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REPORT

PARTNER

Report 2.1

State-of-the art report: Key parameters influencing the alkali aggregate reaction

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L.T. Pedersen and B. Grelk, Rambøll, Denmark
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SINTEF Building and Infrastructure
Concrete

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SINTEF REPORT

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TITLE

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State-of-the art report: Key parameters influencing the alkali aggregate reaction

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ABSTRACT

The purpose of this report is not to write another educational book regarding (Alkali Aggregate Reactions) AAR. The task is, however, to give an update description of the mechanisms of AAR that can influence the results from the different test methods used in the PARTNER project. Evaluation of any structural effects (i.e. effects depending on structure type, dimensions, reinforcement, etc.) is not included in the report.

The ultimate challenge when testing for AAR in a laboratory is to provide quick, reliable results regarding the reactivity of certain types of aggregate, or even more important assessment of specific concrete job mixes (i.e. performance testing). The results are required to mirror the durability behaviour in real structures designed for life time for up to 100 years.

Many parameters will influence the alkali aggregate reactivity. Some of the parameters will only influence the reactivity in the laboratory, while others will have an overall contribution, both in the laboratory and in real structures. The following key parameters are discussed in the report in relation to AAR: temperature; humidity, moisture and degree of saturation; content of alkalis; role of calcium hydroxide (CH); types and content of reactive rock types; aggregate particle size / grading; size of test prisms; air entrainment, paste porosity and water/cement ratio; and finally storage conditions – leaching.

The authors have not made any effort of ranking the influence of the different parameters, because the influence of any changes in a parameter may vary a lot dependent of the situation, both when performing a laboratory test and in a real concrete structure. However, the experiences have shown that in particular variations in the humidity and/or the alkali content (due to leaching) in the test specimens lead to incorrect results. It is also very important to bear in mind the influence of the different parameters when performing a performance test that should reflect how a given concrete mix will behave in a real concrete structure for a long service life.

KEYWORDS	ENGLISH	NORWEGIAN
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GROUP 2	Concrete	Betong
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	Aggregates	Tilslag
	Test methods	Prøvingsmetoder

PARTNER

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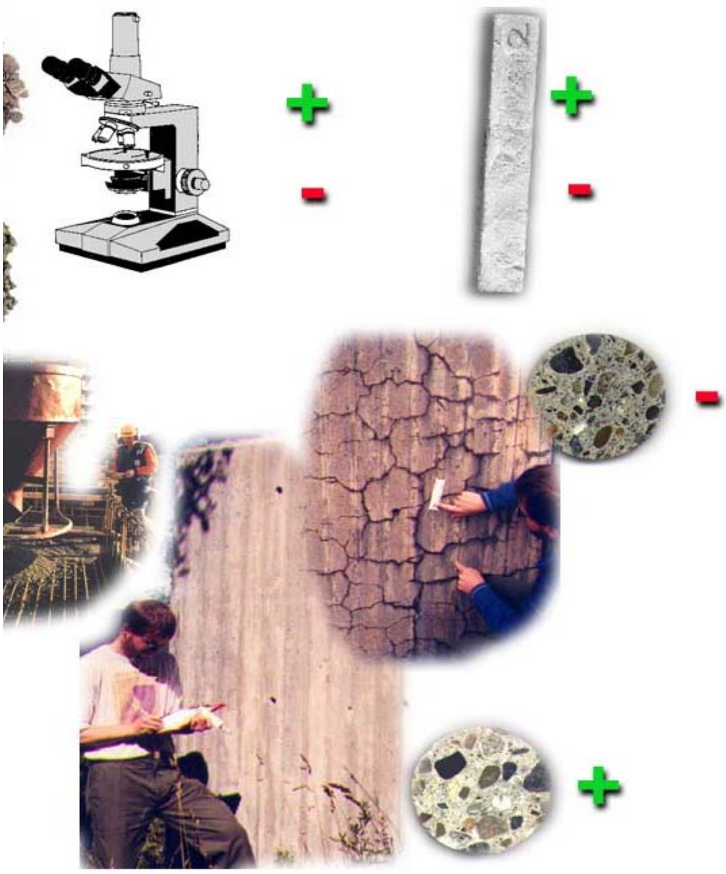

Report 2.1

State-of-the art report: Key parameters influencing the alkali aggregate reaction

Preface

This report is one of a series produced as output from PARTNER, a project funded by the European Community under the "Competitive and Sustainable Growth" programme.

