounds

Drying
- Sealants
- Cladding
- Improved Drainage

TREAT THE SYMPTOM

Crack Filling
- Aesthetics
- Protection (e.g., from Cl^- ingress)

Restraint
- Prevent Expansion
- Strengthen/Stabilize

Relieve Stress
- Saw Cutting/Slot Cutting (accommodate movement)

Figure 4. Potential options for mitigating ASR – treating the cause vs. treating the symptom

With regard to addressing the first of the above conditions, it has been proposed that the alkali content (or pH) could be reduced by injecting ASR-affected concrete with CO_2, which would have the impact of lowering the pH and carbonating the ASR gel (Cavalcanti and Silveira, 1989). However, there are several technical and practical limitations to this method. First, carbonating reinforced concrete can substantially increase the rate of corrosion. Second, injecting gas under high pressure can lead to significant distress in concrete already suffering from microcracking due to ASR. Although it has not been reported in literature, this technique would likely have little impact on ACR as it tends to be driven by a minimal quantity of alkalies, and these alkalies tend to be recycled in the process. Either way, whether it is ASR or ACR, injecting concrete with CO_2 to attempt to reduce the alkali content appears to hold little or no promise in field structures.

There have been several laboratory-based publications related to using lithium compounds to treat concrete already suffering from ASR-induced expansion. Research by Stark et al. (1993), Stokes et al. (2000), and Barborak et al. (2004) have shown that lithium compounds can reduce future expansion of small, ASR-affected concrete specimens in accelerated laboratory tests. Although the mechanism is not fully understood, it is generally believed that lithium compounds enter into the existing gel and change the nature and behavior of the gel from expansive to essentially non-expansive. Because of these positive results in laboratory-based work, there has been considerable interest in treating ASR-affected field structures, especially in recent years under FHWA-funded research (FHWA Project DTFH61-02-C-00097). A detailed review of past field trials using lithium compounds can be found in Folliard et al. (2006), and several field trials are still being monitored under current FHWA projects (East 2007). The most common method of applying lithium compounds in field trials has been via topical application, primarily for pavements (see Figure 5) and bridge decks. There have also been a handful of field trials where lithium was applied either by vacuum or through electrochemical means, both aimed at increasing the depth of penetration of lithium.

Figure 5. Photograph showing topical application of 30 percent-LiNO3 solution to concrete pavement in Idaho.

Although lithium compounds have been found to be effective in laboratory-based research, which has focused on treating small specimens affected by ASR, there is, unfortunately, very little, if any, documentation that lithium is effective in reducing ASR-induced expansion in actual structures in the field. Part of this is due to the general lack of monitoring of field trials in which lithium compounds have been applied to structures (or pavements). However, in recent FHWA-funded field trials (under FHWA Project DTFH61-02-C-00097) in Idaho, Massachusetts, and Texas, which have likely been the most instrumented and monitored lithium-based field trials to date, the depths of penetration of lithium have been measured to be quite minimal, especially for topical applications. In fact, depths of penetration for topical applications in an Idaho pavement were found to be only a few millimeters, with dosages of lithium necessary to suppress expansion measured only down to the first 2 to 3 mm, even after three treatments in heavily cracked pavements. These results are consistent with laboratory evaluations performed under the same project, and when taken as a whole, it appears that due to an inherent lack of penetration, the topical application of lithium compounds shows little, if any, promise of mitigating ASR in structures and pavements, with perhaps the exception where ASR is being exacerbated in the outer surface by an external source of alkalies (e.g., deicing salts). However, research is needed to determine if lithium compounds, applied topically, show merit in this specific case where pavements or bridge decks are exposed to deicing salts that are driving the ASR process. It remains to be seen if topical lithium treatments would be able to reduce future expansion in this situation, given the shear quantities of deicing salt applied annually.

Because of the documented lack of penetration in field and laboratory trials in which lithium compounds have been applied topically, recent focus has shifted towards more aggressive means of driving lithium into ASR-affected concrete, specifically through vacuum impregnation and electrochemical methods. Unfortunately, in research performed under FHWA Project DTFH61-02-C-00097, vacuum impregnation was not found to be effective in the laboratory or in field structures in Texas and Massachusetts. For example, for ASR-affected bridge columns in which lithium nitrate was applied via vacuum, the depths of lithium penetration were found only to be in the present in the outer 9 to 12 mm, drawing into question whether such an elaborative and expensive vacuuming technique is justified. Substantially higher depths of penetration were observed in the same study when lithium nitrate was electrochemically driven into bridge columns, with dosages sufficient to reduce ASR measured all the way down to the reinforcing steel (50 mm from outer surface). However, one "side effect" of the latter process must be addressed. Lithium ions were clearly driven to the reinforcing steel, as was the intention, but because the steel serves as a cathode in the electrochemical process, hydroxyl ions are produced at the surface of the reinforcing steel. To maintain charge neutrality and to offset the production of hydroxyl ions at the reinforcing steel surface, sodium and potassium ions from within the concrete migrated towards the steel surface. This creates an increase in the hydroxyl ion concentration and a subsequent increase in alkali (sodium and potassium) concentration near the surface of the reinforcing steel may exacerbate ASR-induced expansion and cracking in this region. Future monitoring of these columns (expansion, cracking, microstructural evaluations aimed at regions near the concrete/steel interface) should help to determine if the potentially detrimental side effects of electrochemical impregnation outweigh the benefits of the significant lithium penetration. Information on the specific details of the electrochemical method used for this bridge structure can be found in East (2007).

However, one major concern with this technique is that the electrochemical process itself tends to drive alkalies already present in the concrete towards the rebar, which may be a significant obstacle to this technology. It appears that as a whole, this technique is quite powerful in driving external lithium into the concrete, but the rearrangement of internal alkalies and accumulation of sodium and potassium (which in turns leads to an augmented pH near the rebar) is a serious concern that deserves further attention. More work is in progress to evaluate this treatment technique and to quantify the benefits (and downsides) of this approach.

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

Moisture is an essential component of ASR-induced expansion and cracking. Stark (1991) and Pedneault (1996) showed that below a relative humidity (RH) of 80 percent, ASR-induced expansion is significantly reduced or suppressed. As such, any methods that can be applied in the field to reduce the internal RH are worthwhile. This can involve improving drainage for a given structure, for example, by diverting drainage from a bridge deck away from an ASR-affected column. The application of exterior cladding that prevents the ingress of additional moisture may be beneficial, but it should be noted that the moisture already present within the concrete may be sufficient for ASR to remain active, and this fact must be considered when contemplating a cladding as a mitigation measure. A more sound solution with regard to reducing internal moisture is to apply a coating or sealer that prevents external water from penetrating into the concrete, but also allows water vapor from within the concrete to exit, thereby resulting in an overall decrease in the internal relative humidity. A coating or penetrating sealer that will trigger this reduction in internal relative humidity must provide the following characteristics (after CSA864-00):

Be resistant to water absorption.
Penetrate to a measurable depth.
Resist deterioration from ultraviolet (UV) radiation.
Possess long-term stability in an alkaline environment.
Be of long-term stability in an alkaline environment; and
Allow vapor transmission.

Siloxanes and silanes, tend to be most suitable and are, as a result, the most commonly used as mitigation measures for not only ASR, but also to help reduce the ingress of water (to enhance frost resistance) and external chlorides (to reduce the rate of corrosion of reinforcing steel). In recent years, silanes have become the most important and most widely used product for these purposes. There are a variety of silane products available, varying primarily based on the concentration of silane in the specific formulation (ranging from 20 percent to close to 100 percent) and based on the type of carrier with which the silane is combined (either water-based or solvent-based). More stringent restrictions regarding VOC emissions have resulted in more water-based silanes or solvent-based silanes with higher silane contents (and thus lower solvent content and reduced VOCs). Silanes are almost always applied topically (see Figure 6), at a coverage rate similar or slightly higher than that of lithium-based products, and depths of penetration are in the same order of magnitude–a few millimeters. However, unlike lithium compounds, which must penetrate into concrete to reach ASR gel, thereby rendering the gel less expansive, silanes or similar penetrating sealers must only penetrate into the concrete in the outer region to form a functional barrier preventing water from entering but allowing moisture vapor to escape.

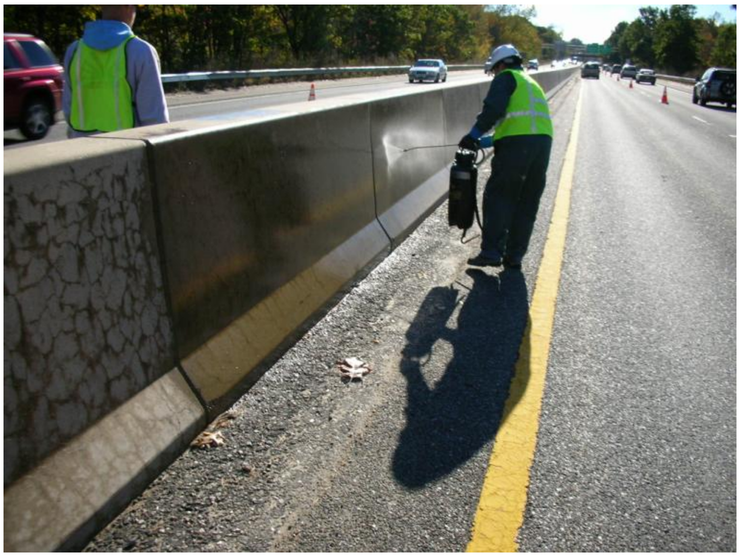

Figure 6. Photograph showing topical application of 40 percent-Silane Solution (solvent-based) to ASR-affected highway barrier in Massachusetts.

Depths of penetration of silane are typically measured to be less than 5 or 6 mm, which is generally adequate for these materials to serve their purpose–preventing liquid water from entering the concrete while allowing internal water vapor to escape, thereby lowering the internal relative humidity of concrete. Deeper penetration of silanes is desirable as it aids in the providing longevity to the treatment as sealers present at or near the surface can be removed with time due to surface abrasion (e.g., bridge decks) or by UV radiation.

Producers and distributors of silanes or similar penetrating sealers generally recommend that the products not be applied when the concrete surface is damp/wet (after rainfall, for example), and some State Highway Agencies, such as New York and Texas, specify maximum surface moisture contents of concrete, as measured by commercially-available moisture meters. A single application of a silane-based product is not generally believed to be a “permanent fix,” as its effectiveness will tend to dissipate with time, especially due to abrasion and UV effects, and as such, it is generally believed that re-applying silanes every 5- years or so is prudent. Carter (1994) reported that the re-application of silane over a surface previously treated with silane resulted in deeper penetrations and enhanced ability to reduce internal moisture. When reapplying silanes or other materials to concrete, manufacturers’ recommendations should be followed to ensure that they are applied properly as there have been some concerns raised in the field on the efficacy of re-applications of silanes or similar products (e.g., water- vs. solvent-based, etc.).

There have been several studies that have confirmed the benefits of applying siloxanes, and especially silanes, to field structures to reduce future ASR-induced expansion (Bérubé et al. 2002a and 2002b; Grabe and Oberholster 2000). The research by Bérubé et al. (2002a and

2002b), illustrated in Figure 7, was particularly encouraging as it showed that applying silane to highway barriers heavily damaged by ASR resulted in a dramatic reduction in future expansion.

Figure 7. Photograph showing benefits of silane treatment of highway barriers in Canada (Bérubé et al. 2002a). The photograph, which shows control section on left and silane-treated section on right, was taken three years after treatment.

As described, silanes can be quite effective in the field in reducing ASR-induced expansion, but there are certainly limitations. For pavements, slabs on grade, wingwalls, or other applications where moisture is available from below (or beneath), silanes will not be as effective as their benefits are only realized from the treated surface. Also, for applications where concrete will be fully submerged or not allowed to dry, silanes will likely not work well because wetting and drying cycles are needed to reduce internal concrete moisture.

Lastly, it should be noted that the application of silanes will not be effective in concrete with large crack widths. For these larger cracks, flexible caulking or similar products should be used to seal the larger cracks. There have been recent developments, including the use of "high-build" paints or elastomeric coatings that may show promise in bridging larger cracks and avoiding the need for caulking of individual cracks. The need to seal larger cracks becomes critical when reinforced concrete is exposed to external chlorides or in regions exposed to cycles of freezing and thawing.

Rather than addressing the underlying cause of ASR, one can attempt to minimize or manage the symptoms or manifestation of the deleterious reaction. In essence, this approach allows ASR to continue but focuses on lessening the impact on the performance or service life of the structure. Following are brief discussions on measures that can be taken to address ASR-induced cracking, primarily through crack filling to minimize ingress of water, chlorides, and other aggressive ions, and ASR-induced expansion, either by confining the expansion or allowing for expansion through slot cutting/concrete removal.

Cracking due to ASR may not only have an impact on the performance of a given structure, but the cracks serve as access points by which water, external alkalies, chlorides, and sulfates can enter the concrete, exacerbating ASR and potentially leading to other forms of distress, such as frost attack, salt scaling, corrosion of reinforcing steel, and sulfate attack. In cases where cracking has become excessive and/or for structures exposed to aggressive environments, crack filling is often the option of choice. In this report, crack width thresholds of 0.15 mm for reinforced members of bridges and 0.30 mm for pavements and non-reinforced members of bridges were proposed, with crack caulking/filling recommended if these limits are exceeded. However, different crack width thresholds can be defined on a structure-by-structure basis, depending on the details of the structure, loading conditions, and surrounding environment.

Flexible grouts or caulking tend to be more effective in filling cracks and keeping water and other entities from entering through the cracks. Although rigid polymer- and cement-based grouts may help to stabilize cracks initially, their rigid nature and strong bonding with the substrate concrete often forces cracks to appear adjacent to the grouted area.

Numerous studies and field trials have shown that physical restraint or confinement (e.g., encapsulation of the affected member by a surrounding non-reactive concrete, applied stress or reinforcement) can significantly reduce deleterious expansion due to ASR in the direction of restraint (Fournier et al. 2004). Because of the unique nature of this mitigation approach and the fact that the structural response is impacted, it is imperative that a structural engineer play the leading role in specifically designing the methodology for a given ASR-affected structure. A detailed structural evaluation is essential, and care must be taken to select and implement this type of mitigation option. Because every structure is different and because a structural engineer is required for this process, no firm guidelines are available herein that can be universally applied. However, some general discussion is provided to shed some light on available options for structural engineers.

Post tensioning in one or two dimensions, or by encasement in conventional reinforced concrete, is currently used as a mean to restore the integrity of the structure; however, it should generally be restricted to relatively small masses of structural concrete because of the huge forces that may result from the expansive process due to ASR (Rotter 1995, CSA 2000). Post-tensioned tendons or cables are considered to be an effective solution for thin arch dams (Singhal and Nuss 1991) or structural members of bridge/highway structures; however, they may be less attractive for large concrete structures because of the necessity of periodic destressing (Rotter 1995). Strengthening by introducing reinforcement with straps, steel plates and tensioning through bolts was also found to be effective in providing containment for selected ASR-affected concrete members (Wood and Angus 1995). Methods to restrain expansion and movement in mass concrete foundations such as tower bases have also included rock anchors and/or encapsulation (Bérubé et al. 1989). Figure 8 illustrates a case in South Africa in which an ASR-affected bridge was remediated by removing damaged concrete, encapsulating the section with new reinforced concrete, and installing prestressing cables through the repaired section.

Care should be taken in designing the encapsulating element because, if sufficient reinforcement is not provided to control stresses due to ASR expansion, the only beneficial effect of encapsulation may be to limit the ingress of moisture (CSA 2000). Strapping or encapsulation of ASR-affected reinforced concrete columns by or with composite materials may be an interesting solution provided that sufficient structural strengthening is assured (Carse 1996).

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

ASR-affected pavements are frequently overlaid, mainly with hot-mix asphalt (HMA) but sometimes with portland cement concrete (PCC). Asphalt overlays will likely have little, if any, positive impact in terms of reducing ASR-induced expansion. In fact, internal moisture can be trapped within (and still available from below), helping to promote ASR, and temperatures within the ASR-affected concrete will tend to increase with the dark color of the HMA layer attracting heat. PCC overlays have been used to a lesser extent than HMA overlays but may show promise for unbounded overlays, where the ASR-affected pavement will not reflect cracking into the overlay.

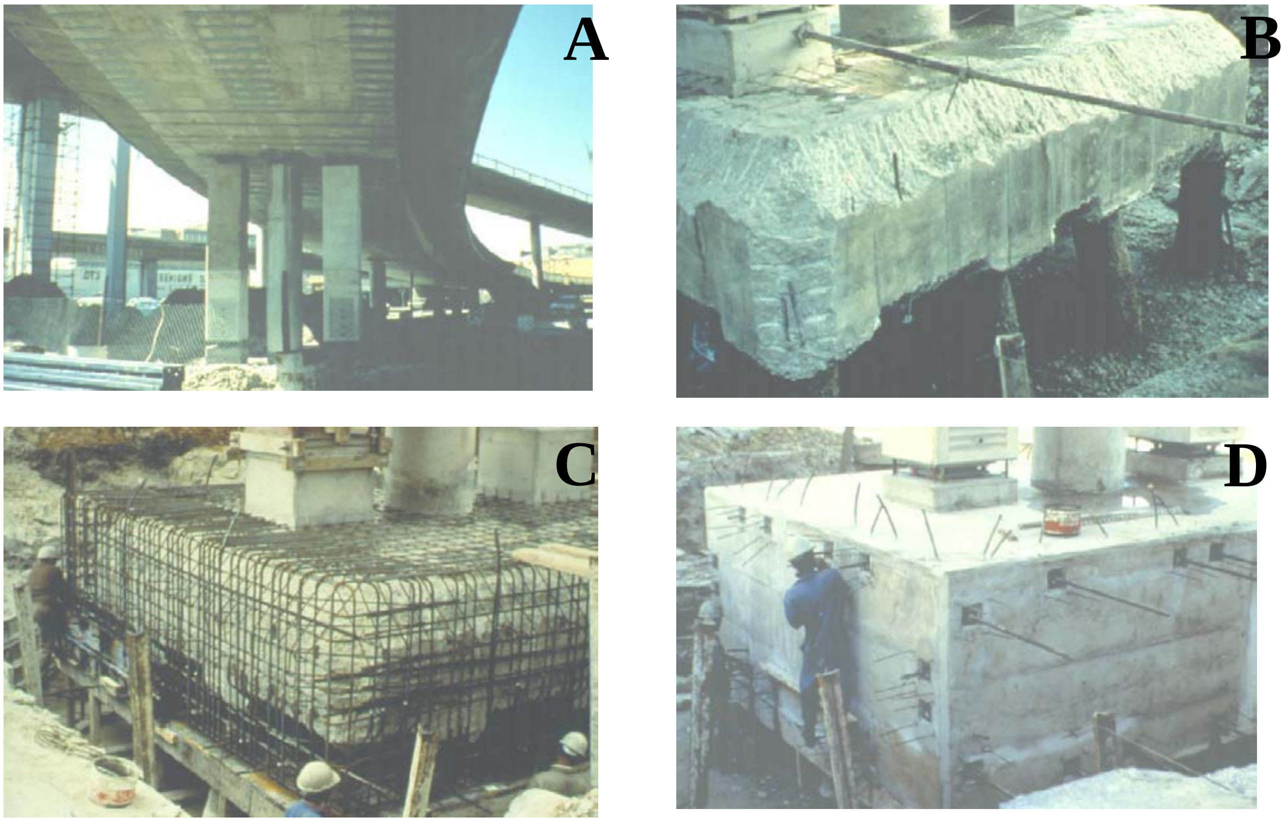

Figure 8. (A) Series of photographs showing strengthening of ASR-affected bridge in South Africa (after Fournier et al. 2004). (B) General view of a highway structure affected by ASR in South Africa. Cracking due to ASR was observed in the pile caps supporting reinforced concrete columns. The cracked concrete was first removed. (C) Additional steel reinforcement was added around the pile cap. (D) External strengthening was provided by means of prestressed cables. (Photos courtesy of R.E. Oberholster, PPC Technical Services, Cleveland, South Africa)

6.3. Selection of Mitigation Measures

This section provides guidance on how an owner/agency can select and implement mitigation options for ASR-affected structures. As mentioned throughout this section, it is not possible at this point in time to develop a definitive, step-by-step methodology for selecting mitigation measures for ASR for several reasons, including:

1. There are few documented case studies, accompanied by adequate monitoring, to quantify accurately the benefits of various mitigation options.
2. Some of the available technologies are still in their infancy and can only be considered experimental at this point in time.
3. Every structure is different, and as such, there are no “one size fits all” approaches that will work across the gamut of structures being used in transportation applications.
4. When ASR-induced expansion has affected the structural integrity of a structure and/or when strengthening or confinement are being considered, a structural engineer is required

to design and implement the selected option. As such, it is not possible to provide broad guidance on such complex situations.

Having stated the above challenges, this section will attempt to provide a framework by which an owner/agency can make an informed decision on potential mitigation measure(s) for a given structure. The ultimate decision on what option to select, if any, will be the responsibility of the owner/agency, and this decision may be affected by many factors, including technical, economic, and others. Lastly, it should be reiterated that none of the options described herein are expected to be permanent in their effects, but rather they should realistically be viewed as methods of extending the service life of a structure. As such, selecting and implementing mitigation option(s) is just one step in the process. Monitoring of the performance of the mitigated structure is essential in terms of quantifying the efficacy of the treatment and in terms of planning future actions.

Figure 9 illustrates the overall approach to selecting mitigation measure(s) for a given ASR-affected structure. This approach combines an integration of information gathered under Section 6.1 (decision factors for selecting mitigation options) and Section 6.2 (discussion on available mitigation options) to select and implement the mitigation measure(s). The figure also illustrates the critical role of monitoring (discussed briefly in Section 6.4) whatever mitigation measure is implemented, and to evaluate the output from the monitoring program as part of the future management of the structure.

To aid in linking the structure-specific decision factors to the selecting of mitigation measure(s), Table 10 is provided, which highlights the key features of the various mitigation option(s) as they relate to specific applications in transportation structures. The output from this overall approach may actually be a combination of mitigation measures, which can then be evaluated for economic and practical feasibility. Certain mitigation options, such as attempting to improve drainage or increasing focus on routine maintenance of the structure, are universally recommended for all ASR-affected structures as the benefits are high and the costs are typically low. Other mitigation options, such as external confinement or strengthening, are reserved for more extreme cases and only when performed under the supervision of a qualified structural engineer. Lastly, some potential mitigation measures, such as CO_2 injection, are not included in the table as they are not deemed to be viable from a technical, safety, or practical perspective.

Table 10 also highlights some of the gaps in our current understanding of certain mitigation measures. For example, there have been very few applications of electrochemical methods in driving lithium into ASR-affected structures. The results of an ongoing field study in Houston (under FHWA Project DTFH61-02-C-00097) are quite encouraging in terms of lithium penetration, as described earlier, but concerns have also been raised because sodium and potassium already present in the concrete have been drawn to the reinforcing steel, potentially increasing the potential for ASR in this locality. This table, in essence, is a work in progress, and it is hoped that field trials conducted in the future will be well documented and monitored; helping to close the gaps in our understanding and to provide more definitive guidance in future reports.

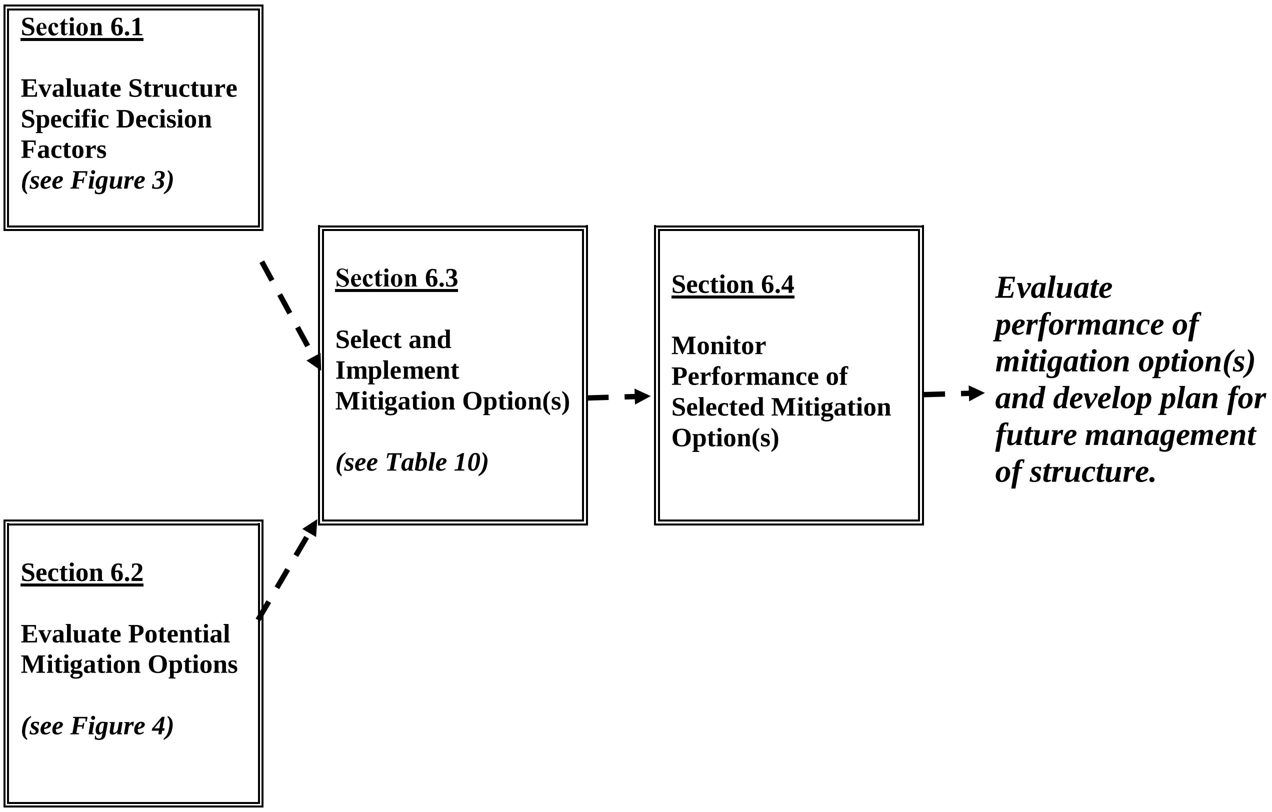

Figure 9. Global flow chart for selecting, implementing, and monitoring mitigation measures for ASR-affected structures.