The overall objective of this project is to provide the basis for a unified test procedure for evaluating the alkali reactivity of aggregates across the different European economic and geological regions. It will enable CEN TC-154, Aggregates, to fulfil the requirements of the Aggregates Mandate, M125, which identifies durability against alkalis as a necessary performance characteristic in the specification of aggregates for concrete (EN 12620) to meet the Essential Requirements of the CPD for Strength and Safety. The project will achieve this by:

- Evaluating the tests developed by RILEM, and some regional tests, for their suitability for use with the wide variety of aggregate and geological types found across Europe.
- Calibrating the results of these accelerated tests against behaviour in concrete in real structures and in field sites.
- Producing an "atlas" of the geology and petrography of European aggregates.
- Educating European petrographers and testing organisations in the effective use of these methods.
- Making recommendations, based on the above work, to CEN for suitable CEN methods of test and specifications to ensure durability against alkalis.

The project has 24 Partners from 14 countries, covering most of Europe, from Iceland to Greece.

Partners

BRE	UK
PC Laboratoriet A/S	DK
SINTEF	NO
SP - Swedish National Testing and Research Institute	SE
RAMBOLL	DK
ISSeP – Institut Scientifique de Service Public	B
LCPC Laboratory	F
VDZ – German Cement Works Association	D
TITAN Cement Company, S.A	EL
LABEIN	ES
CRIC – Centre National de Recherches Scientifiques et techniques pour L'Industrie Cimentière	B
IMBiGS	PL
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VOZ – Austrian Cement Association	A
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STATS	UK
NCC AB	SE
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List of available reports in the PARTNER project

WP 2: Aggregates and structures (leader: RAMBÖLL)

Report no.	Report title	Author
2.1	State-of-the art report: Key parameters influencing the alkali aggregate reaction SBF52 A06018 / ISBN 82-14-04078-7 / 978-82-14-04078-7	Hönnun, RAMBÖLL, SINTEF

WP 3: Test methods (leader: SINTEF)

Report no.	Report title	Author
3.1	Experience from using petrographic analysis according to the RILEM AAR-1 method to assess alkali reactions in European aggregates SBF52 A06019 / ISBN 82-14-04079-5 / 978-82-14-04079-5	SINTEF
3.2	Experience from testing of the alkali reactivity of European aggregates according to the RILEM AAR-2 method SBF52 A06020 / ISBN 82-14-04080-9 / 978-82-14-04080-9	PC-lab
3.3	Experience from testing of the alkali reactivity of European aggregates according to several concrete prism test methods SBF52 A06021 / ISBN 82-14-04081-7 / 978- 82-14-04081-7	BRE
3.4	Experience from testing of the alkali reactivity of European aggregates according to two Danish laboratory test methods SBF52 A06022 / ISBN 82-14-04082-5 / 978- 82-14-04082-5	RAMBÖLL
3.5	Field site tests established in the PARTNER project for evaluating the correlation between laboratory tests and field performance SBF52 A06023 / ISBN 82-14-04083-3 / 978- 82-14-04083-3	VDZ

WP 4: Precision trials (leader: SP)

Report no.	Report title	Author
4.1	PRECISION TRIAL – Determination of repeatability and reproducibility of the amended RILEM methods SBF52 A06024 / ISBN 82-14-04084-1 / 978-82-14-04084-1	SP