Table 10. Summary of mitigation options and applicability to transportation structures

MITIGATION MEASURE	APPLICABILITY TO SPECIFIC TRANSPORTATION STRUCTURE	POSITIVE ATTRIBUTES OF MITIGATION MEASURE	NEGATIVE ATTRIBUTES OF MITIGATION MEASURE	OTHER RELEVANT INFORMATION
Improved drainage and enhanced maintenance	All structures benefit from less contact with water. Benefits are most obvious where drainage problems are most pronounced.	Water is essential to ASR process – lowering RH below 80 percent leads to suppression of ASR-induced expansion.	May not be as effective when source of moisture is from below (e.g., pavement) or from behind (wing wall in bridge structure).	Should be included in overall management strategy for ASR-affected structure, due to high benefit/cost ratio.
Application of penetrating sealers (silanes, etc.)	Most applicable to bridge structures, highway barriers, etc.	Proven to reduce internal RH in laboratory and field trials. Best when element is easily accessible (e.g., highway barrier) and is not in direct contact with water or saturated soil.	Benefits may not be observed when element is directly or permanently exposed to moisture (sealers need wet/dry cycles for internal RH to decrease).	Must apply to dry surface. Typically need to re-apply every five years, sooner when surface is subject to abrasion or UV radiation. Sealer must be breathable.
Application of cladding	Applicable to certain bridge elements.	Can be effective in reducing ASR, provided that concrete below cladding is not saturated and able to sustain ASR.	Can trap in moisture. Also, it is difficult to inspect element after cladding is placed on top of it.	Should take measures to dry out concrete before applying cladding layer.
Application of lithium compounds	Applicable to certain bridge elements and pavements.	Lithium has been shown in laboratory studies to suppress ASR in small specimens. Electrochemical methods appear effective in increasing depth of penetration.	Effectiveness in topical applications is minimal, due to lack of penetration. May be helpful if ASR is exacerbated at surface due to external source of alkalies (yet to be proven). Electrochemical methods cause K^+ and Na^+ to migrate to steel, possibly exacerbating ASR in this region. Minimal depths of penetration when applied with vacuum.	Although optimistic results have been found in laboratory, this technology remains experimental in field applications, due to lack of monitoring/documentation proving its long-term efficacy. Ongoing research under FHWA funding should help to quantify the effects of lithium treatment on transportation structures.
Crack filling	Applicable to most structures.	Flexible caulking or crack fillers work best (as opposed to rigid polymer- or cement-based products). Can be effective in reducing ingress of water and Cl.	Only provides benefit in slowing down ingress of water, chlorides, etc. Does not increase structural integrity or restore mechanical properties.	Flexible caulking is especially beneficial when crack widths are large and structure is still expanding.
Application of restraint to confine/ strengthen structure	Most applicable to columns (especially circular).	Applying sufficient confinement can help to manage stresses generated by ASR. Can use FRP, internal/external reinforcement, etc.	Difficult to confine many structural elements (e.g., square columns). Qualified structural engineer required to design and implement.	Qualified structural engineer must design and implement selected technique, and he/she must monitor subsequent strains to ensure that mitigation is effective and safe.
Saw cutting/slot cutting	Most applicable to pavements and bridge decks (at joints).	Can help to accommodate stresses and joint-related failures.	Does not address underlying causes of ASR, and in fact, allows it to continue unimpeded.	Must ensure proper joint details (dowel bars, opening, etc.) when removing concrete near joints of pavements or bridge decks.

6.4. Monitoring of Structures After Mitigation/Remediation

It is absolutely imperative that structures in which mitigation measures are applied be monitored to quantify the long-term effects on the future progression of ASR. It is the current lack of this type of data and information that makes it so difficult to provide definitive guidance on appropriate mitigation measures for ASR-affected structures. Detailed information on methods to instrument and monitor structures affected by ASR is contained elsewhere in this report and is not repeated herein, for conciseness. However, the type of information and data needed includes:

- **Rates of expansion-**using imbedded gauge studs and appropriate gauge (e.g., DEMEC).
- **Cracking-**using cracking index (CI) approach.
- **Relative humidity**-using portable humidity meters and imbedded measurement ports, or equivalent method. For mitigation measures aimed at reducing internal RH, it is important to establish baseline RH values prior to application of mitigation measures, thereby enabling one to quantify subsequent reductions in internal moisture.
- **Petrographic evaluations-**including damage rating index to track the progression of internal distress after application of mitigation measure.
- **Visual inspections**-including photographs taken regularly to track overall appearance of structure and specific manifestation of cracking, etc.
- **Strain measurements on external confinement-**specifically when FRP wraps are applied to attempt to confine expansion due to ASR. Data can be used to determine if excessive expansions are developing and nearing limits of confining wrap.

7.0 Concluding Remarks

This document described an approach for the diagnosis and prognosis of alkali-aggregate reactivity in transportation structures. A preliminary investigation program is first proposed to allow for the early detection of ASR; the program involves a condition survey aiming at detecting typical visual symptoms of ASR in the structures, along with a quantitative assessment of the extent of cracking in those structural members most susceptible to ASR. The assessment (diagnosis) of ASR is then completed by a sampling program followed by the petrographic examination of a limited number of cores collected from selected structural members.

In the case of structures showing evidence of ASR that justifies further investigations, an integrated approach involving the quantification of the contribution of critical parameters with regards to ASR. For instance, the expansion attained to date by the concrete, the current expansion rate and the potential for future expansion, including the temperature and humidity conditions at the site, the water soluble alkali content in the concrete and the restraint conditions in the structural member, is proposed. It is generally recognized that the potential for future distress in ASR-affected members is best evaluated through a fairly extensive in-situ monitoring program; however, such a program generally requires years to generate reliable data. Consequently, the approach proposed in this report analyses the results from a series of selected laboratory investigations, which are used to provide the best estimate possible of the current condition and future behavior of the structure under study, thus allowing one to select reasonable remedial/mitigation actions (summarized in Section 6.0).

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9.0 Appendices

Appendix A
Diagnosis of Alkali-Silica Reaction (ASR)
Visual Signs of ASR

A.1 Introductory Remarks

Symptoms of ASR affecting concrete structures generally consist of (1) expansion causing deformation, relative movement and displacement, (2) cracking, (3) surface discoloration, (4) gel exudations and, occasionally, (5) pop-outs; however, the presence of one or many of these features is not necessarily an indication that ASR is the main factor responsible for the damage or distress observed.

A.2 Expansion Causing Movements and Deformations

The extent of ASR often varies between or within the various members/parts of an affected concrete structure, thus causing distresses such as relative movement of adjacent concrete members or structural units, deflection, closure of joints with associated squeezing/extrusion of sealing materials and, ultimately, spalling of concrete at joints. There are a number of reasons that can explain such a variation, including:

- Variations in the proportioning and composition of the concrete [cement and alkali contents, nature and proportion of aggregate material (e.g. concrete provided by different suppliers and incorporating aggregates of different reactivity levels, cements of different alkali contents, etc.), localized cement-to-aggregates ratio, etc.].
- Variations in the exposure conditions and loading regimes between and within the various parts/members of the structure.
- Differences in the size and geometry of different parts/members of the structures.
- Variations in the reaction rates between different parts of the structure because of differences in the alkali concentration through alkali migration processes, leaching, dilution, external applications of salts, etc.
- Variations in the structural restraints within (reinforcement detailing density and design, prestressing, postensioning, etc.) and between (confinement, etc.) the different parts/members of the structure.
- Variations in the age of the different structural members (e.g. structure built in different stages).

Staff performing condition surveys should be aware of the factors listed above and of their potential impact as just described so visual evidence of the presence of such conditions should be noted as part of the condition survey reports. However, it is important to remember that deformations in concrete structures may be caused by a range of different mechanisms, such as loading, thermal or moisture movements, differential shrinkage, gravity and foundation effects, hydraulic pressure, creep, impact, and vibrations (BCA 1992).

Examples of expansion causing deformation, relative movements and spalling in transportation structures are illustrated in Figure A1.

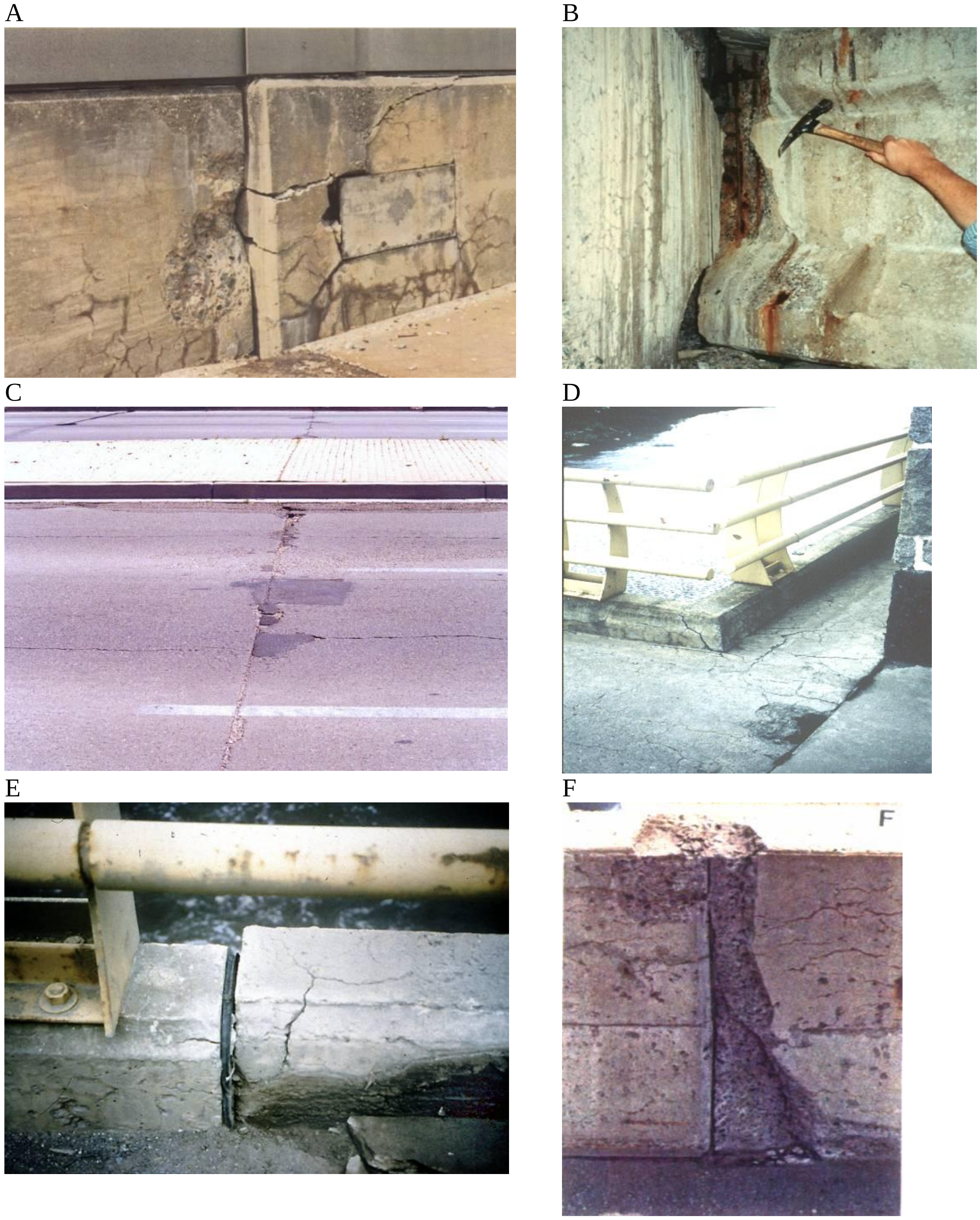


Figure A1. (A). Relative movement of abutting sections of parapet wall in a bridge structure affected by ASR (Stark 1991). (B). Expansion of bridge girder leading to loss of clearance between the girder and embankment and eventually crushing of the girder end with localized spalling. (C). Expansion causing spalling at joints in a concrete pavement incorporating a highly-reactive aggregate; also noted longitudinal cracking in the middle part of the pavement sections. (D). Relative movement between a pier block showing ASR cracking and an adjacent deck slab causing spalling of concrete and extrusion of sealing material along the joint (CSA 2000). (E). Relative movements between pier blocks of a concrete bridge structure affected by ASR (CSA 2000) (F). Expansion with associated severe spalling in abutting jersey barrier sections affected by ASR.

A.3 Cracking

The pattern of cracking due to ASR is influenced by factors such as the shape or geometry of the concrete member, the environmental conditions, the presence and arrangement of reinforcement, and the load or stress fields (restraint) applied to the concrete.

Cracking is usually most severe in areas of structures where the concrete has a constantly renewable supply of moisture, such as close to the waterline in piers, from the ground behind retaining walls, beneath pavements slabs, or by wick action in piers or columns. Concrete members undergoing ASR and experiencing cyclic exposure to sun, rain and wind, or portions of concrete piles in tidal zones often show severe surface cracking resulting from induced tension cracking in the "less expansive" (due to alkali leaching/dilution processes, variable humidity conditions, etc.) surface layer under the expansive thrust of the inner concrete core (Stark and Depuy 1987, ACI 1998). Cracking due to ASR will preferentially (or more rapidly) develop in the exposed portions of concrete structures (Figure A2-A).

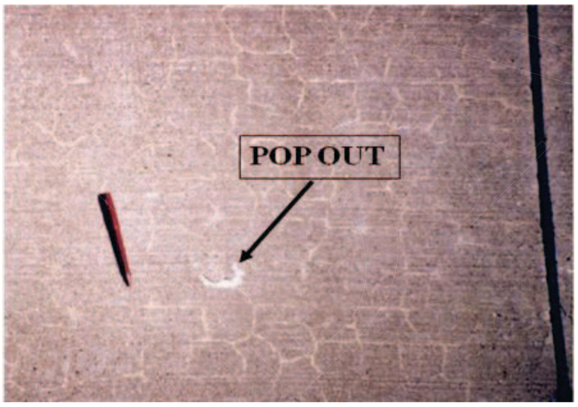
A

B

C

D

E



F

Figure A2. (A). Bent cap in relatively new bridge structure showing signs of ASR, primarily where direct access to moisture/rain prevails. (B). ASR cracking in drilled shaft supporting high-mast illumination pole. (C). Gel staining around cracks in the parapet wall of a bridge structure affected by ASR. (D). Pop-out created by the expansion of a frost-susceptible porous coarse aggregate particle. (E). ASR-induced pop-out in a concrete pavement incorporating highly-reactive aggregates; also noted pattern cracking (F). Efflorescence and exudations of alkali-silica gel at the surface of the concrete foundation of 25-year-old highway bridge affected by ASR.

"Map" or "pattern" cracking is often associated with, but not exclusive to, ASR (Figure A3-A to A3-C); it is often observed in AAR-affected concrete members free of major stress or restraint. Drying shrinkage, freezing/thawing cycles and sulfate attack can also result in a pattern of cracks showing a random orientation. In reinforced concrete members, or under stress and loading conditions, the ASR cracking pattern will generally reflect the arrangement of the underlying steel or the direction of the major stress fields. For instance, in pavements and slabs on grade, ASR cracking usually develops perpendicular to transverse joints and parallel to free edges along the roadside (Figure A3-E), and against the asphalt pavements where is less restraint; these cracks often progress to a map pattern (Figure A3-E). Longitudinal cracking is often observed in reinforced concrete decks, columns and beams affected by AAR (Figure A2-B, A3-D, A3-F, A4-A to A4-D). Concrete members affected by ASR may show more than one pattern of cracking at a time; common associations are predominant longitudinal cracks interconnected by a finer, randomly oriented, cracking pattern (Figure A3-E).

Surface macrocracking due to ASR rarely penetrates more than 25 (approximately 1 inch) to 50 mm (approximately 2 inches) of the exposed surface (in rare cases reaching depths >100 mm or 4 inches) where they convert into microcracks. The width of surface macrocracks generally varies from 0.05 mm (approximately 0.002 inch) to 10 mm (approximately 0.40 inch) in extreme cases. The measurements of crack widths on deteriorating concrete members can be used to monitor the progress of damage between visual condition surveys. This is discussed in further details in the Appendix B of the document.

Cracking will develop in concrete members wherever the tensile strain from the combined effects of internal expansive or shrinkage mechanisms, structural loads and reinforcement restraints exceed the tensile strength of the concrete (ISE 1992). Improper mixture proportioning, poor workmanship or inadequate curing may also cause concrete to crack.

A.4 Surface Cracking

Cracks caused by ASR are often bordered by a broad brownish zone, giving the appearance of permanent dampness (Figure A2-C, A3-B). Sections of concrete members that are badly damaged may develop a patchy surface staining; however, this is not necessarily an indication of ASR.

A.5 Pop-Outs

The expansion of individual unsound or frost-susceptible aggregate particles [such as laminated, schistose and argillaceous, clayey or porous particles or certain varieties (porous) of chert, ironstones] at or near the concrete surface due to frost-action is likely to be the main factor for the development of pop-outs in northern countries (Figure A2-D). Pop-outs can also be caused by a poor bond between the cement paste and dusty coarse aggregate particles.

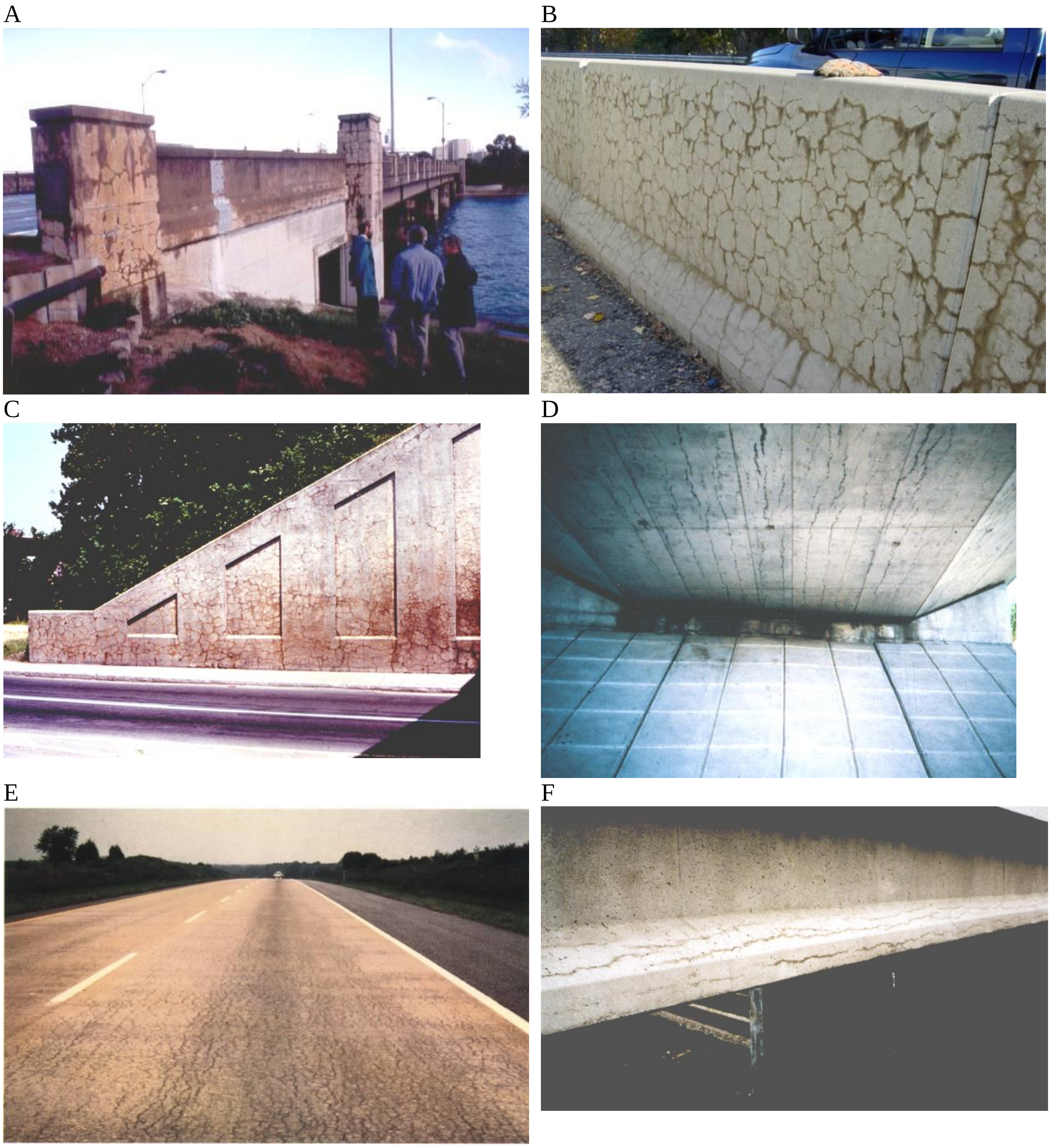


Figure A3. (A). Map-cracking in parapet walls of a bridge structure affected by ASR. (B). Severe map-cracking and associated gel staining around cracks in a median highway barrier affected by ASR. (C). Severe map-cracking in the wing wall of a 30-year-old bridge structure affected by ASR (CSA 2000). (D). Longitudinal cracking on the deck soffit of a 20-year-old highway bridge affected by ASR (CSA 2000). (E). Well-defined crack pattern associated with the development of ASR in highway pavement; the orientation of predominant cracks is longitudinal, while map- or pattern-cracking is also identified. (F). Longitudinal cracking in a precast, reinforced concrete beam affected by ASR. The edge beams, i.e., those exposed to the action of wetting and drying, typically show a more advanced stage of ASR deterioration than the interior beams.

A

B

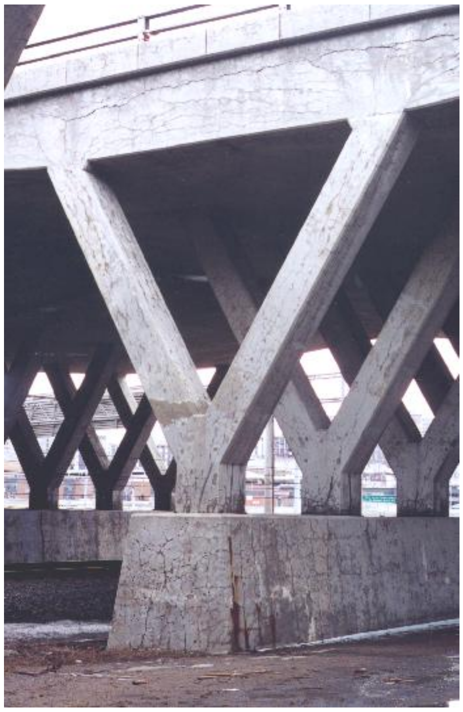


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Figure A4. (A). Fine longitudinal cracking in a reinforced concrete column of a 20-year-old highway bridge structure affected by ASR. (B). Longitudinal cracking on the edge of the deck and in the column of a 20-year-old highway bridge affected by ASR. The presence of reinforcement and related stress fields has influenced the pattern of cracking. (C). Pattern-cracking associated with main longitudinal cracking along the prestressed cable in a reinforced concrete beam of a bridge structure affected by ASR. (D). Cracking in the beam (upper part) and the reinforced concrete column of a bridge structure affected by ASR.

Alkali-silica reactive aggregates undergoing expansion near the concrete surface may induce the detachment of a portion of the skin of concrete leaving the reactive aggregate in the bottom (Figure A2-E). Any particular visual observations that could help understand the expansive process observed, such as traces of rust (reacting ironstone), alkali-silica gel (reactive aggregate), laminated aggregate particle in the bottom part of the "crater" (could suggest frost action), should be noted.

A.6 Surface Deposits (Gel Exudation vs. Efflorescence)

Although surface gel exudation is a common and characteristic feature of ASR, the presence of surface deposits is not necessarily indicative of ASR as other mechanisms (such as frost action or the movement of water through cracked concrete members) can also cause surface deposits called efflorescence (without the present of ASR gel) (Figure A2-F). It is good practice during the condition survey to record the extent and location of surface deposits along with their color, texture, dampness, and hardness. A sample of the surface deposit can be taken and submitted to and/or X-ray analysis to help determine if ASR gel is present.

A field test to detect the presence of ASR silica gel by using uranyl acetate fluorescence was developed under the SHRP program in the United States (D. Stark 1991, Natesaiyer, et al.1991). Care should be taken in interpreting the results (see ASTM C 856).

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Appendix B
Diagnosis of Alkali-Silica Reaction (ASR)
The Cracking Index (CI) Method

B.1 Introductory Remarks

The Cracking Index (CI) is obtained from a crack mapping process that consists in the measurement and summation of crack widths along a set of lines drawn perpendicularly (i.e., parallel and perpendicular to the main restraint(s)) on the surface of the concrete member investigated (Figure B1). The purpose of the method is to quantify the state of cracking on an area of the concrete member. It is applied to structures exhibiting symptoms of internal deterioration of the concrete (multidirectional cracking, map cracking), especially those suffering from ASR.

When carried out for the first time, the Cracking Index will establish a baseline of the extent of cracking in a structural member, while allowing comparative ratings between structural members. Periodic measurements of the CI will generate data on the evolution of deterioration and could thus be used to establish the frequency of monitoring (based on estimate of the expansion rates) and further actions to be taken. Structures that exhibit crack widths in excess of those tolerable may at some point be subjected to more detailed investigations, which may include a structural analysis to determine their integrity.

B.2 Material and Equipment Required

The method is simple to use and require the following basic material/equipments:

- Template for drawing the required axes.
- Pocket size crack comparator (small transparent ruler with width marks ranging from 0.10 to 2 mm) (0.004 to 0.08 in), or graduated micrometer magnifying lens (magnifying power 10 to 20 X, micrometer graduated from 0 to 10/20 mm (0 to 0.4/0.8 in) by increment of 0.05/0.10 mm (0.002 to 0.004 in)).
- Photo camera.
- Drawing material (marker) suitable for concrete and resistant to the environment of the structure (humidity, UV).
- Stainless steel bolts or gage studs with a machined “demec point” at the end and installation equipment required (yard/meter stick, cordless drill, rapid-set cement or epoxy glue, etc.) for marking the corner of the reference grid.

B.3 Laying Out of the Reference Grid

The location(s) of the reference grid(s) is to be chosen so as to represent the cracking pattern present on the structure (or parts of it). In choosing the grid location, one also has to take into account the main cracking system, the shape of structural member, the access ways, the need to insure the reference grid's integrity by protecting it from the potential aggressions of the environment and from vandalism.

The reference grid is made of four axes graduated in tenth of a meter (0.1 m) (4 in): two parallel vertical axis and two parallel horizontal axis of same length (or two axis parallel and two perpendicular to the main restraints in the case of reinforced concrete members) (Figure B1). The laying out and drawing of the reference grid is to be done with caution and with care not to alter the openings of the cracks. The concrete member can be cleaned using appropriate means prior to the drawing of the reference grid. The axes are drawn on the concrete member with a template or a meter (yard) stick (Figure B2-A). It is highly recommended that demec points be set in the concrete in the corner part of the reference grid (Figure B2-A). This has two objectives: 1) ensure that the CI readings be repeated at the exact same location during subsequent field operations, and 2) allow in-situ expansion measurements.