WP 5: Dissemination (leader: PC-lab)

Report no.	Report title	Author
	Final results and recommendations of the PARTNER project. Paper to be published at ICAAR 2008, Trondheim, Norway	BRE + several co-authors
	Database / atlas of the alkali reactivity of European aggregates Published by Geological Survey of Belgium *see www.aarig.org/webatlas/atlas.htm*	PC-lab, ISSEP, Hönnun

Summary

As part of the PARTNER project, a State-of-the-art study regarding Alkali Aggregate Reactions (AAR) has been carried out. This topic has been studied for at least 60 years, and many proceedings, books and articles have been written.. Therefore, the purpose of this report is not to write another educational book regarding AAR. The task is, however, to give an updated description of the mechanisms of AAR that can influence the results from the different test methods used in the PARTNER project. Thus, this report mainly focuses on the different key parameters influencing AAR. Evaluation of any structural effects (i.e. effects depending on structure type, dimensions, reinforcement, etc.) is not included in the report.

The ultimate challenge when testing for AAR in a laboratory is to provide quick, reliable results regarding the reactivity of certain types of aggregate, or even more important assessment of specific concrete job mixes (i.e. performance testing). The results are required to mirror the durability behaviour in real structures designed for life time for up to 100 years.

As discussed in the report, many parameters will influence the alkali aggregate reactivity. Some of the parameters will only influence the reactivity in the laboratory, while others will have an overall contribution, both in the laboratory and in real structures. The following key parameters are discussed in the report in relation to AAR:

- Temperature
- Humidity, moisture and degree of saturation
- Content of alkalis
- Role of calcium hydroxide (CH)
- Types and content of reactive rock types
- Aggregate particle size / grading
- Size of test prisms
- Air entrainment, paste porosity and water/cement ratio
- Storage conditions - leaching

The authors have not made any attempt of ranking the influence of the different parameters, because the influence of any changes in a parameter may vary a lot dependent of the situation, both when performing a laboratory test and in a real concrete structure. However, the experience has shown that in particular any variations in the humidity and/or the alkali content (due to leaching) in the test specimens lead to incorrect results. It is also very important to bear in mind the influence of the different parameters when performing a performance test that should reflect how a given concrete mix will behave in a real concrete structure for a long service life.

Annex A includes a description of the existing national standards and demands for the European countries.

The work was initiated in the beginning of the PARTNER project. However, results from recent research within this project are not included in this report.

Content

Annex

Annex A: National standards and demands

1	Australia
2	Belgium
3	Denmark
4	France
5	Germany
6	Greece
7	Iceland
8	Ireland
9	Italy
10	Netherlands
11	Norway
12	Poland
13	Portugal
14	Spain
15	Sweden
16	United Kingdom

1 Introduction

As part of the PARTNER project, a State-of-the-art study regarding Alkali Aggregate Reactions (AAR) has been carried out. AAR have been studied for at least 60 years, and many proceedings, books and articles have been written regarding this subject. Therefore, the purpose of this report is not to write another educational book regarding AAR, The task is, however, to give an update description of the mechanisms of AAR that can influence the results from the different test methods used in the PARTNER project. Thus, this report mainly focuses on the different key parameters that influence the alkali aggregate reaction. Evaluation of any structural effects (i.e. effects depending on structure type, dimensions, reinforcement, etc.) is not included in the report.

The report is compiled by Børge Johannes Wigum – VGK-Hönnun Consulting Engineers, Iceland (wigum@vgkhonnun.is), Lene Tørnæs Pedersen (LTP@ramboll.dk) & Bent Grelk (bng@ramboll.dk) – Rambøll, Denmark and Jan Lindgård (Jan.Lindgard@sintef.no) – SINTEF, Norway. Various persons have in addition contributed to the review if this report.