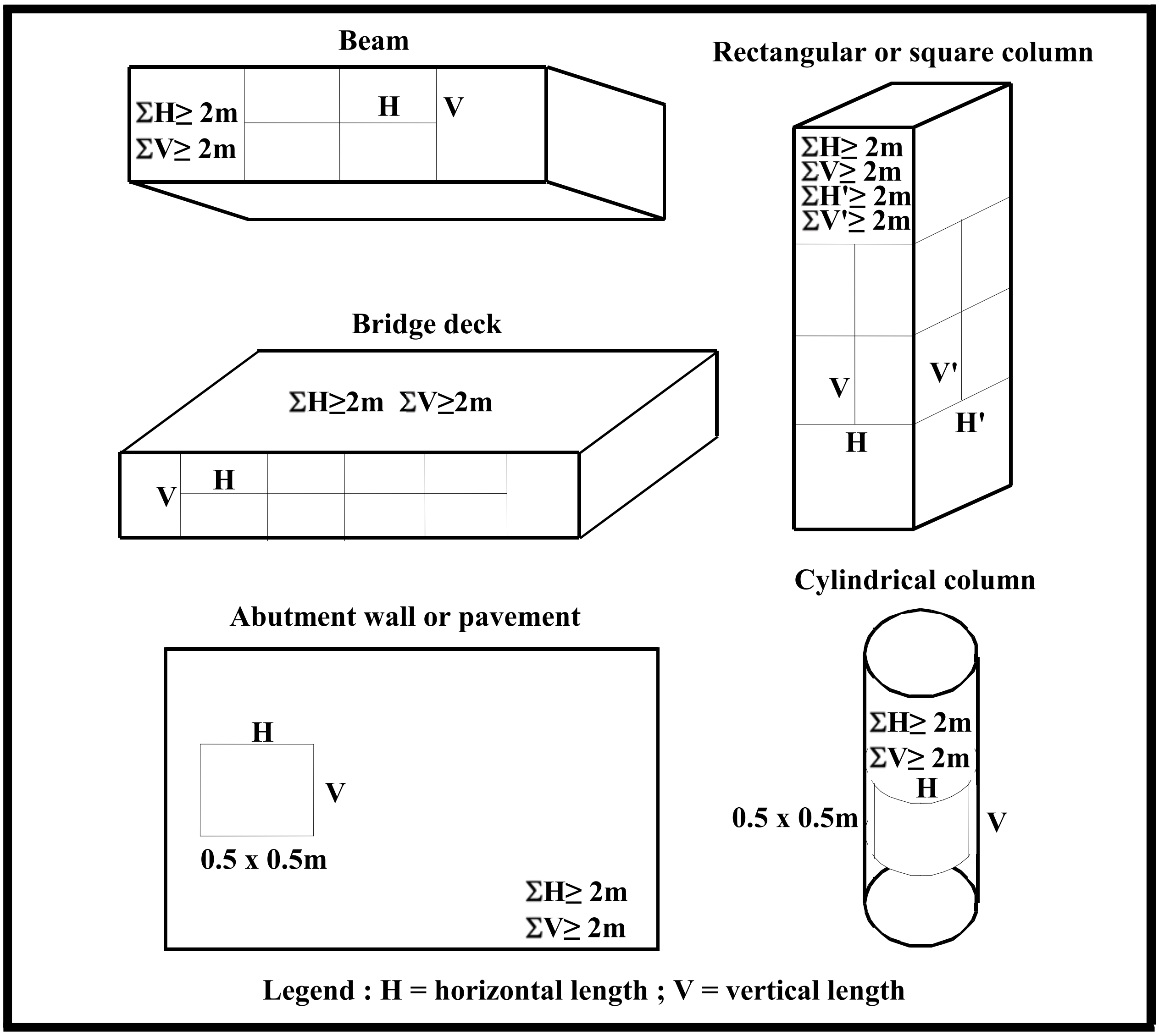


Figure B1. Sketches showing the proposed location of reference grids for cracking index measurements. The number of horizontal and vertical measuring lines should be such that a total of 2 meters of line-measurements should be done in each direction. (Corresponding dimensions in the figure above are as follows: 0.5 m = 20 in, 2 m = 80 in.

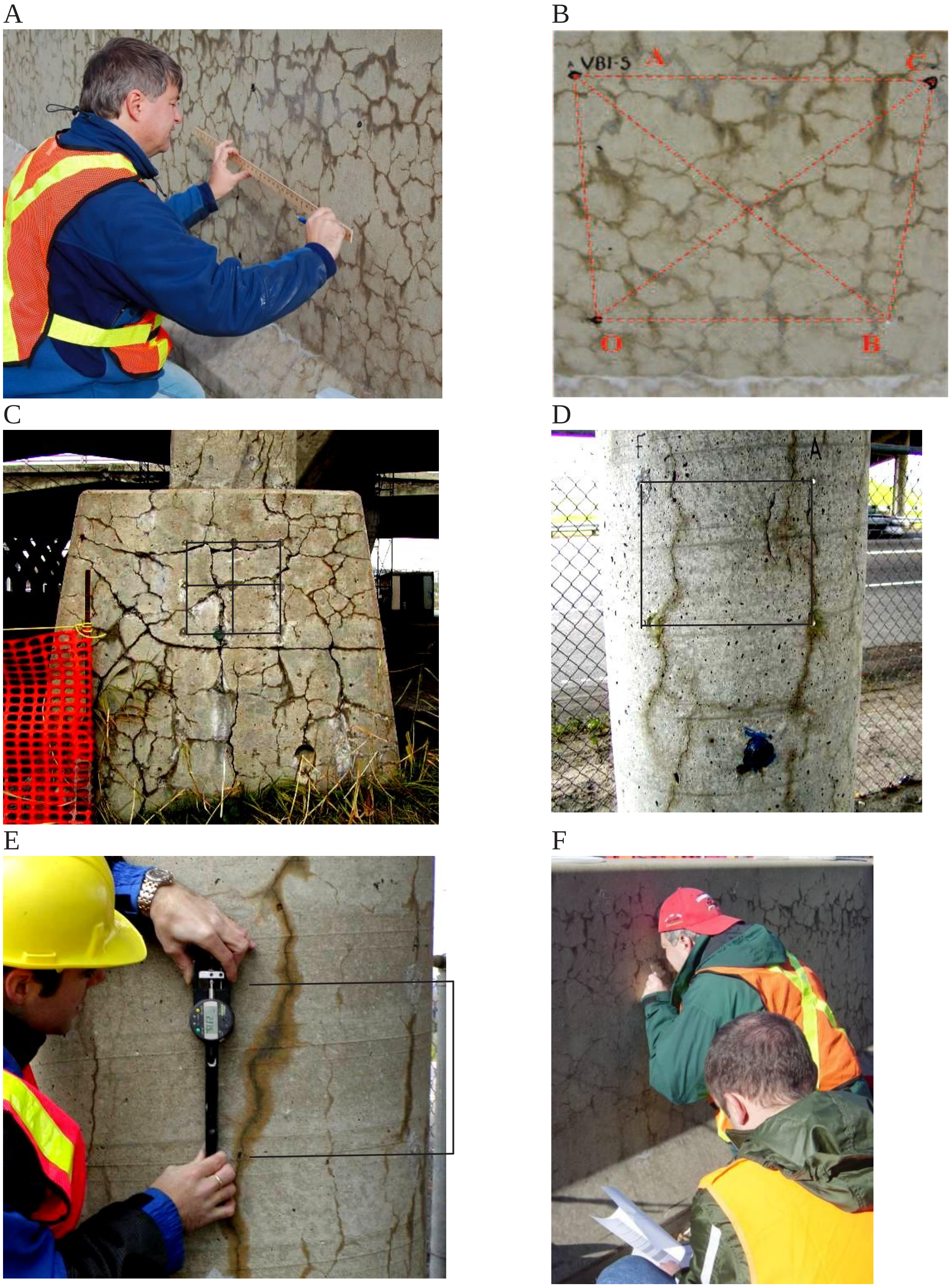


Figure B2. (A and B). Laying out of the reference grid on a concrete median barrier affected by ASR. (C). Four small reference grids are drawn on the exposed surface of the foundation block of reinforced concrete columns of a highway bridge structure affected by ASR. (D). Reference grid on the exposed surface of a reinforced concrete column of a highway bridge structure affected by ASR. (E). Expansion measurement at the surface of the above concrete column. (F). Cracking Index measurements.

The size of the reference grid will depend on the size of the structural member as well as on the gauge length of the length-change measuring device available. Ideally, 0.50 m (40 in) long gauge lengths will be used and thus 0.50 m (20 in) reference grids will be drawn on the member (Figure B1 and B2-B). Smaller (e.g., 0.20 m (8 in) long) lengths could also be used for smaller members or as a function of the length comparator available (Figure B2-C to B2-E). In order to generate a statistically representative assessment of the extent of cracking through the CI method, it is recommended that a minimum of two CI reference grids, 0.5 m (20 in) in size, be drawn on the surface of the member to be evaluated. This will provide a total of eight, 0.50 m (20 in)-long measuring lines (or a total of 2 m (80 in) of measuring lines along each direction). A larger number of smaller-size reference grids will however be required to provide the same statistical evaluation of the cracking system.

The demec points to be installed in the corner of the reference grid consist of approximately 5-7 mm (0.2-0.3 in) diameter by approximately 15-20 mm (0.6-0.8 in)-long stainless steel bolt or gage studs with a machined “demec point” at the end (see Appendix D). In order to insert the gage studs into the concrete, holes are dry drilled into the concrete member. The holes are just large enough to insert the gage stud fairly tight with rapid-set cement or epoxy glue. The head of the gage stud should appear flush with the surface of the concrete member.

B.4 Selection of Structural Members

Cracking Index measurements should be done on the member(s) showing the most severe degree of deterioration and/or on the most critical structural members of the structure that show signs of cracking. For each type of member, the CI should be carried out on the portions of the members exposed to moisture as cracking will normally be more widely developed. As mentioned before, cracking is usually most severe in areas of structures where the concrete has a constantly renewable supply of moisture, such as close to the waterline in piers, from the ground behind retaining walls, beneath pavements slabs, or by wick action in piers or columns. Concrete members undergoing AAR and experiencing cyclic exposure to sun, rain and wind, or portions of concrete piles in tidal zones often show severe surface cracking resulting from induced tension cracking in the “less expansive” (due to alkali leaching/dilution processes, variable humidity conditions, etc.) surface layer under the expansive thrust of the inner concrete core. It is consequently in the exposed portions of the members affected by ASR that the surface cracking is likely to provide the best estimate of the expansion reached to date, i.e., the closest to the current volumetric expansion in the affected member (see Section 5.5.2).

As the extent of expansion in a structural member is very much a function of the physical restraint to which it is subjected to, it is thus critical that CI measurements be taken parallel and perpendicular to the main restraints. Figure B1 provides examples of CI readings for different types of structural concrete members. In the case of reinforced concrete beams and columns, CI readings are taken on the exposed surface in the longitudinal and the two transverse axes of the members.

The Cracking Index is used to estimate the expansion to date in the affected member (see Section 5.5.2). This estimate is used as a criteria for the selection of the time left before proceeding with mitigation/remediation work; the evaluation is based on the number of years left before AAR-induced expansion would have reached a level of about 0.20 percent, level at risk for the stability of the reinforcing steel in the structure. This is assuming that the expansion thus estimated is similar to that reached in the reinforcing steel. One other critical factor is thus the quality of the concrete-steel bonding.

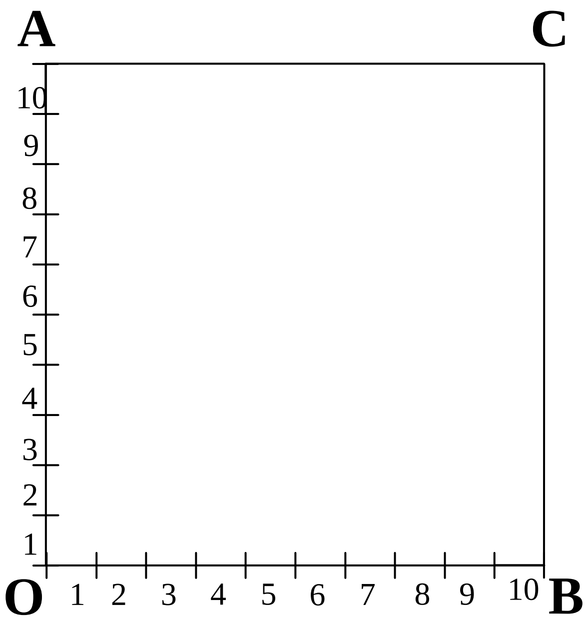
In the case of square or rectangular shaped reinforced concrete members (such as columns and beams), the most prominent cracks are often those appearing above the external bars of the reinforcing cage (i.e., close to the edge of the member). Since the steel-to-concrete bond is likely to be most affected at that level, it is recommended that: 1) the reference grid for the CI measurements stay within (i.e., not covering) those external cracking system, or perhaps better, 2) cover the whole section of the member with the reference grid but not accounting for the larger corner cracks in the calculation of the CI (as the crack width might not be representative of the concrete expansion because of the loss in the concrete/steel bond).

B.5 Measurement of the Cracks

Each crack of width greater than 0.05 mm (0.002 in) is identified and measured with the crack comparator or the graduated micrometer magnifying lens (Figure B2-F). The values obtained for each 0.1 m (4 in) segment of each axis are reported in the corresponding cell of the measurement table (example given in Figure B3). Main cracks can be reported on a sketch if desired.

Each crack is to be measured at least with 0.10 mm (0.004 in) accuracy starting at 0.05 or 0.10 mm (0.002 to 0.004 in) depending on the precision of the measuring device used) where it crosses the axis of the reference grid or at the nearest adequate location if the orientation of the crack differs too much from its general direction at the point of intersection. If the opening of the crack is sealed, swelled or awkwardly shaped, the measurements shall be taken at the nearest adequate location as well. The opening is always measured perpendicularly to its general direction, not following the length crossing the axis. A crack that crosses the axes of the reference grid several times is noted and measured as many times as it crosses an axis. However, one will try to avoid this type of configuration as much as possible when laying out the reference grid. “Crazing” cracking (very fine map-cracking) is not counted and measured, but its presence is reported.

If the measurements taken are part of a periodical inspection, the values obtained are immediately compared to the previous data to avoid gross errors in readings and in transcription. The measurement sheet is to be completed with the date, climatic conditions, and any useful information. Finally, a picture of the reference grid is taken.



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

Figure B3. Example of measurement of the Cracking Index. The values of C.I. are given separately for the vertical and horizontal measurements.

B.6 Timing / Frequency of Measurements

The frequency of the measurements is adapted to the particular needs of each case. The period can go from a few months in the case of recent and heavily damaged structures to a few years for older and well preserved ones. As a general guideline, bi-yearly (i.e., twice a year) measurements should be taken for the first 3 to 5 years and then every five years if the evolution of the damage is slow or nil.

Because of the effect of temperature and humidity on crack widths, CI readings should be carried out and repeated under very similar conditions of sun exposure, outdoor temperature and humidity conditions. Consequently, it is important that weather conditions at the time of reading be recorded as precisely as possible to allow selecting similar conditions for the next reading(s). Ideally, CI measurements will be best performed during cloudy conditions (i.e. limiting extended exposure to sun) with temperatures ideally ranging between 20 and 25 °C (68 to 77 °F). CI readings should also be taken following 24 to 48 hours of dry conditions, i.e., 24 to 48 hours following exposure to rain; this is proposed to allow stabilization of hygrometric conditions in the surface portion of the concrete member. The establishment of such "standard" weather conditions for the crack measurements will also allow better comparison between results obtained by different organizations (e.g. DOT's).

B.7 Handling of the Results

For each of the four axes of the reference grids, one calculates the total width of the openings, the average width of the openings, and the average opening width per meter (40 in) of axis. These values can be completed with a column graphic showing the width distribution of the openings.

B.8 Calculation of the Cracking Index

As mentioned before, for fair size structural members, at least two 0.50 m (20 in) reference grids will be drawn on the member from which crack mapping will be carried out. This will provide a minimum of 4, 0.50 m (20 in)-long measuring lines for each of the two directions of the grid (for a minimum of 2 m (80 in) of measuring lines). Smaller (e.g., 0.20 m (8 in) long) lengths could also be used for smaller members, but the number of reference grids required will be increased accordingly.

Since the extent of cracking (and of AAR expansion as well) greatly depends on the direction of the main restraints, the Cracking Index is calculated separately for each of the two directions of the reference grid, i.e., by taking the average of the four average opening widths measured in each of the two directions, and normalizing to a meter.

B.9 Estimate of the Expansion Reached to Date

The expansion reached to date by the concrete for each of the two or three dimensions (in the case of concrete columns or beams for instance) is estimated from the opening of all cracks intersected along each direction of the axis of the reference grid, as long as the concrete/steel bond remains appropriate or by eliminating the width of cracks above corner reinforcement where there are doubts about the quality of the bond.

Appendix C
Diagnosis of Alkali-Silica Reaction (ASR) *Petrographic Symptoms of ASR*

Petrographic examination is a very powerful technique in the diagnosis of the cause of its deterioration. ASTM C 856 outlines procedures for the petrographic examination of samples of hardened concrete. Interesting information regarding petrographic features of ASR-affected concrete can be found in several publications, including Bérubé and Fournier 1986, Hobbs 1988, Walker 1992, BCA 1992, St. John et al. 1998, CSA 2000, and Walker et al. 2006.

This appendix is meant to provide information on typical petrographic features of AAR, as observed on polished concrete sections, broken surfaces, and thin sections.

C.1 Macroscopic Observations on Cores (as Received)

Several macroscopic signs of concrete deterioration, some of which are related to ASR, can be observed by examining the cores immediately after the extraction or in the laboratory in an as received condition. The observations can be made with naked eye or aided by a small magnifying lens (up to 10X magnification) that can be easily used in the field. Such features can consist of:

- Macrocracks penetrating at different depths in the concrete member (Figure C1-A); fine-to-medium size cracks will stay damp while rewetting (Figure C1-C); macrocracks can be due to several mechanisms other than ASR.
- Gel staining surrounding surface cracks (Figure C1-B).
- Dark reaction rims at the periphery of reacted aggregate particles (Figure C1-D); dark rims may appear at the periphery of weathered gravel particles and are consequently not fully indicative of ASR in the case of gravel aggregates.
- Cracks within reactive aggregates (Figure C1-E), which extend sometimes in the cement paste, with/without reaction products gels.
- Alkali-silica gel in voids of the cement paste (Figure C1-F).
- Deposits of reaction products on the cracked surfaces of cores (Figure C1-G).

Figure C1. (A). Cores extracted from a concrete pavement affected by ASR and showing macrocracks penetrating from the upper and lower parts of the pavement. (B). Gel staining surrounding cracks and gel exudations at the surface of a core extracted from a sidewalk section affected by ASR (C). Fine cracking pattern showing up after re-wetting of the core. (D). Dark rim around the periphery of reactive aggregate particles. (E). Macrocracks in reactive coarse aggregate particles. (F). Deposits of alkali-silica gel in a void on core surface. (G). Deposits of alkali-silica gel on the broken surface of a core extracted from a highway bridge structures affected by ASR.

C.2 Microscopic Description of the Cores

Following the macroscopic description of the cores, various types of specimens may be prepared from the drilled cores. These mainly consist of **polished sections** or slices, **broken (fresh) surfaces**, and **thin sections**. The examination of polished surfaces with the naked eye and low-powered (stereo-binocular; up to 60x magnification) microscopy is an efficient method for studying large areas of concrete and determining the presence, distribution and extent of macroscopical features of ASR. The examination of thin sections will allow further positively identifying diagnostic features of ASR (e.g., sites of expansive reactions, reaction products). To maximize information generated through petrographic examination, polished slabs and thin sections can be prepared from various depths along the core sample. Table C1 lists features obtained from petrography as a function of the various methods of examinations mentioned above. Although not necessarily exclusive to ASR, petrographic signs or features of ASR generally consist of the following:

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

Table C1. Features to look for in petrographic examination and methods of examinations

Methods of Examination	Features
Megascopic examination (using a 10x lens)	• Damp patches, discoloration or staining at the surface of the cores, gel exudations at the surface and/or around the periphery of the core. • Aggregates particle shape and distribution (e.g., preferential orientation of particles) relative to the orientation of concrete placement and the direction of coring. • Cracking type and location/orientation (i.e., surface macrocracking, around and/or through aggregate particles), width, depth, crack orientation in the cement paste and versus the aggregate particle shape. • Secondary products in voids/pores, cracks, around aggregate particles or exuding from the core. • Reaction rims around aggregate particles.
Microscopic examination on polished sections (using a stereo-binocular microscope)	• Characteristics of the fine and coarse aggregates (petrographic nature, texture, grain size, shape, distribution, proportion, etc.). • Characteristics of the micro-cracking pattern, including intensity, size range of crack sizes, apparent association with particular aggregate type, cracking in or around aggregate particles, extension in the cement paste); cracks filled or not with ASR products, etc. • Reaction and/or alteration rims. • Reaction products (location and distribution, etc.). • Estimate of the relative proportion of the concrete constituents. • Characteristics of the air-void parameters. • Cement paste-aggregate bond, etc.
Microscopic examination of thin sections (petrographic microscope, scanning electron microscope)	• Petrographic nature of coarse and fine aggregates, presence of secondary products in air voids (e.g. calcium hydroxide, ettringite, alkali-silica gel, calcium carbonate, gypsum, etc.). • Sites of expansive reaction - occurrences of features that provide evidence of reaction and emanation of expansive forces, i.e. reactive aggregate particles showing cracking internally or at the cement/aggregate interface with cracks propagating into the surrounding matrix and cracks filled or partially filled with gel. • Basic characteristics of the concrete might be assessed by experienced petrographers, such as: water to cement ratio, degree of cement hydration, paste porosity, cement clinker composition, presence of filler or supplementary cementing materials such as fly ashes and slags, presence of contaminants. • Other features such as reaction rims, dedolomitization features, brucite, and carbonation.
Examination of broken concrete pieces (fracture surfaces) (using a stereo-binocular and/or SEM)	• Presence, distribution and abundance of reaction products on fractured surfaces (paste and aggregates), in voids of the cement paste, aggregate-cement paste bond, etc. • Presence of other reaction products (e.g., ettringite, etc.).

C.2.1 Microcracking

BCA (1992) and St. John et al. (1998) compare "idealized" cracking patterns in concrete specimens affected by various deleterious mechanisms (Figure C2). In the case of cracking induced by drying shrinkage (Figure C2-A), cracks are running fairly randomly through the cement paste, connecting aggregate particles through the interfacial zone with the cement paste. In the case of concrete affected by internal sulfate attack (e.g., Delayed Ettringite Formation (DEF)), cracks are found surrounding the coarse aggregate particles and filled with ettringite (Figure C2-B); when internal cracking is induced by shrinkage of the coarse aggregate particles, the cracks are similarly preferentially found surrounding the aggregate particles; however, they are empty (Figure C2-C). In the case of distress generated by frost attack, a random cracking pattern develops in the inner portion of the concrete, together with cracks developing parallel to the exposed surface of the affected concrete member; cracks will be often filled with secondary products such as calcite, portlandite, and/or ettringite (BCA 1992) (Figure C2-D). In the case of concrete affected by ASR when the reactive fraction in part either of the fine aggregate (Figure C2-E) or coarse aggregate (C2-F) fraction, a network of microcracks develops in the inner part of the concrete, with only a few "macrocracks" being observed in its outer portion (i.e., close to the surface). The microcracks are found connecting the aggregate particles; when the reactive material is found in the coarse aggregate particles, cracks typically run through the particles (Figure C2-F). The pattern will be affected by applied stresses/restraint and will develop preferentially parallel to the direction of the main compressive stress(es). Finally, the cracks will be often filled with secondary reaction products (i.e., the alkali-silica gel), which will be found showing a range of microtextures and chemical compositions. This will be discussed in more details hereafter.

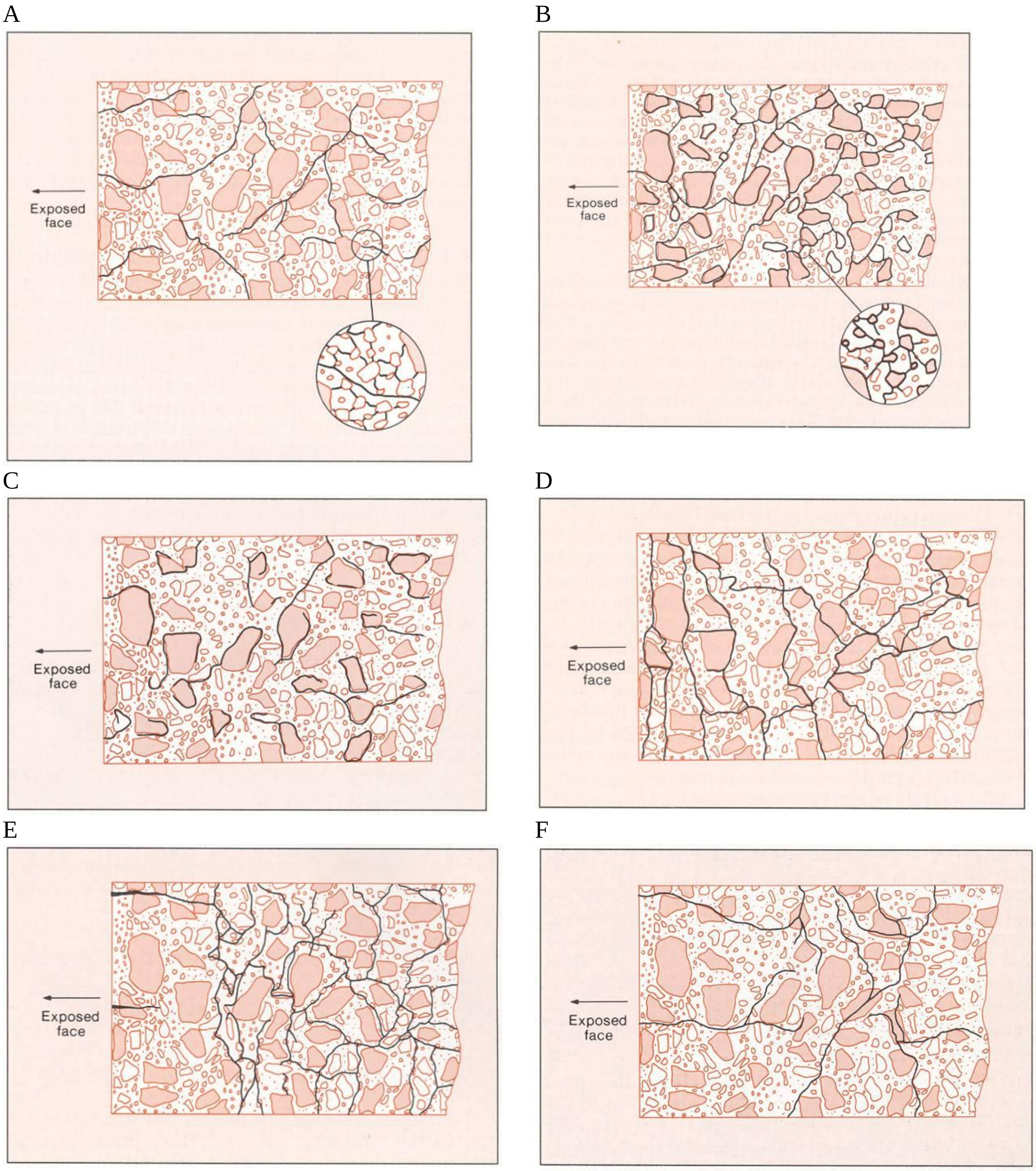


Figure C2. <u>From BCA (1992) (Note: This was taken directly from the BCA publication and should be for internal use / discussion only at this stage, i.e. until copyright issues be resolved)</u> (A). Internal crack pattern which can be induced by drying shrinkage. (B). Internal crack pattern which can be caused by internal sulfate attack for delayed ettringite formation (DEF), or from sulfates derived from the aggregates. (C). Internal crack pattern which can be induced by shrinkage of the coarse aggregate. (D). Internal crack pattern which can be induced by frost attack. (E). Internal crack pattern which can be caused by ASR: reactive silica in the sand fraction. (F). Internal crack pattern which can be caused by ASR: reactive silica in the coarse aggregate.

Microcracking due to ASR is generated through forces applied by the expanding aggregate particles and/or swelling of alkali-silica gel within and around the boundaries of reacting aggregate particles. The extent of ASR-related microcracking in a deteriorated concrete specimen depends on many factors such as the amount of reaction/expansion undergone by the concrete-which in turn depends on the inherent reactivity of the aggregate, the moisture conditions, the alkali content of the mix, etc.-and the total restraint to which the concrete member is subjected.