During the literature search, a selection of references has been reviewed to exemplify the various discussions and opinions. However, it has not been possible to cover all important references, and we apology for any major work not covered in this report. Parts of the content of chapters 3.2.2 and 3.6.4 come from the PhD work presented by Bård Pedersen – Norbetong, Norway (2001 & 2004) and he is acknowledged for his input.

First, a short introduction to the mechanisms of AAR is presented in **chapter 2**. Further examples of damages caused by AAR are presented, in addition to some methods to identify the cause and extent of damage.

The main focus in this report is on **chapter 3**, where the key parameters in relation to AAR are described. For each parameter, an in-depth review is made and its influence on the reaction or on the result of the reaction (the expansion) is discussed. The main focus is made on laboratory experience, however some links to effects in real concrete structures are also included. Much of the work is from recent literature, including major research and PhD. works dealing with the same topic.

One of the main goals of the PARTNER project is to evaluate the different test methods used in Europe today with the purpose of making one or more standardized test methods to be used all over Europe in the future. **Chapter 4** contains a summation of the different test methods used in the European countries today.

Annex A includes a description of the existing national standards and demands for the European countries.

The work was initiated in the beginning of the PARTNER project. However, results from recent research in this project are not included in this report.

2 The Alkali-Aggregate Reactions (AAR)

2.1 Mechanisms of AAR

Alkali Aggregate Reactions (AAR) are chemical reactions where sodium and potassium ions in solution react with certain rock types in the concrete aggregates. It is generally agreed that the chemical reaction between the alkali hydroxides and reactive silica is essentially a dissolution reaction. This occurs as a result of the increased solubility of amorphous, disordered or poorly-crystallised forms of silica minerals in high pH solutions. The reaction forms a hygroscopic alkali-silica gel which imbibes water and swells. The swelling forces generated may be sufficient to disrupt aggregate particles and the surrounding concrete, causing expansion, cracking, and associated deterioration. The mechanisms of reaction have been described by various authors (e.g. Dent Glasser and Kataoka (1981a,b), Chatterji et al. (1986, 1987, 1989), and an extensive literature review is given by Jensen (1993). The mechanisms of reaction will not be discussed further in this chapter.

The AAR can be divided into several reactions, which are associated with the type of alkali-reactive aggregates and the mechanism and rate of the reaction. *Alkali-silica reaction (AAR)*, which was identified first, is the most rapid reaction occurring with various heterogeneously structured, porous, and sometimes hydrous silica minerals. *Slow/late-expanding alkali-silicate/silica reaction*, appears to occur with a delayed onset and at a slower rate and is associated with crystalline quartz-bearing rock types as the reactive constituent. The so-called *alkali-carbonate reaction* which has been reported with some dolomitic limestones will not be considered in this report.

Basically three conditions have to be fulfilled before deleterious AAR can occur. In addition the effect of $Ca(OH)_2$ has been considered regarding the reaction. This is illustrated in Figure 2.1-1 below.

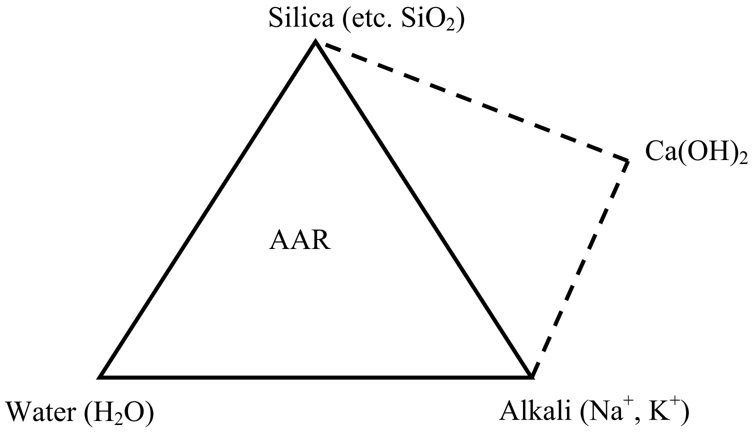

Figure 2.1-1: Illustration of the necessary components for AAR to occur.

The silica is contained in minerals in reactive rock types, for instance in porous flint as found in Denmark, rhyolite as found in Norway, limestone as found in Belgium etc. The water is contained in the concrete or it may come from external sources. The same is the case with the alkalis as these are contained in the cement paste or in special cases in the aggregate or it may come from outside sources as for instant de-icing salt. $Ca(OH)_2$ is present in the cement paste.

2.2 Damage caused by AAR

Basically the results of AAR can be divided into to categories in a structural point of view:

- Deleterious AAR.
- Harmless AAR.

The deleterious AAR is the case of AAR where cracking or unacceptable expansion of the concrete occur. In field structures in many cases the cracking does not lead to severe deterioration of the structure by itself, but it opens up for dangerous side effects such as chloride intrusion. If chloride penetrates all the way to the reinforcement, severe damage can be initiated, as the chloride ions can attack the reinforcement and a concentrated corrosion in the shape of pitting will occur. The cracks also make it easier for water to penetrate the concrete and the risk of damage due to freeze-thaw is highly increased. Another side effect is carbonation along cracks connected to the surface. If the cracks continue to the reinforcement local corrosion can occur in an area around the cracks. This may lead to spalling of the concrete cover, which in some cases can be as well a visual problem as a safety problem for people passing underneath the structure. In structures with severe cracking caused by AAR, spalling of the concrete cover can occur as well, leading to the problems described above.

In other cases deterioration caused by AAR can lead to critical situations for the structure especially in cases of post tensioned structures where the bearing capacity is usually optimized, and repairs will be very expensive to carry out. In such cases it is of great importance to have knowledge about the risk of further development of the AAR to be able to recommend the technical correct and economical optimal strategy for possible repair actions of the structure.

The harmless AAR is the reactions where only the reactive aggregate particles are cracked or dissolved. This could be the case if the content of reactive rock types in the aggregate is very low, or the concrete contains no free $Ca(OH)_2$.

Basically two theories have been proposed regarding the mechanism causing the expansion of the concrete:

a) The gel, which is a reaction product of the AAR, causes the expansion of the concrete.

b) The primary expansion is caused by a solid expansion of the aggregate. Secondary the gel can cause extra expansion when filling up closed cracks.

The fact that empty cracks, caused by AAR, have been observed in several concrete structures may be used as an argument for theory b). However, it is also known that the viscosity of the AAR-gel varies depending on the composition of the gel, resulting in different pressure on the cement paste (theory a). If the tensile exceeds the tensile strength of the cement paste, cracking occurs.

What is more likely (theory b) is that the expansion of the concrete is due to osmotic or hydrostatic pressure inside the reactive aggregate particles when they react. When only one aggregate particle expands, no cracking will be observed on the concrete surface. An expansion of several adjacent aggregate particles close to each other, will lead to tensile forces in the cement pasta between the particles. If the tensile capacity is exceeded, cracking will occur. This is illustrated in Figure 2.2-1 and 2.2-2. Thus, cracking caused by AAR is usually observed as map cracking with the reactive aggregates as the nodes.