The proportion of aggregates showing internal cracking generally increases with progressing ASR. In the early stages of the reaction, microcracks are generally limited to the reacting aggregate particles and the cement paste-aggregate interface (Figure C3-A). With the progress of expansion, microcracks, more or less filled with alkali-silica gel, will extend from the aggregate particles into the cement paste; depending on the extent of expansion, the cracks will cover considerable distances through the paste where they are often filled with secondary reaction products (Figure C3-B to C3-E). In badly deteriorated concrete specimens, cracks-even filled with gel-may run through non-reactive aggregate particles. Consequently, great care should be taken to correctly identify the sites (or the aggregate particles) that have generated the expansive forces.

The examination of epoxy-impregnated polished slabs or thin sections are commonly used methods for the examination of microcracks in concrete; the incorporation of a UV tracer in the epoxy resin allows better detection of microcracking under UV illumination (Figure C3-F).

C.2.2 Reaction product "gel"

The ASR generates secondary reaction products containing silica, alkalis and calcium as typical constituents. The so-called "alkali-silica gel" will be found filling cracks within the aggregate particles (Figure C4-A), lining or filling voids and fractured surfaces of the cement paste and the aggregate particles (Figure C4-B to C4-E). These deposits will cover more or less important surfaces depending on many factors, such as the extent of the reaction-expansion processes that have occurred, the availability of water, etc. The above reaction product-which can be observed under the petrographic microscope-the stereo-binocular, and the scanning electron microscope-is a characteristic feature of ASR.

However, the abundance of gel deposits is not necessarily indicative of the magnitude of any resultant expansion and cracking (BCA 1992). Large amounts of gel in a concrete specimen do not necessarily indicate that large expansion or extensive cracking have occurred in the structure. On the other hand, cracking due to ASR has been observed in many concrete structures while very little gel was found in concrete specimens taken from the affected members.

The confirmation of the presence and the nature of reaction products is not always easy. Great care should be taken when preparing polished or thin sections from affected concrete specimens to avoid "leaching" of the alkali-silica gels. This could be achieved using a non-aqueous lubricant to avoid dissolution of the water-soluble compounds (BCA 1992).

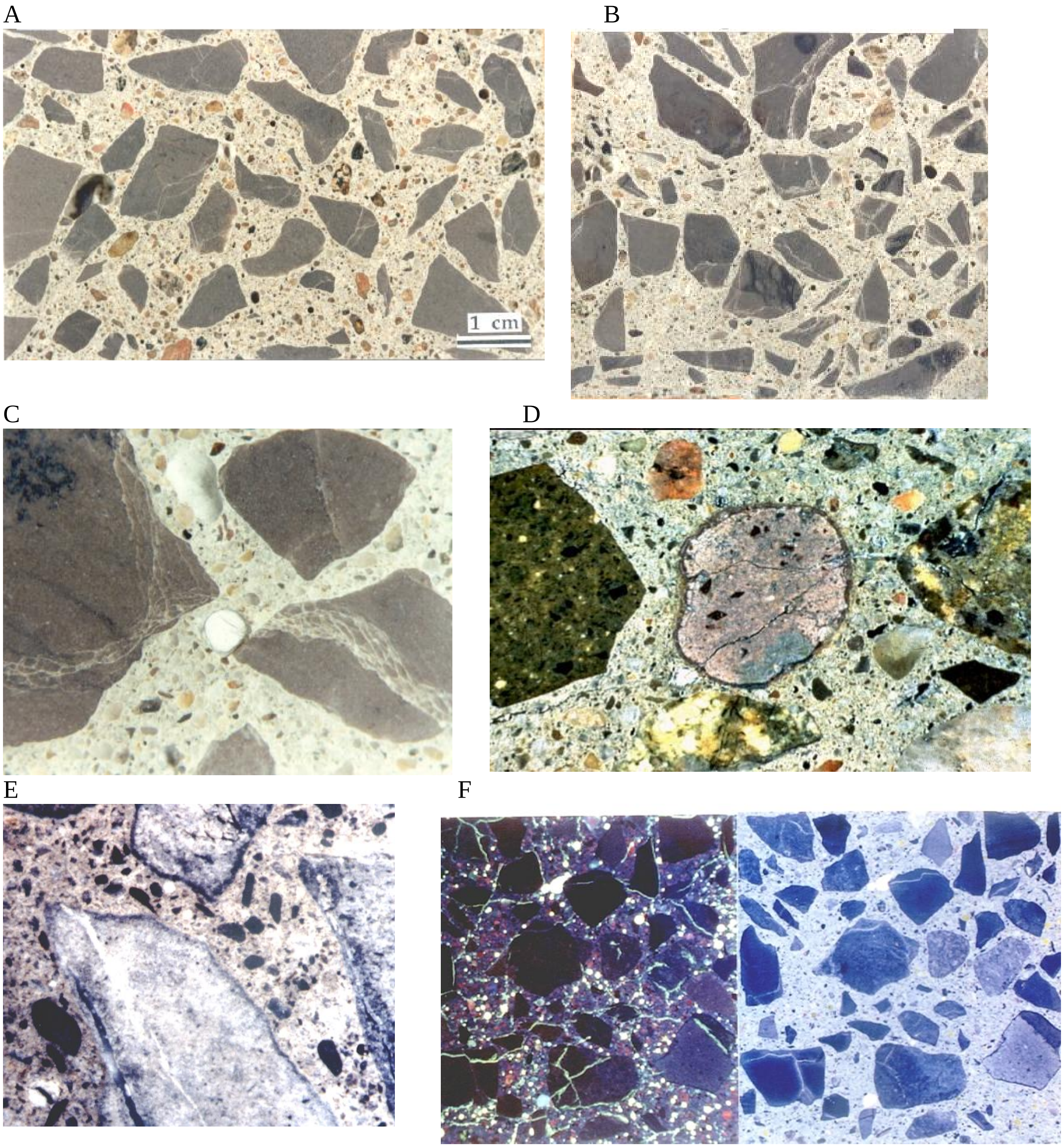


Figure C3. (A to C): Sections of concrete cut and polished from concrete prisms subjected to the Concrete Prism Test and incorporating a highly-reactive limestone coarse aggregate. (A and B). The proportion of cracked aggregate particle and the extent of ASR cracking increase with increasing expansion (A: expansion 0.065 percent; B: 0.149 percent). (C). Extensive cracking both in the aggregate particles and the cement paste; also, void filled with alkali-silica gel (Expansion > 0.25 percent). (D). Cracking due to ASR extending from one aggregate to another through the cement paste. (E). Polished concrete section incorporating a reactive volcanic aggregate and showing reaction rims and cracking filled with gel in the reactive aggregate particles. (F): Polished section of ASR-affected concrete impregnated with a fluorescent dye to help identify the presence and distribution of cracks and voids (left: impregnated; right: natural light).

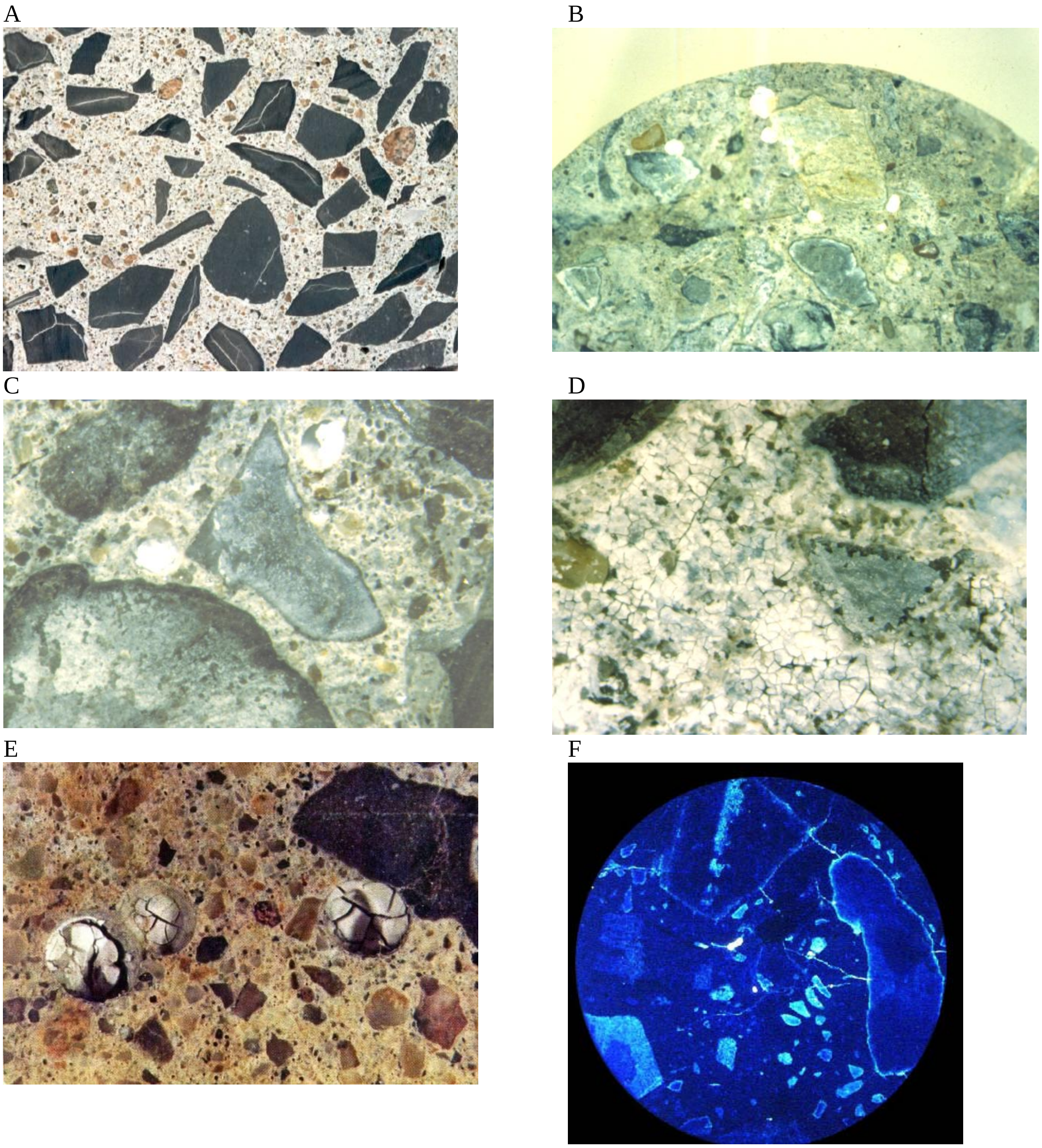


Figure C4. (A). Polished concrete slab showing cracks filled with gel inside reactive limestone aggregate particles. (B&C). Broken surfaces of concrete cores showing deposits of alkali-silica gel on the cracked surfaces of the aggregate particles and in voids of the cement paste. (D). Desiccated alkali-silica gel lining a crack in the cement paste of a concrete core affected by ASR. (E). Micrograph showing desiccated gel filling voids of the cement paste of concrete affected by ASR (F). Broken concrete core after treatment with uranyl acetate solution under UV illumination; the alkali-silica gel, which offers a greenish-yellow staining color, surrounds reacted particles of siliceous sandstone.

Staining techniques have been proposed to facilitate identification of the reaction product gel in concrete affected by ASR (Natesaiyer et al. 1991, Stark 1991, Guthrie and Carey 1997). A technique developed at Cornell University (Natesaiyer et al. 1991) consists in applying an uranyl acetate solution on polished or fresh broken surfaces of concrete specimens to be examined followed by a visual observation of the section under a UV light; the technique has even been used on field structures (Stark 1991, AASHTO 1993, ASTM C 856-02). Stark (1991) indicated that "by applying the uranyl acetate solution to a surface containing the gel, the uranyl ion substitutes for alkali in the gel, thereby imparting a characteristic yellowish-green glow when viewed in the dark using short wavelength ultraviolet light ASR gel fluoresces much brighter than cement paste due to the greater concentration of alkali and, subsequently, uranyl ion in the gel" (Figure C4-F, C5-A, and C5-B). This technique should be used with great care following appropriate health and safety procedures because of the potentially hazardous nature of the product. Technically speaking, the results of the test should be interpreted with great care. Some aggregates fluoresce naturally, which can incorrectly suggest the presence of alkali-silica gel through macroscopic or microscopic examinations; also, although the technique may help locate the presence of gel in the concrete, it will not differentiate between a harmless presence of gel and that which is detrimental (the source of the observed distresses). Actually, any concrete incorporating silica-bearing aggregates may show traces of gel in air voids or surrounding aggregate particles, which is not necessarily an indication of deleterious ASR. On the other hand, some concrete experiencing ASR expansion sometimes contain limited amounts or "visible" gel (i.e., gel visible under commonly used stereobinocular or petrographic microscope); in such cases the uranyl acetate test could be very useful in detecting the presence and the distribution of gel. Overall, when used with care, the method can efficiently support conventional petrographic examination procedures and physical tests for investigating causes of concrete expansion.

The examination of ASR-affected concrete in thin section under the petrographic microscope often allows better locating the presence and the distribution of the reactive aggregates and the secondary reaction products, as illustrated in Figure C5-C to C5-E.

Finally, confirmation of the presence and, to certain degree, of the extent of alkali-silica reaction in the concrete, can be done through examination of polished (Figure C5-F) or broken concrete fragments (Figure C6-A to C6-E) under the scanning electron microscope. This approach allows to precisely identify the presence and distribution of alkali-silica gel through its typical textural and chemical characteristics.

C.2.3 Reaction rims

Dark reaction rims are observed at the internal periphery of a number of alkali-silica reactive aggregates in deteriorated concrete specimens. These are particularly evident on polished sections or slabs of affected concrete cores (Figure C6-F). However, these rims must not be mixed up with pre-existing (e.g., before the introduction of the aggregate particle in the concrete) "weathering" rims that are often found in the outer (but also internal) portion of weathered gravel particles.

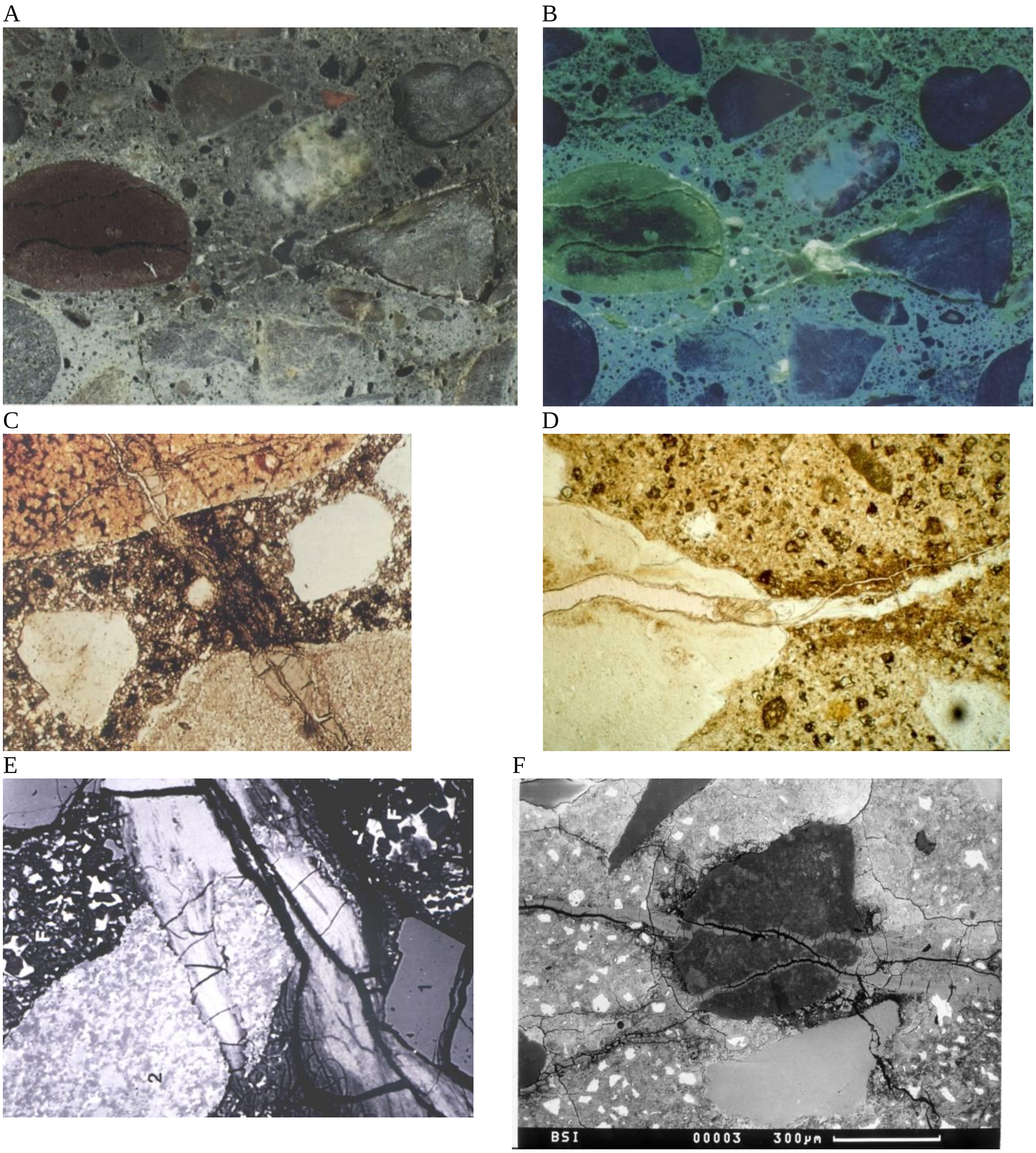


Figure C5. (A, B). Polished concrete sections treated with uranyl acetate solution to enhance the presence of alkali-silica gel (A. natural light; B. under UV illumination showing the gel in greenish-yellow staining color filling cracks in the cement paste in the vicinity of reactive aggregate particles). (C-E). Thin sections micrographs showing cracks filled with gel (desiccation texture) and extending from reactive aggregate particles through the cement paste. (F). SEM micrograph of polished concrete section showing cracks filled with gel and extending from reactive aggregate particles through the cement paste.

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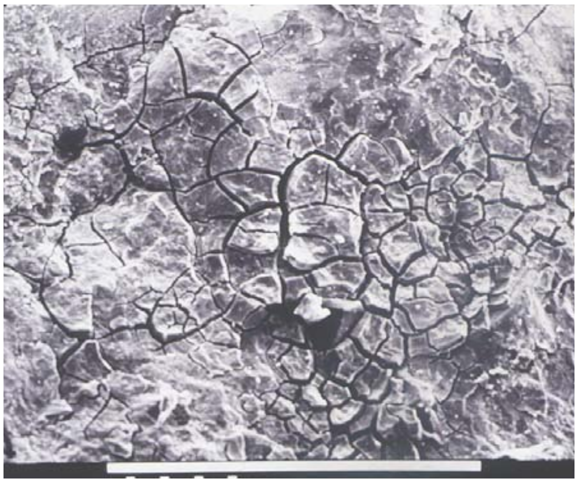

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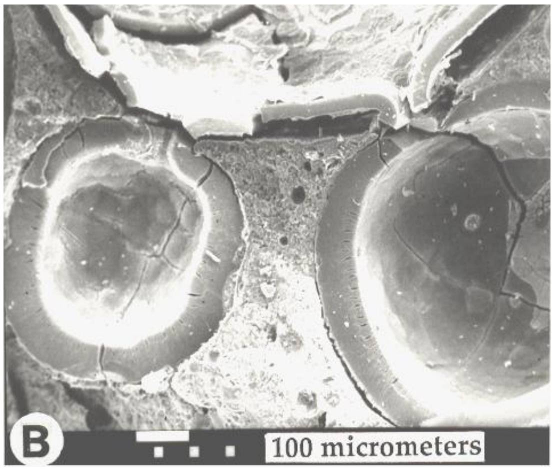

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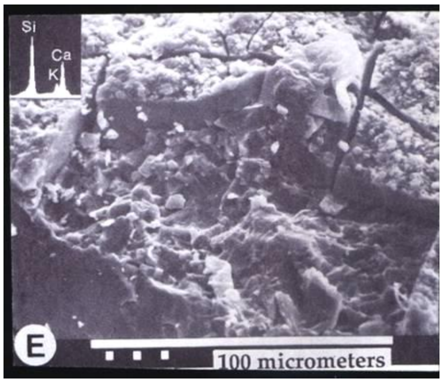

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F

Figure C6. (A). SEM micrograph showing deposits of alkali-silica gel on the surface of a broken concrete core affected by ASR. (B). SEM micrograph showing alkali-silica gel lining voids of the cement paste in a concrete sample affected by ASR. (C). Broken surface of a concrete cores showing deposits of alkali-silica gel on the cracked surface of a reactive aggregate particle and in voids of the cement paste. (E). SEM micrograph showing the layer of gel forming the dark rim on Figure (C). (E). SEM micrograph of the crystalline products showing a rosette-like microtexture corresponding to the white deposits inside the aggregate particle of Figure (C). (F). Polished sections of concrete cores showing dark reaction rims around reacted aggregate particles.

When concrete cores are fractured for examining "fresh" broken surfaces, cracks that have formed within the aggregate particles and the cement paste, due to the ASR processes, will form zones of weakness where the core will preferentially break. The fractured surfaces thus created (which in many cases correspond to "ASR cracking surfaces") often show a dark rim surrounding internal deposits of whitish color (Figure C6-C). Such a feature does not correspond to a reaction rim per se; it actually corresponds to a typical arrangement of reaction products deposited on the cracking surface, i.e., 1) a dark rim covering the immediate internal periphery of the particle, and 2) white deposits going through the central portion of the particle showing a powdery aspect. Examination under the scanning electron microscope (SEM) confirms the dark rim to be a layer of calcium-rich alkali-silica gel (Figure C6-D), while the whitish deposits are formed by a rosette-like crystalline product (Figure C6-E). Those are typical products of ASR.

C.2.4 Loss of the cement paste-aggregate bond

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

C.3 Quantitative Petrographic Assessment – The Damage Rating Index

Grattan-Bellew (1992) and Dunbar and Grattan-Bellew (1995) described a method to evaluate the condition of concrete by counting the number of typical petrographic features of ASR on polished concrete sections (16x magnification). A grid is drawn on the polished concrete section, which includes a minimum of 200 grid squares, 1 cm by 1 cm (0.4 by 0.4 in) in size. The *Damage Rating Index* represents the normalized value (to 100 cm^2) (16 in^2) of the presence of these features after the count of their abundance over the surface examined has been multiplied by weighing factors representing their relative importance in the overall deterioration process (Table C2) (Shrimer 2000). Rivard et al. (2000) used the method to estimate the amount of expansion reached by concrete specimens cored from a large concrete dam affected by ASR; the authors also found that the relative importance of the different petrographic features of ASR, in terms of how they correlate with the measured expansion due to ASR, can vary significantly from one aggregate to another (Rivard et al. 2002). Figure C7 illustrates petrographic observations performed under the DRI method, as obtained by Fournier et al. (2007). Figure C7A to C7D illustrates the various petrographic features of ASR that are quantified as part of the process; the results are then compiled as a function of those various features, as illustrated in Figure C7E, to obtain the DRI for the sections examined. The DRI results in Figure C7E can be seen to correlate with the expansion of the test samples examined.

Table C2. Weighing factors for the Damage Rating Index

Petrographic feature	**Weighing factor**
Coarse aggregate with cracks	x 0.25
Coarse aggregate with cracks and gel	x 2.0
Coarse aggregate debonded	x 3.0
Reaction rims around aggregate	x 0.5
Cement paste with cracks	x 2.0
Cement paste with cracks and gel	x 4.0
Air voids lined or filled with gel	x 0.50

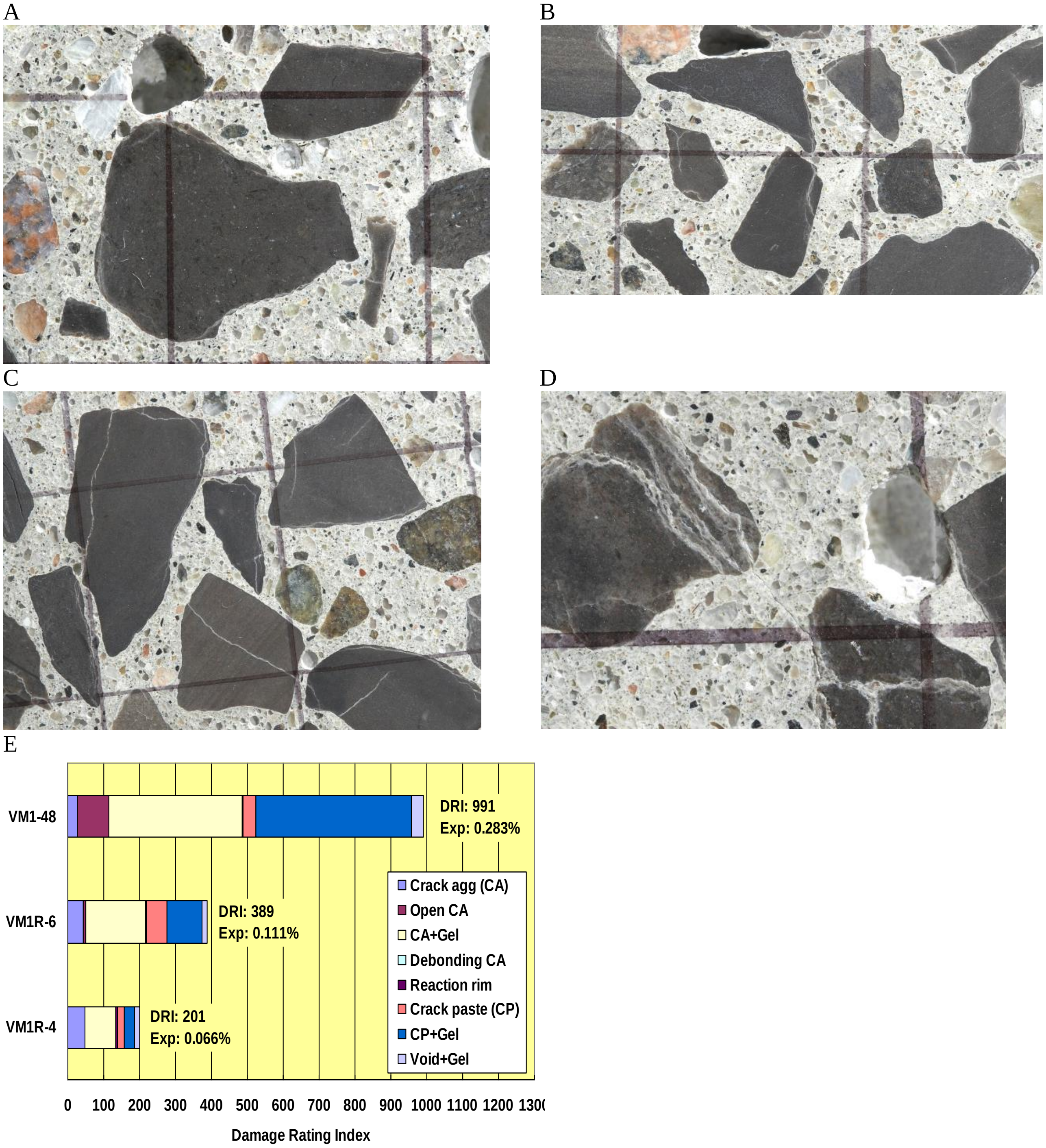

Figure C7. Micrographs showing petrographic symptoms of ASR as quantified as part of the DRI method. The polished concrete sections were cut from C 1293 prisms having reached different amounts of expansion (Spratt limestone reactive aggregate); a reference grid with 1cm (0.4 in) squares was then dawn at the surface of the polished section for DRI measurements. (A). Concrete prisms having reached an expansion of 0.020 percent. (B). Concrete prisms having reached an expansion of 0.066 percent. (C). Concrete prisms having reached an expansion of 0.111 percent. (D). Concrete prisms having reached an expansion of 0.283 percent. (E). Compilation of DRI values for the test prisms examined.