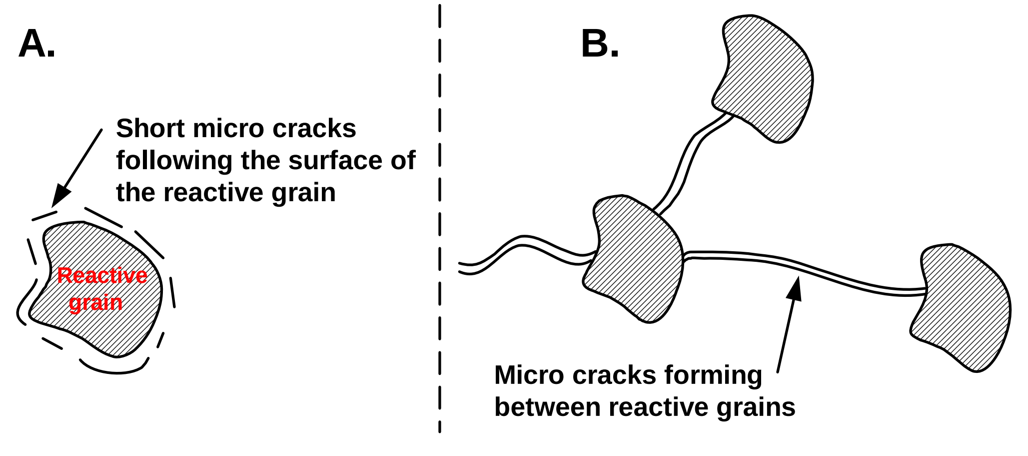

Figure 2.2-1: Illustration of micro cracking of the paste due to AAR.

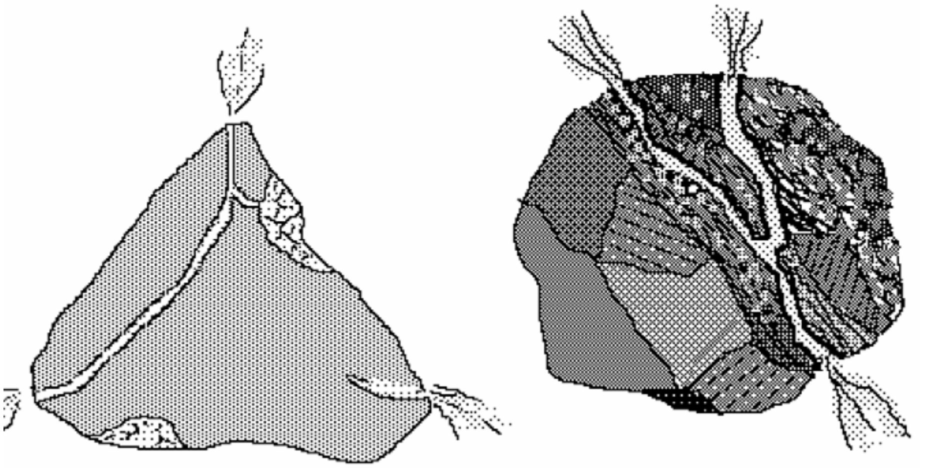

Figure 2.2-2 Sketches of difference in expansion and cracking between rapid- and slow/late reactive aggregates. Left (a); rapid reactive aggregate (e.g. opal). Right (b); slow/late reactive aggregate (e.g. mylonite). From Wigum (1995).

Aside from the environmental conditions (such as temperature and humidity) and the alkali content etc., the development of AAR depends on the type of reactive rock types in the aggregate and the aggregate grading. Some rock types are rapidly reactive such as porous flint, chert and opal, and some rock types containing microcrystalline quartz are slowly reactive as for instance the Norwegian rhyolite. The rapid reactions can develop within a few years, while the slow reactions may develop for as much as 20 years before visual signs occur (Pedersen 2001, VCJ, UK, ROA, 1995).

The direct damage caused by AAR is cracking, pop outs and expansion of whole concrete bodies. But cracks caused by AAR opens up the concrete for more severe deterioration mechanisms. The cracking caused by AAR is usually identified as map cracking. In reinforced concrete and pre-stressed concrete there is a tendency for the AAR cracks to follow the reinforcement or the stress direction (Hobbs 1988, Poole 1992). Another indication of damage caused by AAR is a variation of the extent of damage according to the climatic exposure of the structure. This is illustrated in Figure 2.2-3 (see the photo to the right). In this situation the light area of the bridge deck soffit toward the edge has been penetrated by water from to top of the deck. It is also evident in Figure 2.2-6, (see the photo to the left) where drainage water set off the development of cracking.