More recently, the DRI method was used to quantify the condition of laboratory-made concrete specimens incorporating various aggregate types and subjected to accelerated test conditions in the laboratory (38°C (100°F) and R.H. > 95 percent) (Smaoui et al. 2004a). The same method was used to determine the condition of concrete cores sampled from blocks exposed at the CANMET-MTL outdoor exposure site in Ottawa (Canada) and incorporating similar aggregates (Smaoui et al. 2004b). All of the above specimens were examined at known expansion levels due to ASR. They used the reasonable correlations thus obtained to estimate the amount of expansion reached by concrete cores extracted from structures affected by ASR incorporating siliceous limestone aggregates (Bérubé et al. 2004). The authors concluded that, unfortunately, the DRI method could not differentiate between concretes affected most and least visually/mechanically by ASR. High DRI values and consequently exaggerated estimated expansion to date were obtained for all concretes investigated (Bérubé et al. 2005). For that reason, the method is not recommended for evaluating the expansion to date of concrete affected by AAR.

Despite the above comments, the DRI method can represent a useful tool for the quantitative assessment, based on petrography, of internal damage in concrete due to ASR or other mechanisms. However, as the results are very much related to the experience of the petrographer and since there is currently no standard test procedure available, the method is fairly subjective and the results can be quite variable from one petrographer to another. Consequently, it is impossible, at this stage, to identify limits of DRI numbers that would be indicative of low, mild, and severe AAR ratings. This is even more the case knowing that the relative importance of the different petrographic features of ASR, in terms of how they correlate with the measured expansion due to ASR, can vary significantly from one aggregate to another, which can result in fairly different DRIs for similar level of expansion depending on the reactive aggregate present in the concrete (Rivard et al. 2002). Despite all of that, the DRI method can provide very useful relative information when the examination of a set of cores from the same structure (presumably incorporating the same aggregate) will be carried out by the same experienced petrographer. The method would also identify differences in damage ratings between members of a single structure that would incorporate different reactive or non-reactive aggregates.

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Appendix D
Diagnosis of Alkali-Silica Reaction (ASR)
In-situ Investigations

This appendix is meant to provide additional information on the various techniques used for the in-situ monitoring of expansion, temperature and humidity in concrete structural members affected by ASR.

D.1 Deformation Measurements

In-situ deformation measurements can be easily performed by installing demec points (Figure D1-E) and/or metallic references/devices at the surface of the concrete members showing visual distress indicative of possible AAR.

D.1.1 Selection of members

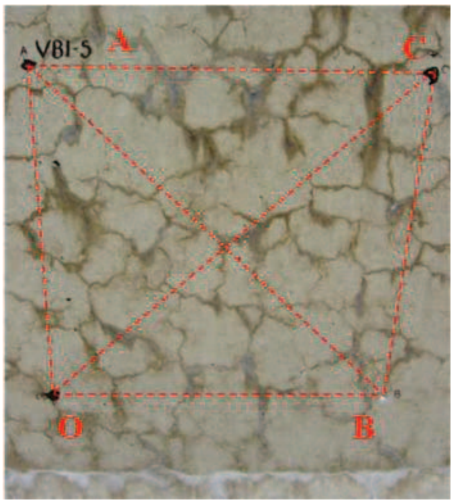
Ideally, demec points should be installed on the same (or a selection of the same) members that have been chosen for crack mapping, i.e., members showing a range of deterioration but always exposed to moisture. Ideally, the demec points will be installed at the end of the crack-mapping reference grid used for CI measurements (Figure D1-A). This will optimize the information generated on the same testing area and will allow for two transverse and two longitudinal length-change measurements.

Holes are dry-drilled into the concrete (Figure D1-B and D1-C) of a sufficient size for the gage stud to be introduced in the hole with the rapid-hardening cement paste or epoxy glue (Figure D1-D). Stainless steel bolt, approximately 5-7 mm (0.2-0.3 in) diameter by approximately 15-20 mm (0.6-0.8 in)-long, with a machined “demec point” at the end were found appropriate for that purpose. The surface of the stud should be flush with, or slightly inside the surface of the concrete member.

D.1.2 Length-change measurements

About 12 to 24 hours after setting of the studs, initial length measurements will be taken. The weather conditions (especially temperature, whether the concrete element is directly exposed to sun/rain or not, etc.) will be noted. Similarly to the CI measurements, and because of the significant effect of temperature and humidity on dimensional changes in concrete, expansion readings should be carried out a minimum of twice a year, ideally three times for the first year, and repeated under very similar conditions of sun exposure, outdoor temperature, and humidity conditions (see Appendix B for more details) (Figure D2-A to D2-F).

A



B

C

D

E

Figure D1. (A). Reference grid for measurement of the Cracking Index (CI). Holes are drilled at each corner of the grid in order to allow length-change measurements along the two main directions of the grid. (B and C). Holes are dry-drilled in the structural member for the set-up of the gage studs. (D). The gage stud is lined with rapid-hardening cement paste or epoxy glue before installing it in the hole on the reference grid. (E). Various types of accessories serving as demec points to be fixed or drilled into the concrete member (picture from Mayes Group website).

Figure D2. (A). Length-change measurements in a concrete pavement affected by ASR. (B). Length-change measurements in a concrete highway barrier wall affected by ASR. (C to F). Length-change measurements in reinforced concrete columns affected by ASR.

D.1.3 Other types of deformation measurements

Periodic length-change measurements can also then be taken using extensometers of various shapes and ranges, invar wires/rods or optical systems (leveling) (Figure D3-A to D3-F)). Fiber-optic and vibrating wire systems can also be used, with deformation measurements being performed and the data transmitted automatically to central servers for further treatment.

Evidence of stress build-up in reinforcing steel and the surrounding concrete resulting from restrained AAR expansion can be obtained from the measurements of stress in reinforcing bars (Figure D4-A and D4-B) and the overcoring technique (Danay et al. 1993) (see Section 5.2.4).

D.2 Temperature and Humidity Measurements

It is highly recommended that relative humidity and temperature readings be made in a selected member of the structure in order to make any necessary corrections to the expansion measurements performed as part of the monitoring program. Such readings can be made periodically using portable probes (Figure D4-C), or automatically using in-situ set-ups (Figure D4-D). The above probes usually generate temperature readings that can be used to make any corrections required for temperature variations. The values could also be used in the process of evaluating the potential for future expansion of the concrete (see Appendix I).

D.3 Full-Scale Loading Testing

In the case of reinforced concrete bridges where the visual survey and the in-situ measurements indicate a severe level of deterioration (severe cracking in structural members), it will likely be necessary to determine whether or not the stability of the structure is at risk. This should be carried out through a full-scale investigation performed by a competent structural engineer. Full-scale loading tests in the field will ultimately permit assessment of the real structural loss in performance (or serviceability) of the affected structure (Figure D4-E and D4-F).

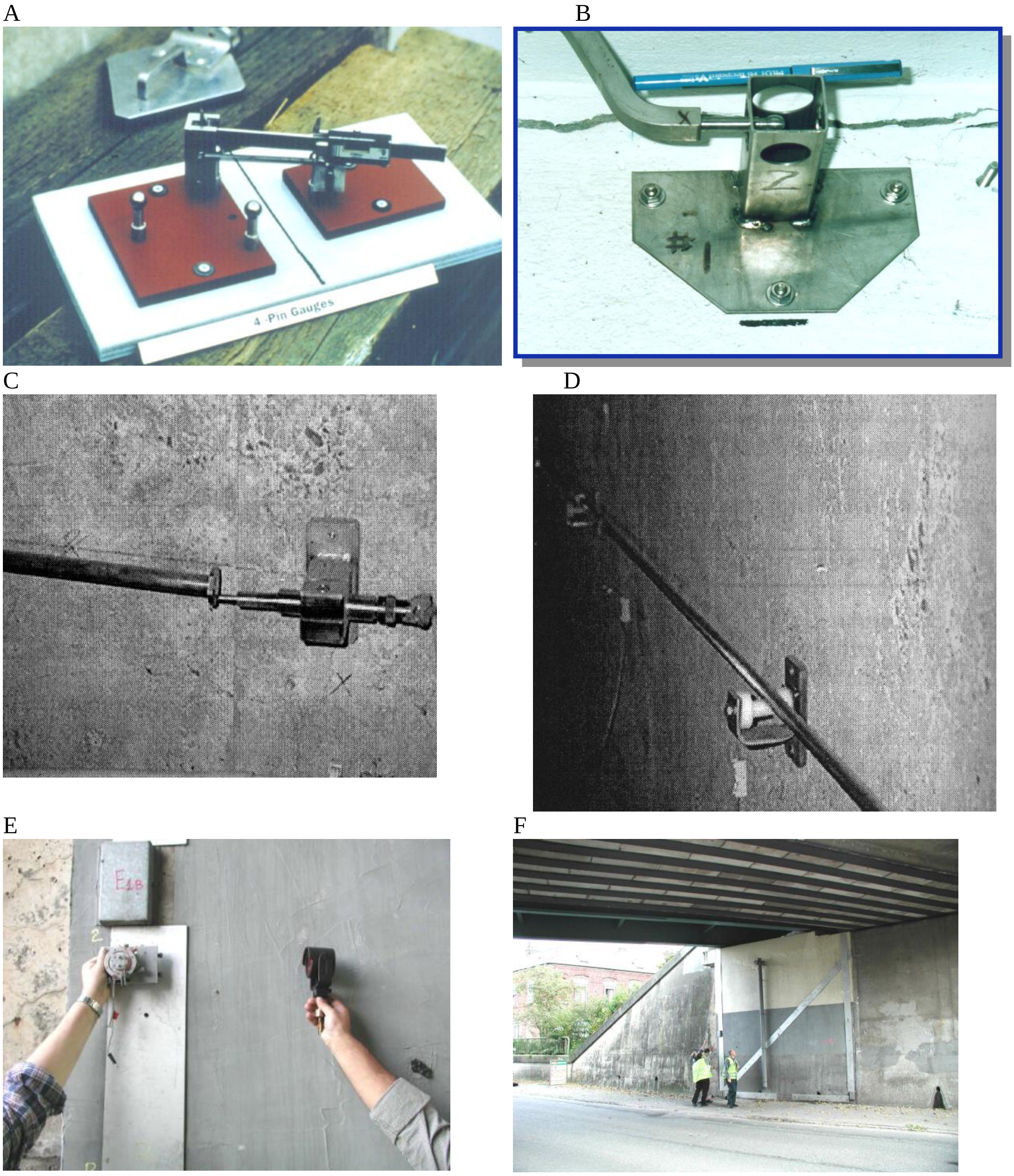


Figure D3. (A and B). Devices used for the in-situ monitoring of deformation/movement associated with cracks (Thompson et al. 1995). (C and D). Invar-bar expansion set-up used for the in-situ monitoring of deformation/movement associated with AAR (Gaudreault 2000). (E and F). (E). Equipment ("infrared distancemetre") used for long-distance in-situ expansion monitoring in the highway bridge structure illustrated in (F) (LCPC 1999); the infrared source is on the right and the target on the left. Both are fixed on the abutment wall section of the bridge structure to ensure accurate readings.

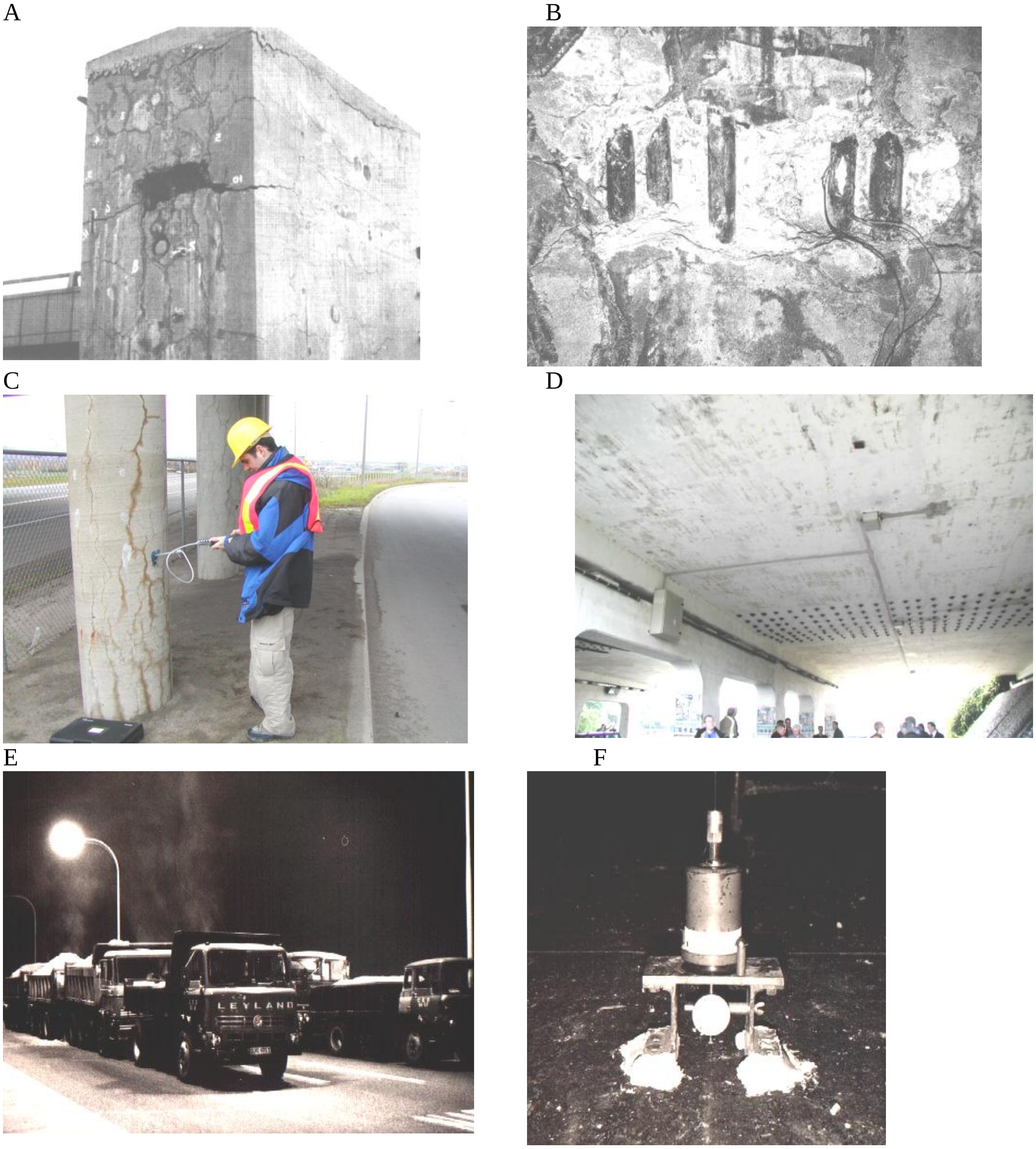


Figure D4. (A). Photograph of the western haunch of a bridge structure showing damage due to ASR. The hole in the middle of the member was cut to expose reinforcement for strain gauging (Blight and Ballim 2000). (B). Strain gages mounted on the second reinforcing bar from the right. (C). In-situ measurement of relative humidity, using a portable probe, in a concrete column affected by ASR. (D). Set-up for the automatic in-situ monitoring of temperature, humidity and expansion (vibrating wire) in a bridge deck affected by ASR (Siemes and Gulikers 2000). (C).. (E). Arrangement of trucks during a full-scale loading test performed on a motorway portal structure affected by ASR in South Africa (structure in C) (Blight et al. 1983). (F). During the full-scale loading test illustrated in (E), displacement was measured at several points of the beam by means of gages bedded on the lower deck of the portal. (Pictures C to F: courtesy of G.E. Blight, University of Witwatersrand, Johannesburg, South Africa)

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Appendix E
Determination of Internal Damage and Expansion to Date in ASR-Affected Concrete
The Modified Stiffness Damage Test (SDT)

E.1 Introductory Remarks

The Stiffness Damage Test (SDT) was originally proposed by Chrisp et al. (1989, 1993) and adopted by the Institution of Structural Engineers (ISE 1992). It is based on the cyclic uniaxial compressive loading of concrete core samples (5 cycles) between 0 and 5.5 MPa (800 psi). The reduction in the Young's elastic modulus, the energy dissipated during the load-unload cycles, which corresponds to the surface area of the hysteresis loops, and the accumulated plastic strain after these cycles, are associated with the closure of the existing cracks and to a slip mechanism, and thus represent a measure of the damage in the specimen (microcracking) in the direction of the applied stress.

The capacity of the test to estimate the expansion attained to date by ASR-affected concrete has been recently investigated in detail (Smaoui et al. 2004a, 2004b, 2004c, Bérubé et al. 2005a). The test was found to be useful in that respect, particularly when applying a maximum load of 10 MPa (1450 psi) during the 5 load/unload cycles (rather than 5.5 MPa (800 psi) in the original method) and using one or both of the two following parameters: (1), the energy dissipated during the first cycle (hysteresis loop 9 see Figure E1), and (2), the accumulated plastic strain after the 5 load/unload cycles. These two parameters progressively increase along with the internal microcracking and the consequent bulk expansion of the concrete under study (Figure E1 and E2). However, as the first parameter was found to be more closely correlated to ASR expansion this is the parameter that is selected in the following procedure. This modified SDT can thus supply useful information about the internal damage (microcracking) of the concrete under study and the expansion attained to date due to ASR (or to any other expansive mechanism causing cracking). Moreover, when the number of cores available for mechanical testing is limited, the elastic modulus can be determined in the modified SDT and the compressive strength can be estimated by reloading the cores up to failure after the 5 load/unload cycles. However, despite the good correlation obtained, between the energy dissipated during the first load/unload cycle and the expansion due to ASR and this, for each type of reactive aggregate investigated up to now in the laboratory, the results are aggregate-dependent (Figure E3); this thus requires the use or the establishment of a calibration curve suitable to the particular reactive aggregate present in the concrete under study or corresponding to a similar type (see Section E.2.9).

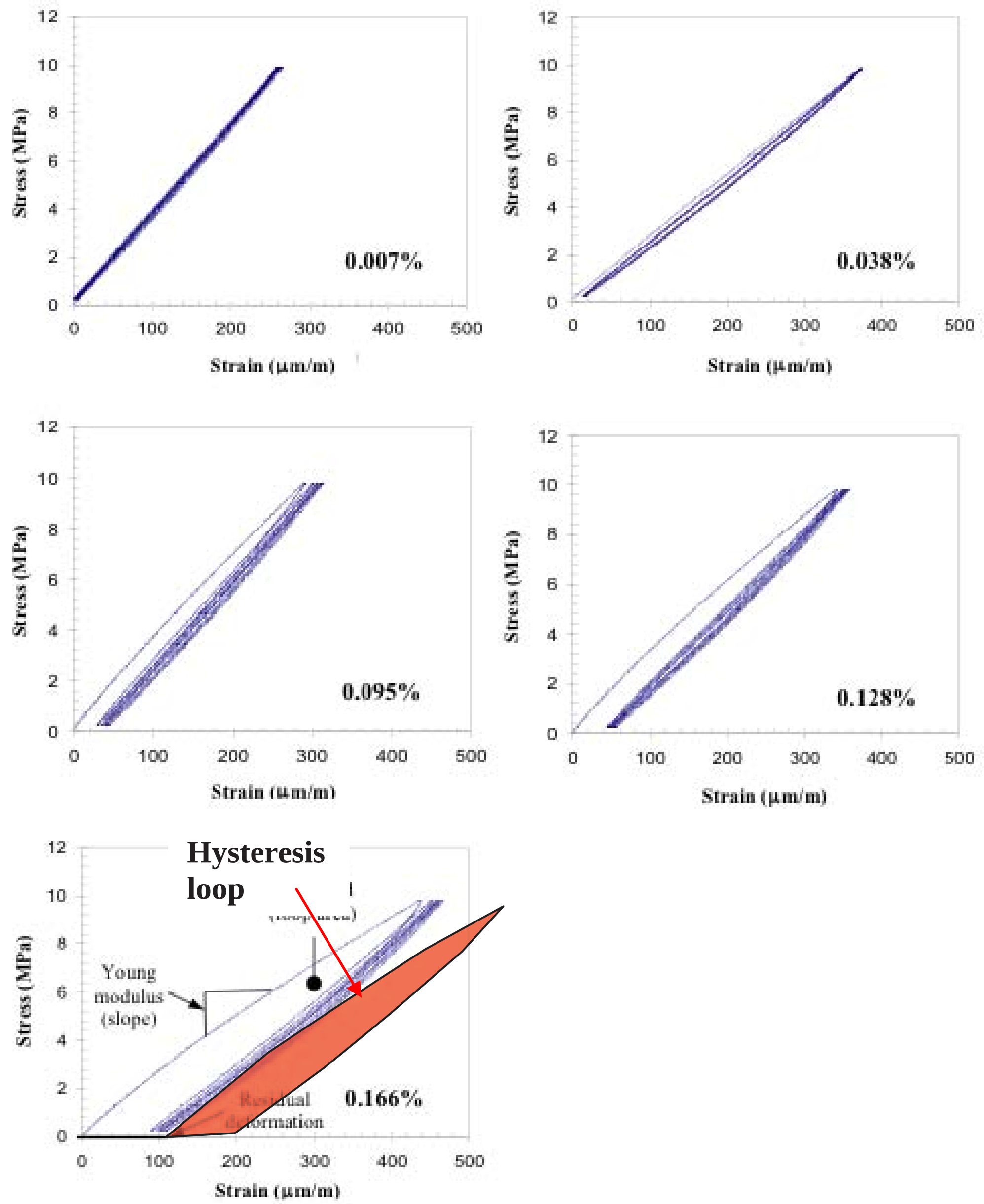

Figure E1. Results from the modified SDT at various expansion levels for concrete cylinders made with a highly reactive siliceous limestone coarse aggregate from Québec (Canada) (After Smaoui et al. 2004a, 2004c).

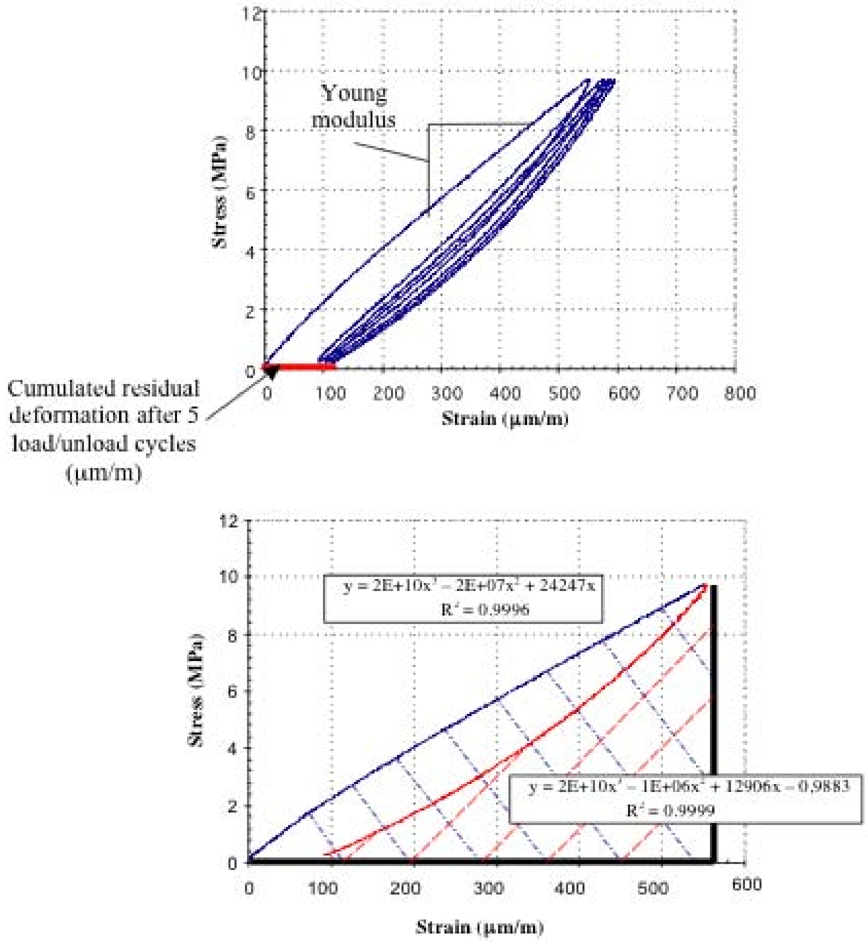

Figure E2. Results from the modified SDT for a concrete cylinder made with an extremely-reactive sand from Texas at an expansion level of 0.392 percent. The energy dissipated during the first load/unload cycle (area of the first hysteresis), in Joule/m^3, is obtained by subtracting the area (in red) under the first unload curve from the area (in blue) under the first load curve. Third-degree polynomial relationships can be obtained for these curves from which the above calculation can be easily made. (After Bérubé et al. 2005b).

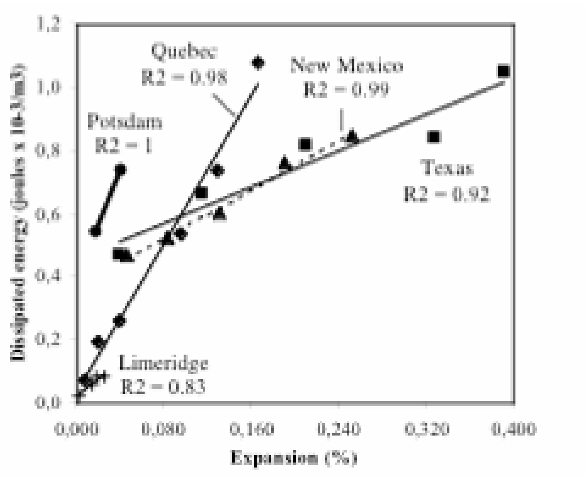

Figure E3. Linear relationships between the energy dissipated during the first load/unload cycle of the modified SDT and the expansion of concrete cylinders made with different coarse (Québec, New Mexico, Potsdam, Limeridge) or fine (Texas) aggregates susceptible to ASR. (After Smaoui et al. 2004a, 2004c).

E.2 Test Procedure

E.2.1 Core diameter

The test procedure is essentially recommended for cores of 100 mm (4-in) in diameter.[7]

E.2.2 Storage conditions before testing

The cores must be tested in the same moisture conditions that prevail in nature. Therefore, they must be tightly sealed in plastic sheets immediately after coring in order to prevent any volumetric change due to drying (e.g., unsealed cores stored at ambient temperature) or to moisture uptake (e.g., unsealed cores stored in a wet room).

E.2.3 Specimen preparation

The modified SDT primarily consists in measuring the deformation of concrete cores when submitted to cyclic uniaxial compressive loading. The test specimens are thus prepared as for the standard uniaxial compression test ASTM C 39 (*"Standard test method for compressive strength of cylindrical concrete specimens"*), i.e., sawed and capped or ground to obtain two end faces that are perfectly parallel and perpendicular to the core axis. A great care must be given to satisfy the above requirement since small volumetric changes have to be precisely measured. The length of cores to test, 100 mm (4-in) in diameter, shall be 200 ± 5 mm (8 ± 0.2 in).

[7] So far, testing has been carried out on 100-mm (4-in) cores only. Preliminary results (Smaoui et al. 2004a) indicated that the results are greatly affected by the core diameter and a study is presently underway with the objective of adjusting the method for cores of 75 and 150 mm (3 to 6 in) in diameter.

E.2.4 Testing machine

Any type of testing machine capable of imposing load/unload cycles at the rate and of the magnitude prescribed in Section E.2.7 may be used, provided the data acquisition system allows the simultaneous measurement at any time of the stress applied and the longitudinal strain. The machine shall also conform to all other requirements of Practices E 4 (*"Standard practices for load verification of testing machines"*) and Test Method ASTM C 39 (*"Standard test method for compressive strength of cylindrical concrete specimens"*).

E.2.5 Measuring device

For measuring the longitudinal deformations during the test, an unbonded sensing device shall be used, which conform to Section 4.2 of standard ASTM C 469 (*"Standard test method for static modulus of elasticity and Poisson's ratio of concrete in compression"*). In addition, this device shall allow measurements at the nearest micrometer along the two diametrically opposite gage lines, since small volumetric changes have to be precisely measured.

E.2.6 Number of test specimens

A minimum of three cores, 100 mm (4 in) in diameter by 200 ± 5 mm (8 ± 0.2 in) in length, should be tested, for each concrete under study.

E.2.7 Load/unload cycles

The test involves 5 cycles of loading up to 10 MPa (1450 psi) and unloading down to 0 MPa.[8] The rate of loading and unloading is 0.1 $N/mm^2/s$. The stress applied and the longitudinal deformations along each of the two gage lines are recorded by the data acquisition system for each increment of 0.5 N/mm^2 during loading and unloading.

When the 5 cycles are completed, the compressive strength can be obtained by reloading the specimen tested up to failure.

E.2.8 Handling and interpretation of test results

Stress-strain graph - A graph of the stress applied as a function of the longitudinal strain (average of the two diametrically opposite measurements) is drawn (Figure E1 and E2).

Elastic modulus - The Young's elastic modulus can be estimated from the slope of the linear relationship corresponding to the first loading (Figure E1 and E2). This parameter is of great interest in the diagnosis of AAR and for determining the progress of the reaction (provided tests are periodically conducted over time on core samples).

[8] Considering that the parameter used for assessing the expansion to date is the energy dissipated during the first load/unload cycle, thus neglecting the residual deformation after 5 cycles, only one cycle should be sufficient. However, the behavior of the cores tested during the other 4 cycles is useful information.

Energy dissipated during the first load/unload cycle - The energy dissipated during the first load/unload cycle (area of the first hysteresis), in Joule/m^3, is obtained by subtracting the area (in red in Figure E2) under the first unloading curve from the area (in blue in Figure E2) under the first loading curve. Third-order polynomial relationships can be obtained for these two curves from which the energy dissipated can be easily calculated. In general, these relationships present correlation coefficients of 0.999 or more (Figure E2).

Expansion attained to date - The expansion attained to date by the ASR-affected concrete under test is ideally estimated from an empirical linear relationship between the energy dissipated during the first load/unload cycle and the expansion due to AAR, established in the laboratory for concrete cylinders made with the reactive aggregate present in the concrete under study or a similar type of reactive aggregate (see Section E.2.9). As mentioned earlier (Section E.1), the results of the test are greatly aggregate dependent (Figure E3). In absence of the above information, a relationship already obtained for quite similar reactive aggregates can be used. The results presented in Figure E3 have been obtained for cylinders made with 4 different types of reactive aggregates from the U.S.A. and Canada (Smaoui et al. 2004a, 2004c). Each corresponding relationship presents a correlation coefficient (R^2) of 0.92 or more (Table E1) but is clearly different from one aggregate to another. In the absence of any more relevant information, the expansion to date could be estimated using the linear relationship in Table E1 corresponding to the reactive aggregate the most similar to the type present in the cores subjected to the modified SDT. It is important to recall that these relationships apply to concrete specimens of 100 mm (4 in) in diameter, only.

Table E1. Relationships obtained in the SDT for different types of reactive aggregates. (After Smaoui et al. 2004a, 2004c).

Reactive aggregate type	**State (Country)**	**Relationship**	**R^2**
Siliceous limestone	Quebec (Canada)	Energy = 0.00003614 + 0.5871 • expansion	0.98 (6 results)
Natural gravel rich in volcanic particles	New Mexico (U.S.A.)	Energy = 0.0003741 + 0.1900 • expansion	0.99 (5 results)
Quartzitic sandstone (Potsdam Group)	Quebec (Canada)	Energy = 0.0003494 + 0.9702 • expansion	1.00 (2 results)
Natural sand rich in volcanic particles	Texas (U.S.A.)	Energy = 0.0004577 + 0.001427 • expansion	0.92 (5 results)

E.2.9 Establishment of the calibration curve

The specimens required for the establishment of the calibration curve for a particular reactive aggregate correspond to concrete cylinders, 100 mm in diameter x 200 mm in length. These cylinders are made and tested in accordance with the Concrete Prism Test ASTM C 1293, except that different sets of specimens must be submitted to the SDT at different expansion levels (including a “zero” level). It is recommended that a minimum of 5 expansion levels (including the “zero” level) be considered using a constant increment between two successive levels. This increment should depend on the maximum expansion that the particular aggregate could generate, but it should never exceed 0.04 percent. For instance, if the maximum expansion is 0.10 percent, thus the recommended expansion levels are: 0.00 percent, 0.025 percent, 0.050 percent, 0.075 percent, and 0.10 percent (5 different expansion levels).[9] If the maximum expansion is 0.20 percent, thus the recommended expansion levels are: 0.00 percent, 0.04 percent, 0.08 percent, 0.12 percent, 0.16 percent, and 0.20 percent (maximum increment of 0.04 percent). Considering that a minimum of three specimens must be tested per SDT, i.e. for each expansion level, a minimum of 18 cylinders must then be made in the latter case for the establishment of the calibration curve. After the last expansion measurement, the specimens are immediately sealed with several layers of plastic wrap in order to prevent any moisture loss, stored for one day at room temperature, then subjected to the SDT following the above steps E.2.3 to E.2.8. When specimens cannot be tested immediately, they can be stored in a freezer in order to stop expansion. The specimens have to be thawed for 24 hours at room temperature, unwrapped, and then subjected to the SDT.

E.3 References

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[9] It is obvious that exact expansion levels cannot be reached and that all specimens to be tested at the same expansion level will not present exactly the same expansion. The important point is that these specimens be close enough to the desired expansion level, while the results (i.e., exact expansion and SDT result) for each specimen tested are individually reported on the graph of expansion against SDT and used for calculating the calibration curve.

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Appendix F
Evaluation of the Potential for Further Expansion due to ASR (Prognosis)
Procedures for Expansion Tests on Concrete Cores

F.1 Introductory Remarks

The potential for further expansion due to ASR is an important parameter in the process of assessing the current condition of ASR-affected concrete and selecting appropriate remedial actions. The in-situ monitoring of the current deformations is the only accurate method of estimating this potential. The current rates of deformation are measured periodically or continuously, and can be then extrapolated to forecast future behaviors. Moreover, "in-situ" measurements on existing structures often lead to more optimistic results than expansion tests on concrete cores taken from these structures, which are not tested under the same environmental (temperature, humidity, wetting-drying, freezing-thawing, etc.) and stress conditions. However, in-situ monitoring usually requires several years to generate relevant data, i.e., data where the permanent and cumulative deformation due to ASR can be clearly distinguished from the reversible and cyclic movements related to mechanical (loading, traffic, operation condition etc.) and thermal/climatic (daily and seasonal) variations. On the other hand, expansion tests on cores can yield results in a relatively short period of time, e.g., one year. This accounts for their common use in assessing the potential for further expansion of ASR-affected concrete (prognosis), and for diagnosis as well (e.g., confirmation of the expansive character of the concrete under study).

The procedures used for testing cores from ASR-affected structures vary greatly from one study to another (Bérubé et al. 2004). Two test procedures are recommended hereafter, where the cores are: (1), tested in air at > 95 percent R.H. and 38°C (100°F), and (2), tested in 1N NaOH solution at 38°C (100°F). The first test is considered the most realistic for evaluating the “residual concrete expansivity” and the potential for further expansion of ASR-affected concrete (Bérubé et al. 2002). The concrete is tested with its own alkali content and the test conditions used (temperature and humidity) are similar to those used in the Concrete Prism Test ASTM C 1293. The second test in alkaline solution is recommended for determining the “residual aggregate reactivity” in the concrete under study (Bérubé et al. 2002). The results from expansion tests on cores can be used in combination with a number of other parameters for estimating the current rate of expansion of ASR-affected concrete in the field (see Appendix I).

F.2 Sampling

F.2.1 Core diameter

The diameter of the cores should be at least three times the maximum size of aggregates, and their length should be two to three times their diameter. For concrete with aggregates of nominal size smaller than 35 mm (1.4 in), a core diameter of 100 mm (4 in) is the most practical. However, as discussed hereafter, a core diameter of 150 mm (6 in) is always recommended when testing in humid air (> 95 percent R.H. and 38°C (100°F)), which reduces alkali leaching during the test and postpones the consequent leveling off of the expansion curve.

F.2.2 Surface effects and depth of cores

The skin of concrete (from the exposed surface down to about 25 to 75 mm (1 to 3 in) in the concrete) is usually more macro-cracked and may have suffered from significant alkali leaching or, conversely, alkali concentration through evaporation or supply of de-icing chemicals. Therefore, this concrete may be not representative of the ASR-affected member under study regarding the evaluation of potential for further expansion. In particular, wetting/drying and freezing/thawing cycles may have greatly contributed to the development of cracking in the surface concrete. It is recommended that, at least, the first 50 mm (2 in) of concrete be avoided.

F.2.3 Number, location, and orientation of cores

Large variations in mixture proportioning, exposure conditions (humidity, temperature, etc.), stress conditions, and internal deterioration (expansion and microcracking), may occur within a single structure or a single member of a structure, leading to variations in the test results. It is therefore important to take cores from various members of the concrete structure under investigation in order to have a good coverage of such variations. For a given type of concrete member, it is recommended to test a minimum of three specimens in humid air at >95 percent R.H. and 38°C (Section F.7), and a minimum of two specimens when testing in 1N NaOH at 38°C (Section F.8). More than one test specimen can be taken from cores of sufficient length, provided that the above considerations about concrete variability are taken in account.

At the time of coring, it is important to note the orientation of the cores in the concrete member(s) under study and, if possible, to monitor the deformations of the cores during the expansion tests along three directions: the longitudinal direction and two diametrical directions at right angles, with one perpendicular to the casting plane.[10]

F.3 Storage of Cores Before Testing

F.3.1 Initial adjustments

After coring, the volume of the core samples may change to reach a relative equilibrium with respect to the new stress and environmental (temperature and moisture) conditions to which the concrete is now exposed. These short-term variations can be related to:

- Thermal expansion or contraction, whether the temperature in nature was lower or higher than the testing conditions.

[10] Smaoui et al. (2004a, 2007) reported that ASR expansion is greatly reduced in the direction of the main restraints (reinforcements, prestressing, postensioning, loading…), while restraining concrete samples in one or two directions can actually contribute at increasing expansion in the unrestrained (or less restrained) direction(s). Moreover, a number of studies also indicated that the ASR expansion of non-reinforced concrete is greater in the direction perpendicular to the casting plane, which phenomenon is amplified in the presence of flat and elongated reactive aggregate particles (Smaoui et al. 2004b).

- Expansion due to release of restraining stresses due to reinforcement, prestressing, pretensioning, loading, etc. (with the possibility that contraction takes place in the less restrained directions).
- Expansion due to moisture uptake or contraction due to drying shrinkage (particularly if the cores were not sealed after coring), whether the humidity conditions in concrete were lower or higher than in nature.
- Free expansion of ASR gel already present in the core samples, as a result of lower internal stress on this gel with respect to the conditions prevailing in nature.
- Expansion of ASR gel already present in the core samples due to additional water sorption, as a result of higher humidity with respect to the conditions prevailing in nature.

F.3.2 Storage conditions

In order to minimize the effects of drying (shrinkage, microcracking) and expansion due to rewetting at the beginning of the tests, the core samples should be sealed immediately after coring by wrapping them in heavy duty shrink wrap and storing them in sealed polyethylene bags.[11]

It is recommended to wait at least one week before the cores are subjected to expansion tests in order to reach a relative mechanical equilibrium (stress release). However, the volumetric changes taking place during the storage period (which could be related to stress release when the cores are kept moist after coring), could also be monitored by installing gauge reference studs and taking initial length measurements shortly after coring (Section F.4.2).

F.4 Preparation of Cores

F.4.1 Sawing of cores

The core samples are sawn perpendicular to their axis using a diamond saw. As already mentioned, their length should be two to three times their diameter (see Table F1). This step is not absolutely required when only lateral and diametrical measurements are made, but the presence of flat ends facilitates the handling and the storage of the cores.

[11] This also prevents carbonation. It was observed that cores tested in 1N NaOH at 80°C (176°F), which were allowed to dry for one month in the laboratory before being tested, presented higher expansions and mass increases at the beginning of the tests, i.e. during the so-called "preconditioning" period discussed hereafter (Section F.5), than companion cores kept sealed since coring (Bérubé et al. 1994). However, after correction for these short-term effects which are not related to further ASR expansion in the new storage conditions, the expansion results were similar for both series of cores. Nevertheless, it is always recommended that all cores be wrapped immediately after coring in order to prevent drying. This could actually contribute at significantly reducing the duration of the preconditioning period, which can be particularly long (up to 2-3 months) when cores that were allowed to dry have to be tested in humid air.

Table F1. Suggested length of core samples tested as a function of core diameter.

Core diameter (mm)	Recommended length[1] (mm)	Minimum length[1] (mm)
50 (2 in)	150 (6 in)	100 (4 in)
75 (3 in)	225 (11 in)	150 (6 in)
100 (4 in)	300 (12 in)	200 (8 in)
150 (6 in)	300 (12 in)	300 (12 in)

[1] Approximately (max ± 5mm)

F.4.2 Installation of gage reference studs

Stainless steel bolts or gage studs, 13 mm in length by 3 mm in diameter (approximately 0.5 by approximately 0.12 in) with a machined "demec point" at the end are commonly for that purpose. They are installed in small holes drilled dry and cemented with a shrinkage-free cement paste. The drill holes are about 8 mm (approximately 0.3 in) in diameter by 20 mm (0.8 in) deep.

For the reasons mentioned before (i.e., influence of directions of principal stresses and casting plane), the gage studs should be installed in order to monitor the (longitudinal and diametrical) expansion of the cores along the three directions, including one direction perpendicular to the casting plane.

Axial (longitudinal) measurements – They are more recommended than lateral (longitudinal) measurements because they generally result in lower experimental variability (Bérubé et al. 1994, Smaoui et al. 2004b). Gage studs are centrally installed at both ends of the core, and the measurements can be performed using an arch-type device (Figure F1-A). Axial measurements can also be performed with the measuring device used for the Concrete Prism Test ASTM C 1293, provided that appropriate gage studs are fixed at both ends of the cores.

Lateral (longitudinal) measurements – They are also possible, however less recommended than axial measurements because of the larger variability often observed between the individual measurements performed on the same core (Bérubé et al. 1994).

For such measurements, at least two diametrically opposite lines of two gage studs are installed on the cores tested, with 50 to 250 mm (2 to 10 in) between the studs, depending upon the length of the core specimens and the measuring device available (Figure F1-C).[12]

The studs should be placed not less than 20 mm (0.8 in) from the ends of the cores (50 mm (2 in) from the end corresponding to the exposed concrete surface) (see Table F2).

Diametrical measurements – They are performed by installing, at mid-length of the cores, two diametrically opposed studs. Measurements can also be made near (≥ 20 mm) (0.8 in) both ends of the cores (Smaoui et al. 2004b). They are performed using the same arch-type measurement device as for axial measurements (Figure F1-B).

[12] BCA (1992) recommends that nine gage studs be fixed to each core along three equally spaced lines with two 50 mm (2 in) gage lengths each (6 readings per core).

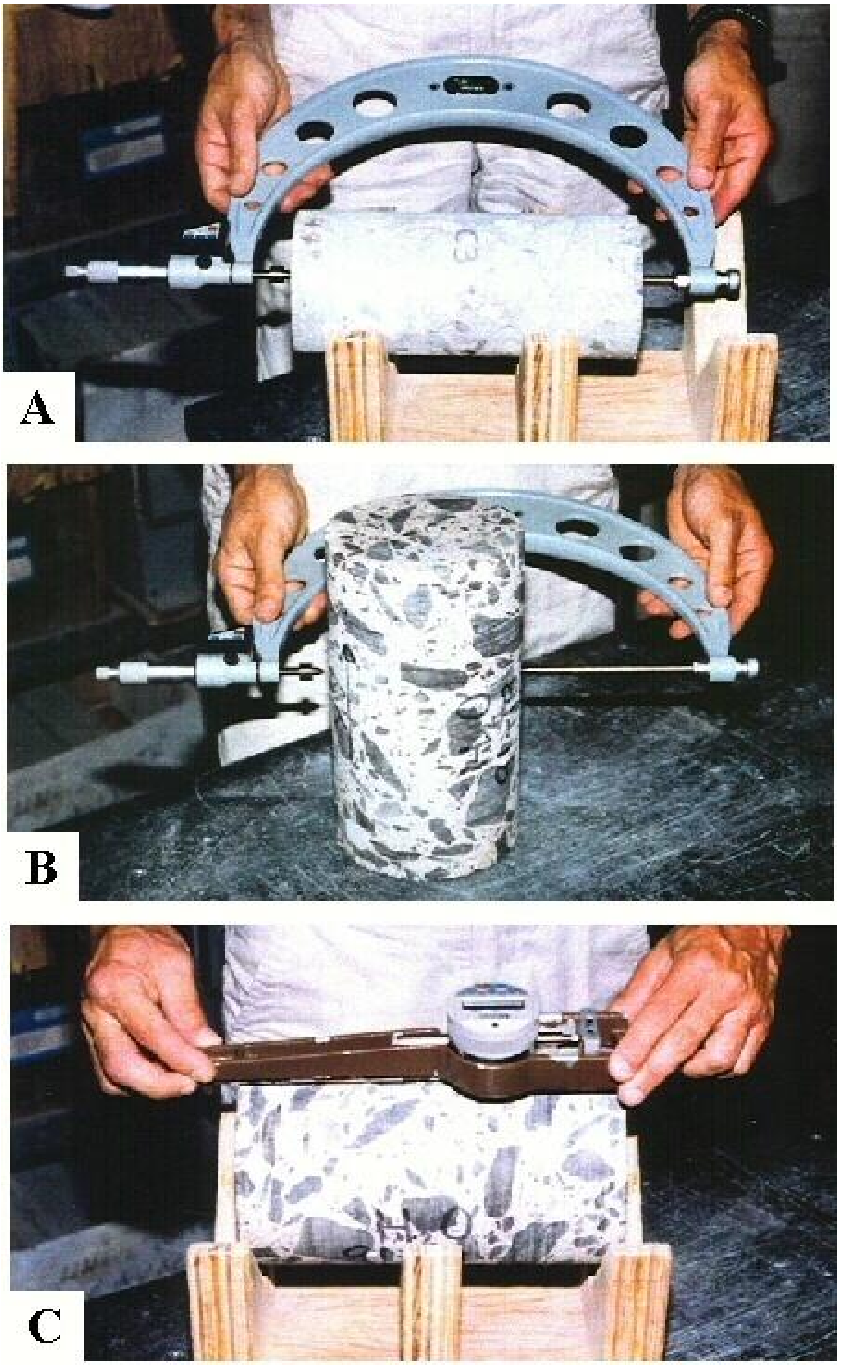

Figure F1. Expansion measurements. A) Axial reading using an arch-type measuring device. B) Diametrical measurement using the same measuring device. C) Lateral reading using standard conventional length-change measuring device used for freeze-thaw / creep testing.

Table F2. Suggested distance between gage reference studs for lateral measurements on cores.

Length of cores (mm)	Distance between studs (mm)	Distance between studs and end portions of cores (mm)
90 – 140 (0.36 – 5.6 in)	50 (2 in)	20 – 45 (0.8 – 1.8 in)
140 – 190 (5.6 – 7.6 in)	100 (4 in)	20 – 45 (0.8 – 1.8 in)
190 – 240 (7.6 – 9.6 in)	150 (6 in)	20 – 45 (0.8 – 1.8 in)
240 – 290 (9.6 – 11.6 in)	200 (8 in)	20 – 45 (0.8 – 1.8 in)
≥290 (≥ 11.6 in)	250 (10 in)	≥ 20 (≥ 0.8 in)

F.5 Early Thermal and Moisture Adjustments ("Preconditioning")

When testing length changes of cores, a preliminary expansion phase is observed due to various mechanisms unrelated to the residual/further ASR expansion (see Section F.3.1):

- Thermal expansion (concrete), since the test temperature (38°C) (100°F) is normally higher than the temperature at which the core samples were stored before being tested.
- Stress release (concrete and preexisting ASR gel) when the samples are tested too early after coring, as a result of lower stress with respect to the conditions prevailing in nature.
- Moisture uptake (concrete and preexisting ASR gel), since the test humidity conditions are normally higher than in nature (cores sealed immediately after coring), or if the core samples have been allowed to dry since coring. In some cases, the expansion due to rewetting may reach 0.1 percent or more (e.g. about 0.04 percent in Figure F2-B), while AAR expansion afterwards might be negligible.

The only way to properly account for these short term variations consists in conditioning the samples under the same conditions as for the tests, until a relative equilibrium in mass and expansion is reached. Then, the time and expansion scales are reset to zero (see Figure F2). The time at which this equilibrium is reached is recognized by the presence of points of inflexion on corresponding mass and expansion curves. The points of inflexion are sometimes difficult to locate precisely, particularly on the expansion curve, but it is better to take the zero reading later rather than too early; the most critical point is to ensure that all initial adjustments are completed; otherwise the expansion due to residual/further ASR would be overestimated. On the other hand, exceeding the equilibrium point just delays the completion of the tests.

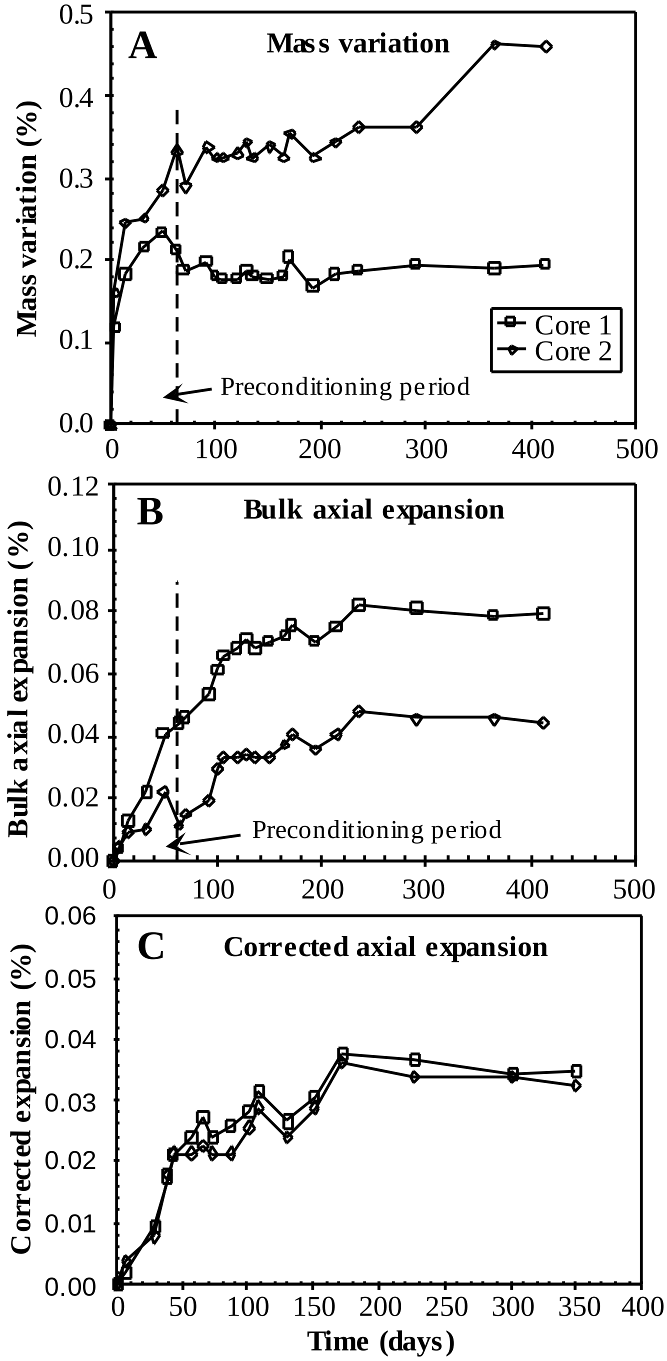

Figure F2. Results from expansion tests in air at > 95 percent R.H. and 38°C (100°F) on core samples taken in the abutment of a bridge located in the Québec City area and incorporating a siliceous limestone coarse reactive aggregate. A) Mass variation. B) Measured bulk axial expansion. C) “Reset” (after removing the portion related to the preconditioning) axial expansion after a preconditioning of 60 days.

F.6 Periodic Measurements (Expansion, Mass and Temperature)

In order to recognize more easily the above short term volumetric changes unrelated to the residual/further ASR, the mass of the core samples is measured in addition to their volumetric changes (diameter, length). Moreover, the temperature of the immersion solution (test in 1N NaOH) or the storage room (test in humid air) at the time of measurement is also recorded in order to normalize all length values with respect to the nominal test temperature of 38°C (100°F). The correction factor used is 0.001 percent/°C which is an estimate of the thermal expansion coefficient of conventional concretes. The temperature correction is particularly critical when testing cores in humid air at > 95 percent R.H., because small expansions are generally obtained (Bérubé et al. 1994).

F.7 Expansion Test in Air at > 95 percent R.H. and 38°C

F.7.1 Experimental procedure

- The initial mass and dimensions (length, diameter) of the core samples are first measured.
- The core samples are then placed vertically above water (> 95 percent R.H.) in sealed plastic pails with wicks inside (or in alternative containers), which are stored in a room (or in alternative reactor) maintained at 38°C (100°F). These conditions are those used in the Concrete Prism Test ASTM C 1293.
- Expansion and mass measurements are taken one day after at > 95 percent R.H. and 38°C (100°F), as thermal equilibrium is attained (see Section F.5). Periodic measurements of expansion, mass and temperature of the storage room are then taken after 1, 2, 3, 4, 6, 8, 10, 12, 16, 20, 24, 30, 36, 44 and 52 weeks, then once every 2 months, if necessary.
- The short-term length-change results obtained are analyzed in order to determine at what time the core samples reached a relative equilibrium in mass and expansion (after hygrometric equilibrium, which also includes potential additional expansion of preexisting ASR gel). The preconditioning period may range from a few days to a few months, depending upon the storage conditions before testing (e.g., drying). The time and expansion scales are then reset to zero (Figure F2).
- The measurements are made until one year at least after the preconditioning period, and even further when the one-year expansion is relatively low (e.g., < 0.025 percent).
- The specimens are always measured hot. Immediately after the plastic pail (or alternative container) has been removed from the storage room (or alternative reactor) and after a core sample has been taken out from its container, the longitudinal and diametrical deformations of the core sample are measured, then its mass. This overall process should take less than one minute. Because of the low expansion rates usually obtained, it is highly recommended that the diameter and length measurements be corrected for the actual temperature in the storage room (or reactor) at the moment of measurement, as described before.
- The final result is expressed as the average annual rate of expansion over the testing period, excluding the preconditioning period. Linear regression analysis is recommended in order to determine the annual rate of expansion with a better accuracy.