Figure 2.2-3 To the left: Cracks in a post-tensioned bridge deck (box girder) caused by AAR. The cracks are observed in the anchorage zone of the cables. The cracks follow the cables. To the right: Cracking of the outer zone of a road bearing bridge deck. The membrane of the bridge deck is damaged and water containing de-icing salt penetrates the bridge deck. The cracking is remarkably most severe in the area where the reactions have been accelerated by alkalis from de-icing salt. In both cases whitish pop outs are seen along the cracks. Both cases are from Danish structures containing porous flint.

Figure 2.2-4: Delamination and cracking of a bridge deck where the primary cause of damage is AAR (same structure as the one to the right in Figure 2.2-3).

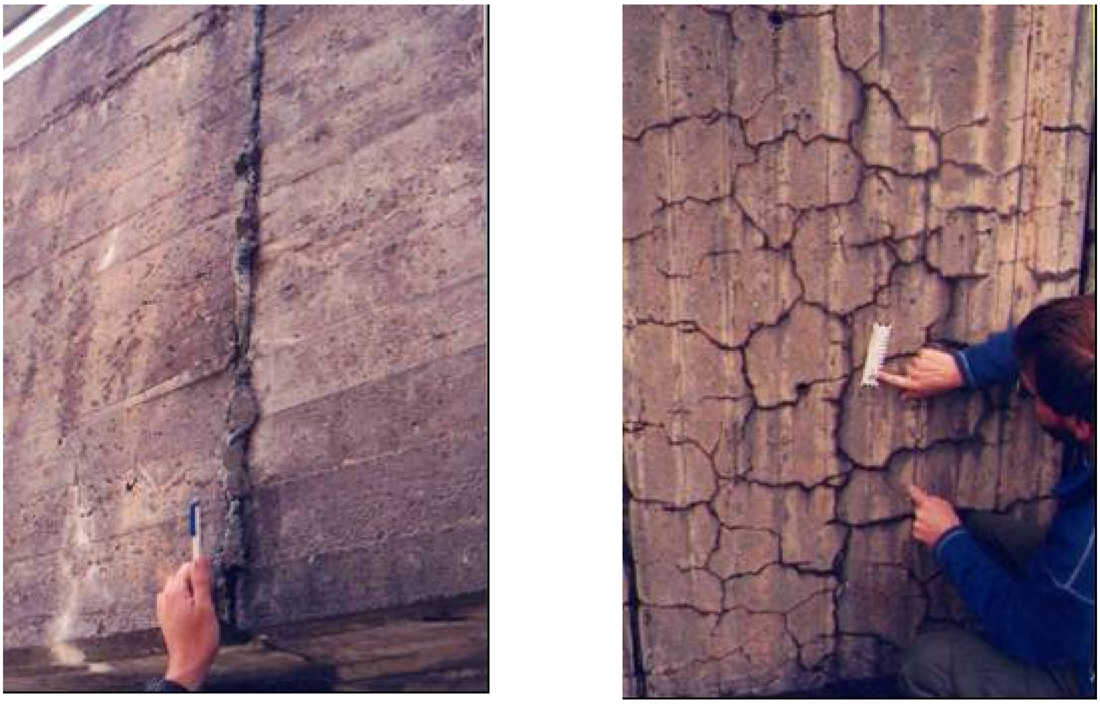

Figure 2.2-5: To the left: Expansion of whole concrete bodies in a Norwegian structure has reduce the expansion joint and the material in the joint is squeezed out. To the right: Heavy map-cracking in a Norwegian concrete wall. High water pressure behind the wall provides high levels of humidity in the concrete.

Figure 2.2-6: To the left: Concrete bridge in Norway. Draining water from the bridge deck flows down the concrete wall and set off the development of map-cracking in that area. To the right: Norwegian concrete foundation for high-voltage pylon exhibiting severe map-cracking due to AAR in combination with freeze/thaw damage.