F.7.2 Expansion limit criterion

Because the alkali content of field concrete is normally lower than for laboratory concrete made in accordance with the Concrete Prism Test ASTM C 1293, relatively low expansions are usually obtained. However, an expansion as low as 0.003 percent per year, which is the lower limit considered in the methodology proposed in the Appendix I, may be of great importance for the existing structure under study. In fact, expansion rates in the range of 0.002 to 0.005 percent per year are common in the case of AAR-affected concrete structures (CSA 2000).[13]

On the other hand, very low expansions, e.g., ≤ 0.005 percent per year, are not statistically significant considering the experimental variation despite the reading precision of the expansion measurements is about ± 0.0005. In order to improve the statistical significance of the results, (1), the tests are often extended over the usual one-year period; (2), the measurements are more frequent than in the standard Concrete Prism Test ASTM C 1293, and (3), linear regression analysis is recommended in order to better assess the annual rate of expansion.

F.7.3 Interpretation of the test results

The expansions measured in the tests may have been underestimated if the concrete specimens tested were abnormally fractured or porous compared to the overall concrete member under investigation. In such a case, some ASR gel produced during the test may have expanded freely in voids without causing additional expansion. The interpretation of the test results is thus not always easy. Table F3 may be useful in this regard. [14]

[13] For instance, in the study by Bérubé et al. (2002b) on the effectiveness of sealers against the ASR developed in median barriers located in Québec City (Québec, Canada), an expansion rate of 0.005 percent per year has been measured for the unsealed control sections of barriers.
[14] One can note in Table F3 that the petrographic examination of the concrete cores before the tests (cracking, primary reaction products, etc.) the measurement of the water-soluble alkali content of the concrete, the verification of the presence of secondary reaction products after the tests, and the expansion results obtained for companion cores immersed in 1N NaOH at 38°C (100°F) (section F.8) may greatly help in the interpretation of the results of tests in humid air.

Table F3. Interpretation of expansion tests on cores in humid air at > 95 percent R.H. and 38°C (100°F).

1-yr exp.[1] (percent)	Case	Conclusion about the concrete	Preexisting ASR gel (before test)	Secondary ASR gel (after test)	Result for companion cores tested at 38°C (100°F) in 1N NaOH	Further expansion expected in the field
<0.003	1	Non-expansive since construction	No/small amounts	No/small amounts	Non-expansive (non-reactive aggregates) or expansive (low alkali concrete content)	Non-expansive (non-reactive aggregates or low-alkali content)
	2	Non-expansive anymore	Yes	No/small amounts	Non-expansive (aggregates non-reactive anymore) or expansive (alkali content not sufficiently high anymore)	Non-expansive (aggregates non-reactive anymore or alkali content not sufficiently high anymore)
	3	Reactive but non-expansive (cracked/porous concrete)	Yes or no depending on age and humidity	Yes	Non-expansive (porous/ cracked concrete) or expansive (concrete not as cracked)	Expansive or non-expansive depending on porosity/ cracking, humidity, and confinement
>0.003	4	Expansive	Yes or no depending on age and humidity	Yes	Expansive	Expansive or non-expansive depending on humidity and confinement

[1] Excluding the preconditioning period. The expansion results obtained could be underestimated if the concrete tested is significantly more porous/cracked (open spaces for free expansion of the ASR gel) than the overall concrete of the field member under study.

Nevertheless, significant expansion in this test does not necessarily mean that it will be the case in the corresponding structure, as the humidity conditions may not be sufficiently high in nature or the concrete may be subjected to “beneficial” compressive stresses.

Alkali leaching and influence of core diameter - Considering the relatively low expansion rates obtained for most field concretes tested and the necessity to often perform the test up to 2 years, it is highly recommended to test cores of 150 mm (6 in) in diameter, in order to minimize alkali leaching and to prevent the expansion to level off and to be underestimated[15] (see Figure F3).

[15] Concrete prisms made in accordance with ASTM C 1293 (i.e. 75 by 75 by 275 to 405 mm in size) (3 by 3 by 11 to 16 in) suffer significant alkali leaching when tested in humid air at 38°C (100°F) (Rogers and Hooton 1993, Rivard et al. 2003). In a particular study (Bérubé et al. 1994, Landry 1994), prisms of 56 by 56 by 300 mm (2 by 2 by 12 in) and of 75 by 75 by 300 mm (3 by 3 by 12 in), and cylinders of 150 by 300 mm (6 by 12 in) and of 255 by 300 mm (10 by 12 in), were tested in air at 38°C (100°F) and > 95 percent R.H. The smaller the specimens tested, the higher was the expansion in the short term; this was explained by the easier access of humidity inside the concrete. However, the smaller the specimens tested, the lower was the expansion in the long term (Figure F3); this was explained by more alkali leaching from the smaller specimens, as confirmed by measurements of the water-soluble alkali content of the specimens tested at the end of the tests, using the hot-water extraction method described in Appendix H.

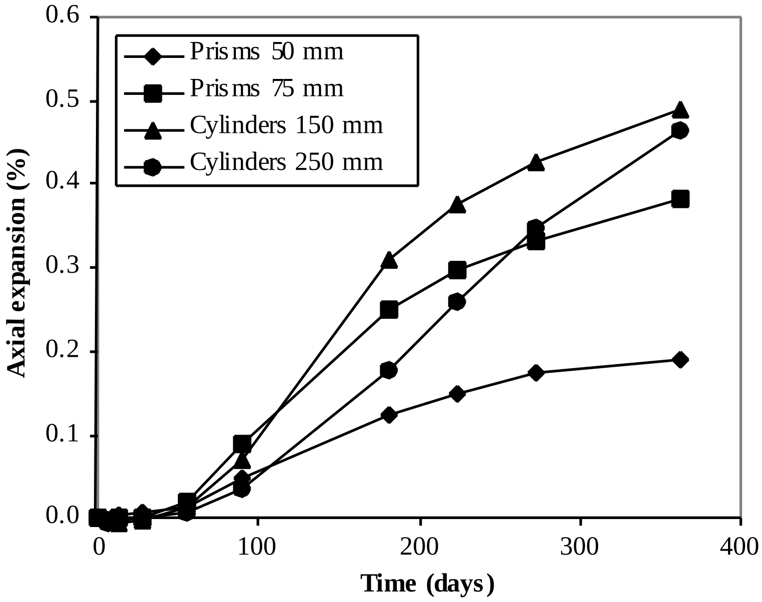

Figure F3. Expansion of concrete specimens of various sizes, made with a rhyolitic tuff and stored in air at 38°C (100°F) and > 95 percent R.H. (After Landry 1994).

Determination of the ultimate expansion in the field - According to the ISE (1992), the maximum AAR expansion that a concrete member may attain in the field corresponds to the maximal expansion obtained for concrete cores tested in humid air (> 95 percent R.H.) at 38°C (100°F). However, the maximum expansion is likely to be underestimated this way since cores suffer significant alkali leaching under the above test conditions; consequently, the expansion tends to level off after a certain time, which is not really due to consumption of reactive mineral phases or alkalies (Rogers and Hooton 1993).[16]

In this respect, the greater the core diameter, the lesser the alkali leaching during the expansion test in humid air and the higher the expansions in the long term (Bérubé et al. 2004, Landry 1994). The alkali leaching during expansion tests in humid air is also influenced by the concrete permeability (i.e., water-to-cement ratio), which varies from one concrete to another.

F.8 Expansion Test in 1N NaOH at 38°C (100°F)

F.8.1 Experimental procedure

- The initial mass and dimensions (length, diameter) of the core samples are first measured.
- The samples are then immersed in a 1N NaOH solution in sealed plastic pails (or alternative containers), which are stored in a room (or in alternative reactor) maintained at 38°C (100°C).

[16] This is clear from Figure F4 where the expansion results obtained for concrete cores taken in concrete blocks (230 by 230 by 810 mm in size) (9 by 9 by 32 in) having reached different expansion levels, are compared with the expansion of a companion block. Whatever the expansion level at which the blocks were cored, the expansion of the core samples leveled off after a relatively short period of time while the companion block continued to expand at a regular rate.

- Expansion, mass and temperature (solution) measurements are performed as when testing in air at > 95 percent R.H. (Section F.7.1). When the core samples are removed from the hot immersion solution, the excess solution is removed with a wet cloth before the expansion and mass measurements are performed.
- The preconditioning period before reaching a relative equilibrium in expansion and mass normally requires less than one week in NaOH. The time and expansion scales are then reset to zero. Again, it is better to exceed the equilibrium point than to be too early.
- The final result is expressed as the expansion obtained one year after the preconditioning period.

F.8.2 Expansion limit criterion

The proposed expansion limit criterion for distinguishing between innocuous and deleteriously expansive concrete when testing in 1N NaOH solution at 38°C (100°F) is 0.04 percent at one year after the preconditioning period (Bérubé et al. 1994).

F.8.3 Interpretation of the test results

Expansions in alkaline solution are affected by numerous parameters, such as the core diameter, the concrete alkali content, the water-to-cement ratio, the concrete permeability, and the extent in preexisting cracking (Bérubé et al. 1994). In particular, (1) the larger the core diameter (longer delay before the test solution impregnates the core samples), (2) the lower the alkali content (slower equilibrium between the initial concrete pore solution and the immersion solution), and (3) the lower the water-to-cement ratio (lower permeability and slower penetration of the alkaline solution), the lower is the expansion in the short term. However, in the long term, e.g., after one year, the influence of these three parameters is in general not as important, provided the water-to-cement is not too low. [17]

The interpretation of the test results is not always easy. Table F4 may be useful in this regard. One can note that the petrographic examination of the concrete before the tests (cracking, primary reaction products, etc.), the measurement of the water-soluble alkali content (Appendix H), the verification of the presence of secondary reaction products after the tests, the chemical analysis (dissolved silica) of the immersion solution after the tests[18], and the expansion results obtained for companion cores stored at > 95 percent R.H. and 38°C (100°F) (Section F.7) may greatly help in the interpretation of the test results. It must be noted that the K/Na ratio of the secondary reaction products formed during the immersion test in NaOH is generally significantly lower than in the primary products, due to the presence of NaOH, since most cements contain a larger proportion of potassium than sodium.

[17] When the water-to-cement ratio is in the range of 0.30 or less, the concrete almost behaves as a closed system such as the expansion is mostly controlled by the initial concrete alkali content, exactly like if the test was performed in humid air (Bérubé et al. 1994).
[18] It might be that residual/further ASR develops during the test without significant additional expansion if the concrete was already presenting a lot of cracks where the ASR products might have expanded freely or if the reactive silica was just dissolved in the immersion test solution**.** The latter phenomenon has been often observed for concretes incorporating the Potsdam siliceous sandstone (Bérubé et al. 1994, 1996; Pedneault 1996).

Table F4. Interpretation of expansion tests on cores immersed in 1N NaOH at 38°C (100°F).

1-yr exp.[1] (percent)	**Case**	**Conclusion of the test about aggregates**	**Preexisting ASR gel (before test)**	**Secondary ASR gel (more Na) (after test)**	**Results for companion cores tested at 38°C and >95 percent RH**	**Further expansion expected in the field**
<0.04	1	Non-reactive since construction	No/small amounts	No/small amounts	Non-expansive	Non-expansive (non-reactive aggregates)
	2	Non-reactive anymore	Yes	No/small amounts	Non-expansive	Non-expansive (aggregates non-reactive anymore)
	3	Reactive but cracked/porous concrete	Yes or no depending on age, concrete alkali content, and humidity	Yes	Non-expansive (porous/ cracked concrete or low-alkali content) or expansive (concrete not as cracked)	Expansive or non-expansive depending on porosity/cracking, alkali content, humidity, and confinement
	4	Reactive but silica dissolved in the test solution	Yes or no depending on age, alkali content, and humidity	No/small amounts	Non-expansive (low-alkali concrete content) or expansive	Expansive or non-expansive depending on alkali content, humidity, and confinement
	5	Reactive but very-low permeability concrete	Yes or no depending on age, alkali content, and humidity	No/small amounts	Non-expansive (low-alkali concrete content) or expansive [2]	Expansive or non-expansive depending on alkali content, humidity, and confinement
>0.04	6	Reactive	Yes or no depending on age, alkali content, and humidity	Yes (higher K/Na ratio)	Non-expansive (low-alkali content) or expansive	Expansive or non-expansive depending on alkali content, humidity, and confinement

[1] Excluding the preconditioning period. The expansion results obtained could be underestimated if the concrete tested is significantly more porous/cracked (open spaces for the ASR gel expand freely) than the overall concrete of the field member under study.

[2] Considering the low concrete permeability, the expansion results should be similar in 1N NaOH and in humid air.

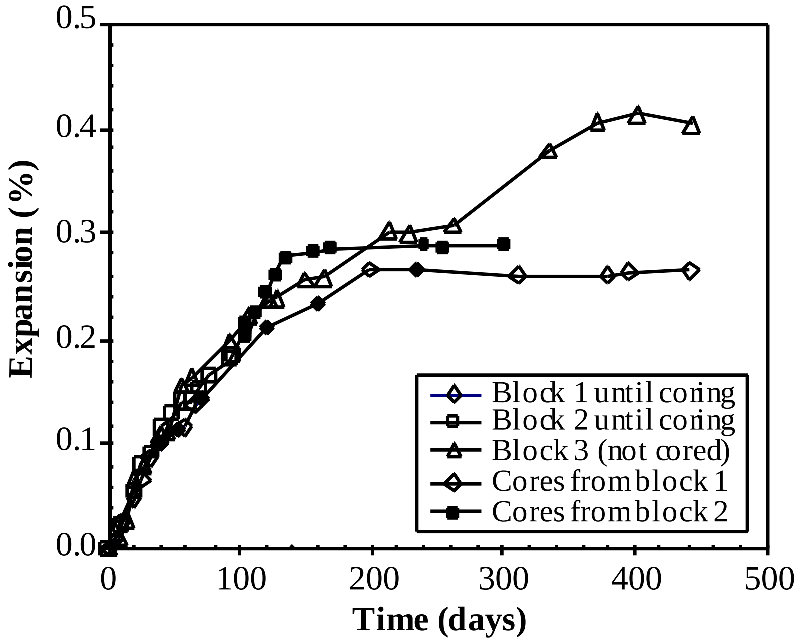

Figure F4. Expansion results in humid air at 38°C (100°F) and > 95 percent R.H. for three non-reinforced concrete blocks, 230 by 230 by 810 mm (9 by 9 by 32 in) in size, made with an extremely-reactive gravel from New Mexico, and for cores, 100 mm (4 in) in diameter, taken in two of these blocks at different times. (Axial measurements on cores and transverse measurements on blocks along the same direction as the core axis).

Immersion tests in 1N NaOH solution can be useful to assess the residual reactivity of the aggregates present in the concrete under study. However, significant expansion in the tests cannot be directly used to estimate the potential for further expansion in the existing structure. The immersion solution used is highly basic and alkaline and the humidity conditions are extreme. Despite the presence of highly-reactive aggregates, the concrete might not expand in the existing structure if the two other essential conditions for ASR, i.e. high humidity and high concentration of alkali hydroxides (or pH) in the concrete pore solution, were never or are no longer satisfied in the field. Also, compressive stresses in the field tend to reduce the expansion due to ASR.

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Appendix G
Determination of the Current Rate of ASR Expansion
Test Procedure for Extraction of Coarse Aggregates from Concrete Cores

G.1 Introductory Remarks

In the procedure proposed by Bérubé et al. (2002) and described in the Appendix I, the current rate of ASR expansion in field concrete can be estimated in the laboratory, based on the "residual aggregate reactivity" and the "water-soluble alkali content" (see Appendix H) measured for the concrete under study, combined with estimates/measurements of the temperature, humidity and restraining (compressive stress) conditions prevailing in the field (see Appendix I).

The "residual aggregate reactivity" can be determined by testing cores in 1N NaOH solution at 38°C (100°F) or, better for coarse reactive aggregates, by conducting Concrete Prism Tests ASTM C 1293 on aggregates extracted from the cores. This extraction can be made using the procedure first proposed by Grattan-Bellew and Danay (1992) (also described in Grattan-Bellew 1995), which consists of rapid cycles of freezing in liquid nitrogen and thawing in a microwave oven. The procedure was revisited by Pedneault (1996) who established the standard procedure described hereafter. It must be mentioned that only the coarse aggregates can be isolated using this procedure, which is then not useful when the reactive aggregate material belong to the sand fraction.

G.2 Test Procedure

G.2.1 Sample preparation

The mass of concrete necessary to recover enough coarse aggregate particles to make three concrete prisms, in accordance with the Concrete Prism Test ASTM C 1293, varies with the nature of the coarse aggregates, their maximum size, and the pre-existing degree of concrete deterioration. For concretes containing coarse aggregates in the range of 5 to 20 mm (approximately 0.25 to 0.75 in), Pedneault (1996) obtained recoveries ranging from less than 20 percent (for highly friable reacted Potsdam sandstone particles of 5 to 10 mm (approximately 0.25 to 0.38 in) in size), to almost 100 percent (for unreacted limestone particles of 14 to 20 mm (approximately 0.50 to 0.75 in) in size).

The concrete sample is broken with a hammer in order to obtain individual fragments of less than 2 kg. After crushing, the sample is immersed in tap water at 23°C (73°F) for at least 36 hours.

G.2.2 Freeze-thaw cycles

The concrete sample (or a sub-sample) is placed in a stainless steel mesh basket and immersed in liquid nitrogen for 25 minutes, using a solid/liquid ratio of 0.40 or less. In order to minimize evaporation, it is recommended to put the liquid nitrogen in well insulated steel container. Despite this, it should be expected that about half of the volume of liquid nitrogen might evaporates by the end of the immersion period of 25 minutes.

The stainless steel mesh basket containing the concrete sample (or sub-sample) is pulled out of the liquid nitrogen, and the concrete sample (or sub-sample) placed in a microwave oven, set at maximum power for 35 minutes (maximum mass of 4 to 5 kg (9 to 11 lbs) of concrete per batch).

Freezing and thawing cycles are repeated until the sample (or sub-sample) is completely cracked. A minimum of four cycles is generally required for good quality concretes while only one cycle may be necessary in the case of severely deteriorated concretes.

G.2.3 Aggregate extraction and cleaning

The coarse aggregates are manually separated from the concrete fragments using a small hammer. Additional wetting/freezing/drying cycles (Sections G.2.1 and G.2.2) may be required at this stage if the coarse aggregate particles are still difficult to extract.

The extracted particles are cleaned of any residual adhering cement paste by immersion for 30 seconds in diluted hydrochloric acid (5 percent).

G.3 Concrete Expansion Test ASTM C 12935

The Concrete Prism Test ASTM C 1293 can be conducted using the extracted aggregate particles as the coarse aggregate (see Notes 1 and 2).[19, 20]

G.4 References

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[19] Since the recovery of aggregate particles in the range of 5–10 mm (approximately 0.25 to 0.38 in) is more difficult, it may be necessary to crush the coarsest extracted particles to satisfy the requirements of the test method ASTM C 1293 for which equal masses of particles of sizes 5–10 (approximately 0.25 to 0.38 in), 10–14 (approximately 0.38 to 0.50 in),, and 14–20 mm (approximately 0.50 to 0.75 in) are required.

[20] It is recommended that the ASTM test be performed using the original fine aggregate, if still available, or a similar fine aggregate.

Aggregate Reactions in Hydroelectric Plants and Dams, eds. Canadian Electrical Association & Canadian National Committee of the International Commission on Large Dams, Fredericton (Canada), 23 p., 1992.

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Appendix H
Evaluation of the Potential for Further Expansion due to ASR (Prognosis)
Test Procedure for Determination of the Water-Soluble Alkali Content of Concrete

H.1 Introductory Remarks

H.1.1 Alkali concentration in the pore solution and effect of ASR

The OH^- ions are the aggressive species towards the aggregates; however, they are supplied by the portlandite $Ca(OH)_2$ to reach about equilibrium with the alkali cations in the pore solution. Consequently, the higher the alkali concentration in the pore solution, the higher the [OH^-] (then the pH), and the higher the risk for AAR. In the absence of supplementary cementing materials, the higher the cement alkali content and the lower the water-to-cement ratio (lower amount of pore water), the higher the alkali and OH^- concentrations in the pore solution.

In the presence of ASR, reaction products are formed that contain silica, calcium, and alkalis, which were originally present in the pore solution. As ASR progresses and more reaction products are formed, the alkali concentration in solution progressively decreases, reducing the [OH^-] and the intensity of ASR. Some experiments suggest that ASR expansion in the presence of highly reactive natural aggregates can be minimized when the alkali concentration in the pore solution falls under 0.6N [NaOH+KOH] in the long term (Duchesne and Bérubé 1994a). Therefore, the precise determination of this concentration could be useful for determining whether or not this concentration is sufficient for ASR be maintained, and may facilitate decisions on the measures of mitigation/remediation.

H.1.2 Water-soluble alkali content of concrete

It is very difficult to extract pore solution from old concrete, which could then be chemically analyzed for alkali and OH^- concentrations (Bérubé and Tremblay 2004). A more common but indirect method consists in determining the active- or water-soluble alkali content of concrete, on a kg/m^3 (or lb/yd^3) Na_2O_{eq} basis, by subjecting a representative ground sample of the concrete under study to the hot-water extraction method first proposed by Rogers and Hooton (1993). These alkalis were present in the concrete pore solution before the sample was dried before testing. The alkali and OH^- concentrations in the original pore solution could then be calculated from the water-soluble alkali content (see Section H.3). Bérubé et al. (2002a) applied the hot-water extraction method to a number of concretes incorporating various types of aggregates showing different degrees of alkali-silica and alkali-carbonate reactivity, thus allowing determining the potential alkali contribution by the aggregates themselves during the test. The procedure described in this appendix is based on the results and the recommendations of this study.

H.1.3 Distinction between total and water-soluble alkalies

Limiting the total-alkali content in concrete is considered to be an effective way of minimizing concrete expansion in the presence of aggregates susceptible to ASR (see *Report on Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction,* Thomas, et al. 2008). However, a clear distinction must be made between total alkalis (often restricted to the total amount of alkalis in the cement) and water-soluble alkalis (as measured in the hot-water extraction method). In fact, a significant amount of the cement alkalis are incorporated in the cement hydrates and, in the presence of ASR, in the products from this reaction. Previous studies (Taylor 1990, Duchesne and Bérubé 1994b) indicate that hydrates from ordinary portland cement (OPC) contain about 0.4 to 0.5 percent Na_2O_{eq}. The results by Diamond (1989) and Duchesne and Bérubé (1994b) also suggest that the alkali content in hydrates from OPC is about proportional to the total amount of alkalis contained in the cement used. Based on these findings, one considers that about 40 percent of the total alkalis from the cement are incorporated in hydrates from OPC concrete. In such a situation, 3 kg/m^3 (5lb/yd^3) of total Na_2O_{eq}, for instance, would correspond to about 1.8 kg/m^3 (3 lb/yd^3) of water-soluble Na_2O_{eq} (i.e., 60 percent x 3 kg/m^3) (60 percent x 5 lb/yd^3).

H.1.4 Usefulness and limitations of the hot water extraction method

The water-soluble-alkali content of concrete is a useful parameter for determining whether or not the concrete tested contains sufficient alkalis for ASR to develop (diagnosis) or to be sustained (prognosis). In the approach proposed in the Appendix I the water-soluble alkali content is used, in combination with other relevant information (e.g., expansion tests on cores, environmental and stress conditions in the field), to estimate the current expansion rate in ASR-affected concrete structure; the higher the water-soluble-alkali content, the higher the value calculated for the current expansion rate.

Nevertheless, the experimental variability of the test method is relatively high, the estimated coefficient of variation ranging between 10 and 15 percent, even when using a control concrete (Bérubé et al. 2002a).

H.2 Test Procedure

H.2.1 Sampling of concrete and storage before testing

The water-soluble alkali content may vary considerably within a given structure and even within a single member (Bérubé et al. 2002b). This may be due to variations in the concrete mixture proportioning, the extent of ASR (which consumes alkalis) and cracking (alkali concentration through evaporation or alkali leaching along cracks), and in the exposure conditions which may greatly affect the water-soluble alkali content in the near-surface concrete (external sources of alkali, alkali migration in/out of the sampled zones, alkali concentration at the surface through evaporation, or alkali leaching by rain). It is then recommended that the test method be applied to concrete samples, usually cores, taken in different members exposed to various conditions. Moreover, the concrete close to the surface should be discarded or analyzed separately, as this concrete is more susceptible to contamination by external sources of alkalis, or alkali leaching by rain, particularly if it is cracked or very porous, or to alkali concentration through evaporation.

The concrete samples should be wrapped with plastic film/sheets and placed in sealed plastic bags immediately after coring, to prevent any drying (or wetting).

H.2.2 Preparation of concrete samples

A representative sample of minimum 2 kg (4.5 lbs) of concrete should be crushed with a hammer to < 25 mm (1 in) and allowed to dry immediately before testing. Density and water content measurements could be performed at the same time to further estimate the alkali concentration in the concrete pore solution (in mole/liter of NaOH+KOH), to which the extent of the reaction is more directly related (see Section H.3).

After drying, the concrete sample is progressively crushed and ground to pass the no. 100 sieve (< 160 µm).[21] Two 10-g (0.02 lbs) representative sub-samples are taken for the test.

Two 10-g representative sub-samples of a control concrete with a well-known water-soluble alkali content are also prepared and tested in parallel.[22,23]

H.2.3 Hot-water extraction

Each 10-g (0.02 lbs) sub-sample of the concrete under study and the control concrete are individually immersed in 100 mL of distilled water. The distilled water is boiled for 10 minutes and the suspension is allowed to stand overnight at room temperature. The next morning, the suspensions are filtered and the solution volume topped to 100 mL by adding distilled water.

H.2.4 Chemical analyses and expression of results

The sodium and potassium concentrations in the solution are determined by flame photometry or atomic absorption in accordance with Clause 17.1 ("Total Alkalis") of ASTM C 114. The results are expressed in %Na_2O, %K_2O, and %Na_2O equivalent per kg of concrete and averaged for each concrete tested (sample and control).[24] The average result for each concrete, in %Na_2O_{eq}, is

[21] Grinding to <160 rather than <80 µm in the original procedure (Rogers and Hooton 1993) is easier, allows a lower alkali contribution by aggregates, while not significantly affecting the extraction of alkalis in solution before the concrete has dried (Bérubé et al. 2002a).
[22] Testing in parallel a control concrete with a well-known water-soluble alkali content on the basis of which the results are corrected (see Section H.2.5), greatly improves the precision of the test (Bérubé et al. 2002a).
[23] A given ground concrete can be used for control purposes. In order to prevent variations over time, the control concrete sample shall be kept sealed in a plastic bag under vacuum. Despite this, the water-soluble alkali content measurements of control specimens kept as described above were found to decrease slightly with time (Bérubé et al. 2002a); it is thus recommended that a new control sample be ground every 6 months from a representative uncrushed sample of the control concrete. In order to avoid variations in the water-soluble alkali content of the control concrete (from further hydration or ASR), the latter shall consist in a well-hydrated (e.g., kept for at least 90 days in air at >95 percent R.H. and 23^{o}C (73°F) before the first "control" measurement is taken) non-reactive concrete which is wrapped humid in a cling film and stored in a freezer inside an air-tight plastic bag.
[24] For a given sample, the two results should not differ by more than 5 percent of the average.

expressed in kilograms (lbs) of Na_2O equivalent per cubic meter (yd^3) of concrete, provided the corresponding concrete density (ω, in kg/m^3 or lb/yd^3) is known or reasonably estimated[25]:

Water-soluble concrete alkali content (in kg/m^3 or lb/yd^3 of Na_2O_{eq}) = % Na_2O_{eq} x ω / 100

For example, if the concrete sample tested contains 0.041 %Na_2O_{eq} (= %Na_2O + 0.658 x %K_2O from the chemical analysis) and its density is 2 400 kg/m^3 (4045 lb/yd^3) (measured or assessed), the calculated water-soluble alkali content is 0.98 kg/m^3 (1.65 lb/yd^3) of Na_2O_{eq} (= 0.041 x 2,400 / 100) (= 0.041 x 4,045 / 100).

H.2.5 Correction with respect to the control sample

The (average) result obtained for the concrete under study is normalized by comparison with that obtained for the control concrete (known water-soluble content), before applying a correction for the contribution by aggregates described in Section H.2.6[26]:

Corrected value $_{sample}$ = measured value $_{sample}$ x (known value $_{control}$ / measured value $_{control}$)

H.2.6 Correction for the contribution by aggregates in the test

The result obtained above is adjusted for the alkali contribution by aggregates during extraction, which greatly varies from one aggregate to another.[27] The water-soluble alkali content measured for a number of different rock types from Canada, are given in Table H1. In absence of any information, reasonable estimates may be obtained from the values given in Table H1.

[25] In absence of any information, a default value of 2,400 kg/m^3 (4,045 lb/yd^3) can be used for the concrete density.
[26] For instance, if the result obtained for the control concrete is 13 percent higher than the actual (well-known) value, the result obtained for the concrete under study is automatically reduced by 13 percent using the above relationship.
[27] Aggregates in concrete may release significant amounts of alkalis in the test, e.g. 0.70 kg/m^3 (1.18 lb/yd^3) Na_2O_{eq} on average for the 17 aggregates tested by Bérubé et al. (2002a); consequently, it is important that the results obtained be corrected for this contribution. A general trend is that the higher the total-alkali content in the aggregates, the higher the absolute amount of alkalis released in the test. The most susceptible rock types are those rich in nepheline (phonolite, nepheline syenite), feldspars (granite, granitic gneisses, rhyolitic tuff, andesite, lithic grave etc.), and clay minerals (shales, clayey limestone etc.). The aggregates with the highest contributions in the test procedure are likely those also supplying, with time, the highest amount of alkalis to the pore solution of the concrete during its service life (Bérubé et al. 2002b, Bérubé and Fournier 2004).

Table H1. Alkali contribution of aggregates from Québec and Ontario (Canada) in the Hot-Water Extraction Method. (After Bérubé et al. 2002b, Bérubé and Fournier 2004)

Aggregate	Degree of ASR	Water-soluble content in the aggregate (percent)			Potential Na_2O_{eq} contribution to concrete (kg/m^3)	
		Na_2O	K_2O	Na_2O_{eq}	coarse or fine [1]	coarse + fine [2]
Spratt siliceous limestone	High ASR	0.013	0.011	0.021	0.22	0.47
Rhyolitic tuff	"	0.024	0.033	0.046	0.48	0.74
Potsdam sandstone	"	0.011	0.017	0.022	0.23	0.49
Shaly siltstone	"	0.015	0.087	0.072	0.76	1.02
Trenton siliceous limestone	"	0.009	0.012	0.017	0.18	0.44
Sudbury gravel	Moderate ASR	0.025	0.037	0.049	0.51	0.77
Beekmantown dolostone	Marginal ASR	0.016	0.028	0.034	0.36	0.62
Andesite	Non-reactive	0.042	0.015	0.051	0.54	0.80
Pure limestone	"	0.016	0.015	0.026	0.27	0.53
Diorite	"	0.020	0.020	0.033	0.34	0.60
Granitic gneiss	"	0.023	0.030	0.042	0.44	0.70
Granite	"	0.018	0.043	0.046	0.48	0.74
Greywacke	"	0.015	0.032	0.035	0.37	0.63
Phonolite	"	0.119	0.015	0.129	1.36	1.62
Quartzite[1]	"	0.006	0.004	0.009	0.09	0.35
Granitic sand	"	0.019	0.028	0.037	0.26	-
Average	-	0.024	0.028	0.042	0.43	0.70

[1] Based on aggregate factors of 1,050 kg/m^3 for the coarse aggregates and 700 kg/m^3 for the granitic sand.
[2] Contribution by the corresponding coarse aggregate (1,050 kg/m^3) plus the one by the granitic sand (700 kg/m^3).

H.2.7 Interpretation of results

Since about 40 percent of the cement alkalis are incorporated in the cement hydrates in the case of OPC concrete (see Section H.1.3), the (measured) amount of water-soluble alkalis is generally lower than the (calculated) amount of total alkalis from the cement (calculated as the cement factor times the cement alkali content). However, some alkalis may be supplied to the pore solution by other concrete constituents (aggregates, mineral admixtures, mixture water, superplastisizer etc.) and external sources (deicing/anti-icing chemicals, sea water etc.). At the same time, alkalis may be progressively incorporated in the ASR products, migrate in/out of the sampled concrete zones, concentrate by evaporation near the concrete surface, and/or be leached from this surface or along cracks by rain or running water. Most of the above mechanisms will be most active in the outer portions of the concrete members and their effect could thus be minimized by taking concrete samples at sufficient depths inside the concrete members under study, as specified in Section H.2.1. The results should also be interpreted in accordance with the other limitations mentioned in Section H.1.3.

H.3 Alkali Concentration in the Concrete Pore Solution

A realistic estimate of the alkali and OH^- concentrations in the concrete pore solution can be obtained if the original water content (W) of the concrete is known. This can be determined by drying, under vacuum, a representative concrete sample in an oven at 80°C (176°F) until equilibrium, provided that the concrete was not allowed to dry or to absorb humidity since coring (see Section H.2.2). The following relationship is used for calculation:

[Na+K] ≈ [(Na,K)OH] ≈ [OH] = 32.3 x (kg/m^3 of water-soluble Na_2O_{eq}) / (ω x W/100)

where 32.3 is a constant to transform kg/m^3 Na_2O_{eq} mass units to moles/liter [Na+K] concentration units, ω[28] is the concrete density in kg/m^3 (see Note 1) and W is the pore water content of the concrete (in %), as determined by mass loss after drying until equilibrium.

H.4 References

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[28] In absence of any information, a default value of 2,400 kg/m^3 (4,045 lb/yd^3) can be used for the concrete density.

Appendix I
Evaluation of the Potential for Further Expansion due to ASR (Prognosis)
Procedure for Laboratory Assessment of the Current Rate of ASR Expansion in Field Concrete

I.1 Introductory Remarks

The potential for further expansion due to ASR is a critical factor in the process of selecting the most appropriate measures of mitigation/remediation. ASR and associated expansion and deterioration processes will continue in a concrete member as long as the reactive mineral phases within the aggregate particles are not completely consumed, and the two other essential conditions are still satisfied, i.e., high humidity and high alkali concentration in the concrete pore solution. Expansions are also function of temperature and restraint. In-situ monitoring of concrete deformation and movement is the best way to assess the current and future expansion due to ASR in existing concrete members or structures; however, reliable results usually take over three years to obtain, as the ASR contribution needs to be isolated from seasonal variations in temperature and humidity. Laboratory tests on cores generally lead to fairly reliable estimates within 6 to 12 months, and are commonly used to assess the potential for further ASR expansion.

In the procedure proposed by Bérubé et al. (2002) and described hereafter, the risk for further expansion and damage in a concrete member or structure due to ASR, is estimated from the following parameters (Table I1 and Figure I1):

(1) The **"residual concrete expansivity"**, determined in the laboratory by testing core samples in humid air at >95 percent R.H. and 38°C (100°F) (see Appendix F).
(2) The **"residual aggregate reactivity",** determined in the laboratory by testing core samples in 1N NaOH solution at 38°C (100°F) (see Appendix F) or, even better, when the reactive aggregates belong to the coarse aggregates, by extracting these aggregates from cores (see Appendix G) and testing them through the Concrete Prism Test ASTM C 1293.
(3) The amount of **"water-soluble alkalis"** in the concrete, which is estimated using the hot-water extraction method described in Appendix H.
(4) Reasonable estimates of the **humidity**, **temperature**, and **restraints** (reinforcement, prestressing, postensioning, confinement, loading,...) in the field.

The individual risk indices corresponding to each of the above parameters are then combined to estimate the potential/current rate of ASR expansion in the field of the concrete member under study, either already affected by ASR or not.

Table I1 – Classification of the various coefficients proposed to estimate the current rate of ASR expansion in concrete members in service either already affected by ASR or not.

Coefficient RCE - Residual concrete expansivity in the laboratory (core testing in air at >95 percent RH and 38°C (100°F))

percent exp./yr [1]	**Residual exp.** [1]	**RCE** [1]	**percent exp./yr** [1]	**Residual exp.** [1]	**RCE** [1]
< 0.003	negligible	0	0.015 to 0.02	moderate	6
0.003 to 0.005	very low	1	0.02 to 0.025	high	9
0.005 to 0.01	low	2	0.025 to 0.03	high	12
0.01 to 0.015	moderate	4	> 0.03	very high	16

Coefficient RAR – Residual aggregate reactivity in the laboratory
(core testing in 1N NaOH at 38°C (100°F) or Concrete Prism Test ASTM C 1293 on aggregates extracted from cores)

percent exp. at 1 yr [1]	**Reactivity** [1]	**RAR** [1]	**percent exp. at 1 yr** [1]	**Reactivity** [1]	**RAR** [1]
< 0.04	negligible	0	0.12 to 0.20	high	3
0.04 to 0.08	low	1	> 0.20	very high	4
0.08 to 0.12	moderate	2	-	-	-

Coefficient WSA - Water-soluble alkali content of concrete (hot-water extraction method)

kg/m³ (lb/yd³) Na_2O_{eq}	**Alkali content**	**WSA**	**kg/m³ (lb/yd³) Na_2O_{eq}**	**Alkali content**	**WSA**
< 1.0 (0.6)	very low	0	2.0(1.2) to 2.5(1.5)	high	3
1.0(0.6) to 1.5(0.9)	low	1	> 2.5(1.5)	very high	4
1.5(0.9) to 2.0(1.2)	moderate	2	-	-	-

Coefficient HUM - Humidity conditions in service (internal or external)

Internal humidity	**Humidity risk**	**HUM**	**Internal humidity**	**Humidity risk**	**HUM**
< 80 percent RH	very low	0	90-95 percent RH	high	0.75
80-85 percent RH	low	0.25	95-100 percent RH	very high	1
85-90 percent RH	moderate	0.5	-	-	-

[1] Values are considered underestimated if the concrete cores were abnormally fractured or porous compared to the overall concrete member under study (RCE and RAR) or quite impermeable to the alkaline solution (RAR).

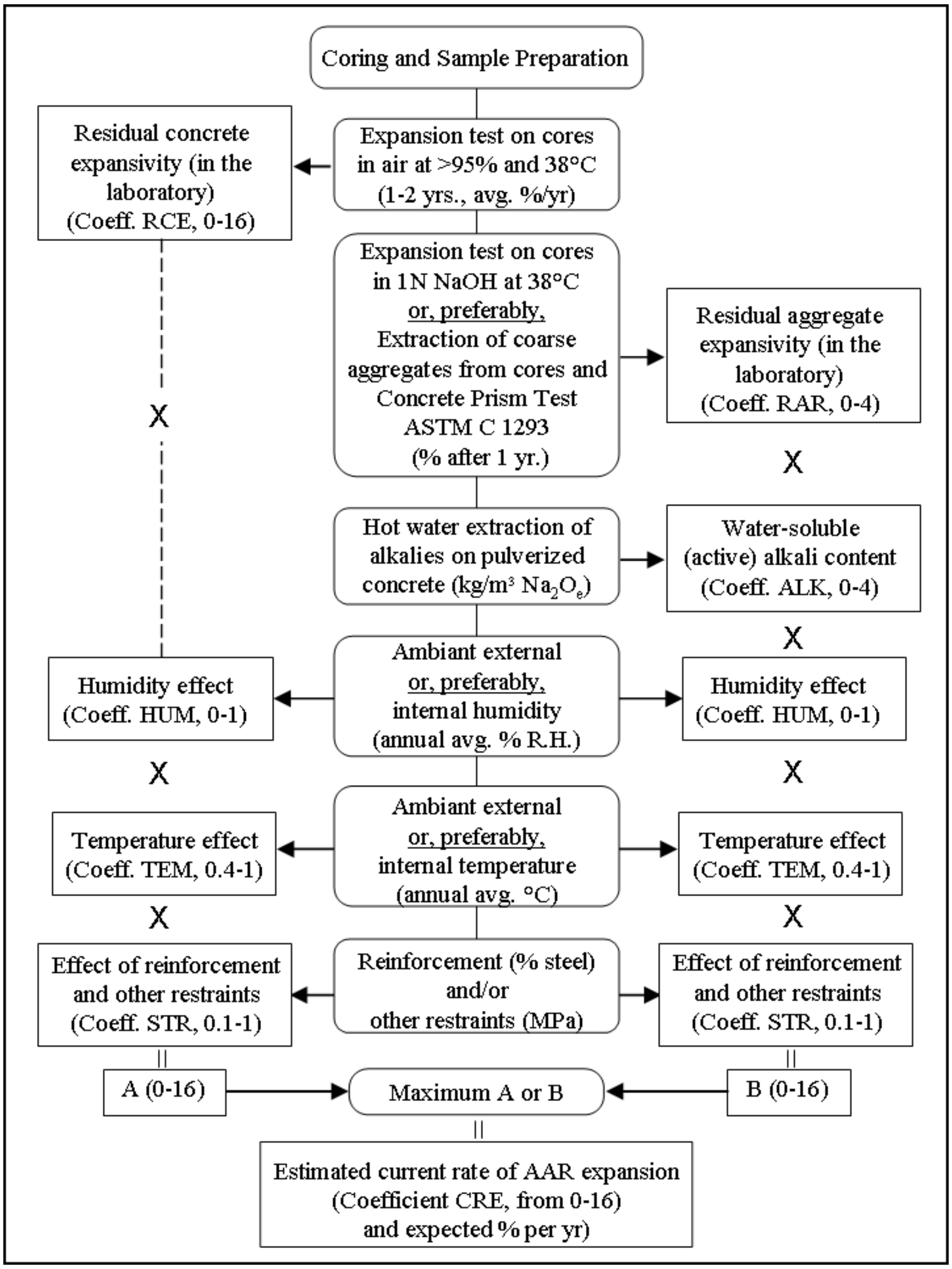

Figure I1. Laboratory assessment of the current rate of ASR expansion in concrete members in service either already affected by ASR or not.

I.2 Test Procedure

I.2.1 Residual concrete expansivity (Coefficient "RCE")

The "residual concrete expansivity" of the concrete under study is determined through expansion tests on cores in humid air at >95 percent R.H. and 38°C (100°F). The test procedure is described in the Appendix F, which also provides insights for the interpretation of the test results;

the method also generally requires an examination of the cores before and after the expansion tests for petrographic signs of ASR. After the concrete cores have reached a mass equilibrium through a preconditioning period, the rate of expansion is calculated over a testing period of one year or more; the expansion rate is then used to determine the coefficient "RCE", which varies from 0 to 16 (Table I1).

An alternative method of estimating the residual concrete expansivity is based on the "residual aggregate reactivity" (coefficient "RAR", ranging from 0 to 4; Section I.3), which is combined with the "water-soluble alkali content" of the concrete (coefficient "WSA", ranging from 0 to 4; Section I.4). The product of the above two parameters varies from a minimum of 0 to a maximum of 16 (Table I1 and Figure I1).

I.2.2 Residual aggregate reactivity (coefficient "RAR")

The "residual aggregate reactivity" is based: (1), on expansion tests on cores stored in 1N NaOH at 38°C (100°F) (see Appendix F), and/or (2), even better when the reactive aggregates belong to the coarse aggregate fraction, on the expansion of concrete prisms incorporating the coarse aggregates extracted from the cores following the Concrete Prism Test ASTM C 1293 (see the Appendix G).

Expansion tests on cores – After the concrete cores have reached a mass equilibrium through a preconditioning period, the one-year expansion value thus obtained is used to determine the coefficient "RAR", which varies from 0 to 4 (Table I1). The interpretation of the results from expansion tests on cores in NaOH solution should also require the petrographic examination of the cores before and after the expansion tests, as discussed in the Appendix F.

Concrete Prism Tests ASTM C 1293 – The one-year expansion of concrete prisms incorporating aggregates extracted from the concrete cores can also be used for determining the coefficient "RAR" (Table I1). One must consider though that the concrete prisms made and tested according to ASTM C 1293 conditions may depart from the field concrete under study with regards to reactive aggregate content, aggregate size, air content, and/or water-to-cement ratio, all parameters which affect the expansion due to AAR (Bérubé et al. 2004).

I.2.3 Water-soluble alkali content (coefficient "WSA")

The test procedure for the measurement of the water-soluble alkali content of the concrete under study, in kilograms of Na_2O_{eq} per cubic meter (or lbs/yd^3), is described in the Appendix H along with some fundamental considerations on the subject. The result of the test is used for establishing the coefficient "WSA", which varies from 0 to 4 (Table I1).

I.2.4 Humidity conditions in service (coefficient "HUM")

It is generally well established that minimum <u>external/ambient</u> relative humidity conditions in the range of 80 to 85 percent are required for excessive expansion due to AAR to develop in concrete. However, according to a number of experimental studies, the less reactive the aggregate, the higher the critical humidity level required for significant ASR expansion to occur

(Bérubé et al. 2002). Moreover, the higher the temperature, the lower this critical level (Olafsson 1987). Also, a clear distinction must be made between internal and external/ambient humidity, with different minimum levels for ASR (Table I1). In a particular study, a minimum level of "internal" humidity of 85 percent R.H. was necessary to sustain AAR in concrete specimens made with very reactive aggregates and high alkali contents, and stored at 38°C (100°F), which corresponded to external/ambient humidity conditions of less than 70 percent R.H. (Bérubé et al. 2002).

Internal humidity conditions in concrete can be periodically or continuously monitored using dedicated instrumentation (see Appendix D). On the other hand, the external/ambient humidity conditions may be continuously recorded by local or regional weather stations. The average yearly internal or external humidity conditions can be determined from such monitoring/ recording processes, otherwise reasonably estimated. The result is then used for establishing the coefficient “HUM”, which varies from 0 to 1 (Table I1). This coefficient takes into account the fact that the humidity inside thick concrete members tend to remain higher than inside thin members, which makes the former less influenced by the external humidity conditions. It also takes into account the type of climate involved (i.e., desertic or not).

I.2.5 Temperature conditions in service (coefficient “TEM”)

The expansion tests described before are performed at a constant temperature of 38°C (100°F); however, concrete structures exposed outdoors can be subjected to fairly different average annual temperatures depending on their location across North America. On the other hand, the higher the temperature, the greater is usually the expansion rate due to ASR, while the ultimate expansion is not necessarily greater.

Temperature inside concrete can be periodically or continuously monitored using dedicated instrumentation (see Appendix D). On the other hand, external temperature may be continuously recorded by local or regional weather stations. The average yearly internal or external temperature can be determined based on the above approaches, otherwise reasonably estimated. The result is then used for establishing the coefficient “TEM”, which varies from 0 to 1 (Table I1).

I.2.6 Restraints in service (coefficient “STR”)

In general, internal (prestressing, reinforcement, etc.) and external (post-tensioning, loading, confinement, etc.) compressive stresses applied to concrete can significantly reduce expansion due to ASR, however not always the surface cracking. It is then necessary to apply a correction factor to the results obtained from expansion tests on cores or prisms (e.g. ASTM C 1293) that are free from restraints. The coefficient of correction “STR” is, however, a difficult parameter to estimate because of the relatively limited amount of relevant information generally available. Moreover, a same level of reinforcement or compressive stress, for instance, is more effective in the presence of a marginally reactive aggregate than with a highly-reactive one (Smaoui et al. 2007). The values proposed in Table I1 must then be considered with circumspection. They correspond to the median results found in a report by the Institution of Structural Engineers (ISE 1992).

When steel reinforcing bars are installed in a single plane in the concrete member (1D or 2D, depending if all bars are parallel or at right angle) or in several parallel planes (2D or 3D, again depending if all bars are parallel or at right angle), but without any anchorage between the different planes, the coefficient "STR" applies in the direction(s) of the reinforcing bars; however, this can result in expansions even higher in the unrestrained directions than when no confinement is applied (Smaoui et al. 2007). Similarly, in the cases of uniaxial (1D) or biaxial (2D) compressive stresses (prestressing, postensioning, loading, confinement,…), "STR" applies in the direction(s) of the stresses while, once again, expansions even higher than when no confinement is applied can develop in the unrestrained direction(s).

I.2.7 Current rate of AAR expansion in service (coefficient "CRE")

The coefficient "CRE" is proposed as an estimate of the current rate of expansion due to AAR in concrete members in service either already affected by ASR or not. This coefficient takes into account all of the above coefficients. As illustrated in Figure I1, it is obtained as follows:

CRE = (maximum [RCE] or [RAR x WSA]) x HUM x TEM x STR

The CRE ranges between 0 and 16. In accordance with Table I1, the current rate of ASR expansion in the concrete member under study can range from negligible to very high. This qualitative rating would be of greater interest/use if translated into an expected current rate of expansion (in %/yr). The information is very limited relating expansion results obtained in the laboratory with those observed in the field. However, after the above methodology had been applied to a number of existing structures in Québec (Canada), it appears realistic to use the same classification as for the above coefficient RCE (Table I1), which corresponds to Figure I2. In other words, the assumption is made that the expansion rate in service (in %/yr) will be somewhat similar to the one observed in the laboratory for cores tested in humid air at >95 percent R.H. and 38°C (100°F) when the following conditions are satisfied in service: (1), very high humidity conditions (i.e., HUM = 1); (2), temperature over 30°C (86°F) (i.e., TEM = 1), and (3), no reinforcement neither other restraints applied to concrete (i.e., STR = 1).

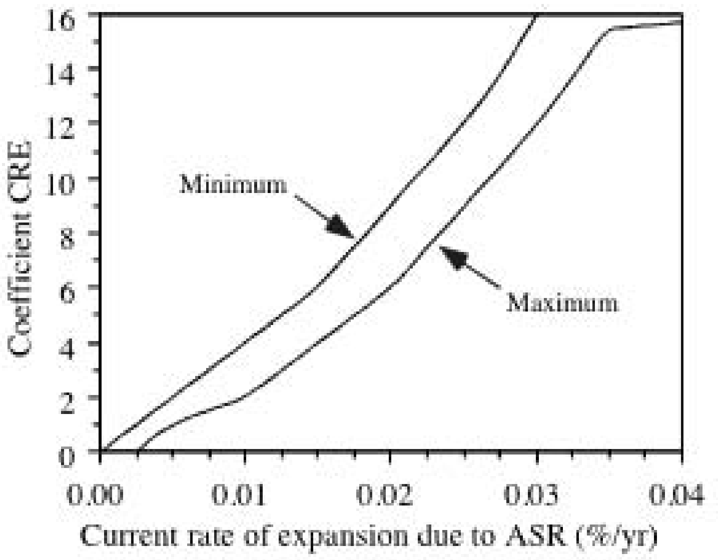

Figure I2. Approximation of the current rate of ASR expansion in concrete members in service either already affected by ASR or not, as a function of the coefficient CRE.

As discussed in the Appendix F, results from expansion tests on cores are considered underestimated if the concrete cores tested were abnormally fractured or porous compared to the overall concrete member under study (coefficients RCE and RAR) or quite impermeable to the alkaline solution (coefficient RAR); this would result in underestimated values for the overall coefficient CRE and the corresponding estimated rate of current expansion in the field.

I.3 Concluding Remarks

The overall coefficient CRE takes into account many parameters which all affect, to some extent, the rates of ASR reaction and expansion. It takes a zero value when at least one of the three necessary conditions for ASR is not satisfied, i.e., when the aggregates are not reactive (i.e., RAR = 0), when the concrete alkali content is low (i.e., WSA = 0), or when the humidity conditions in service are low (i.e., HUM = 0). Also, it can predict the anisotropic expansion in concrete members whose different parts are exposed to different humidity, temperature and/or stress conditions. However, it does not predict for how long the expansion will continue in the concrete structure but just gives an indication of the current (or next future) rate of expansion due to AAR. It must be also emphasized that the coefficient CRE is mostly based on laboratory test results. As mentioned before, long-term in-situ monitoring remains the only reliable way to obtain relevant information on the current rates of expansion, which can then be extrapolated for the next 5 to 10 years.

Table I2 – Classification of the various coefficients proposed to estimate the current rate of ASR expansion in concrete members in service either already affected by ASR or not. (cont)

External (ambient) humidity		Thin member (<0.5m)(0.55yd)		Thick member (>0.5m) (0.55yd)	
		Humidity risk	HUM	Humidity risk	HUM
Indoor	• <70 percent RH	very low	0	low	0.5
	• 70-80 percent RH	low	0.25	moderate	0.75
	• 80-90 percent RH	moderate	0.5	high	1
	• 90-95 percent RH	high	0.75	very high	1
	• 95-100 percent RH or immersed	very high	1	very high	1
Outdoor in deserts	• Not in contact with the ground	very low	0	moderate	0.5
	• In contact with the ground	low	0.25	high	0.75
Outdoor in other areas in North America					
• Not exposed to rain nor in contact with the ground		moderate	0.5	high	0.75
• Not exposed to rain but in contact with the ground		high	0.75	very high	1
• Exposed to rain, immersed or buried		very high	1	"	1

Coefficient TEM - Temperature conditions in service

Annual avg. temp. (°C)(°F)	TEM	Annual avg. temp. (°C)	TEM	Annual avg. temp. (°C)	TEM
< 0 (32)	0.4	10(50) to 20(68)	0.7	> 30(86)	1.0
0(32) to 10(50)	0.55	20(68) to 30(86)	0.85	-	-

Coefficient STR - Reinforcement and other restraints in service (along the direction(s) of rebars or restraints)

percent of steel	STR	percent of steel	STR	Restraint (MPa)	STR	Restraint (MPa)	STR
0	1,0	1	0.3	0	1,0	1	0.4
0.25	0.75	2	0.25	0.25	0.85	1.5	0.3
0.5	0.55	≥ 3	0.2	0.5	0.7	2	0.2
0.75	0.4	-	-	0.75	0.55	≥ 3	0.1

Coefficient CRE - Current rate of AAR expansion in concrete members in service

CRE	Qualification	CRE	Qualification	CRE	Qualification
0	negligible	1 to 2	low	>6 to 12	high
> 0 to 1	very low	2 to 6	moderate	>12 to 16	very high

I.4 References

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