Figure 2.2-7: Norwegian concrete bridge. Due to expansion of the whole bridge plate, a major crack has occurred in the bridge abutment.

2.3 Identifying damage caused by AAR

As mentioned AAR can be suspected as the cause of damage if map cracking or cracking orientated along the lines of reinforcement or pre-existing stress is observed on site. The crack pattern of a map is caused by the fact that the cracks occur between the reactive aggregates, which are distributed throughout the concrete, cf. Figure 2.2-1.

Another indicator of AAR is gel leaking out of the cracks. This is usually seen as transparent / brown wet parts of the concrete on both sides of the crack (cf. Figure 2.2.6 – to the right) – opposite to water, which is only seen at the lower side of the crack because of the influence of gravity. When the gel is dried out, it becomes white.

To be sure that AAR is actually the primary cause of damage, drilled concrete cores must be examined in a laboratory. For this purpose representative concrete cores must be drilled from relevant parts of the structure. If the cores are immediately wrapped in cling film and sealed in polythene bags, the cores and the AAR-gel are prevented from drying out. The cores are inspected visually at the laboratory and they are examined for traces of AAR – e.g. cracks and AAR-gel in cracks and in air voids. Furthermore the aggregates of the concrete are examined. For some European aggregates it is possible to visually evaluate if the aggregate is potential reactive, but for most aggregates petrographic examination in thin sections are required.

Fluorescence impregnated plane and thin sections are very good tools in determining the cause of damage. Under the preparation the samples are vacuum impregnated with epoxy containing a fluorescent dye. By using such impregnation even very fine cracks will be visible while exposing the plane section to ultra violet light. The crack pattern and the extent of damage can then be determined. An experienced petrographer can also identify if the concrete contains potential reactive rock types (in most cases thin sections are then required).

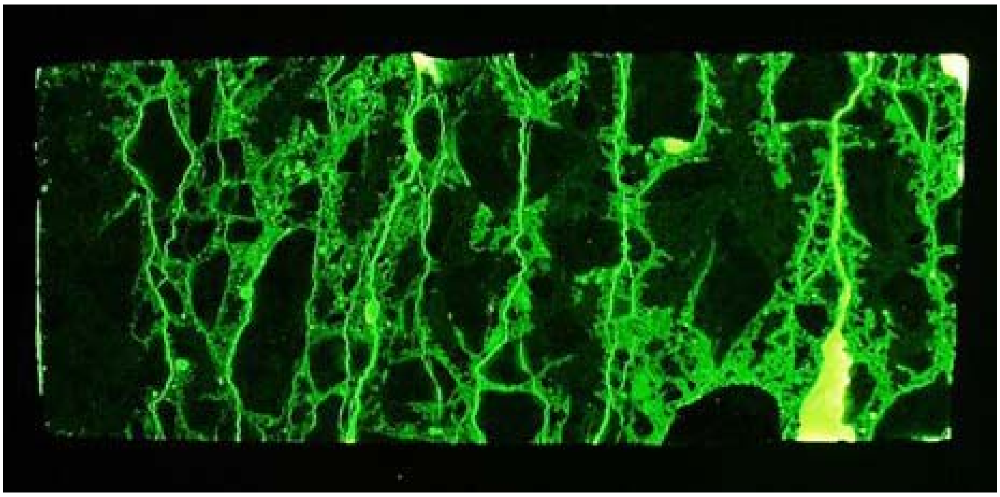

Figure 2.3-1: Fluorescence impregnated plane section from a Danish bridge deck containing porous flint in the sand fraction. The damage cause is determined as a combination of AAR and freeze/thaw. The length of the core is approximately 160 mm.

Various researchers have developed methods to quantify the degree of cracking and reaction in drilled concrete cores (Grattan-Bellew and Danay (1992), Broekmans (2001), Lindgård et al. (2004), Rivard and Ballivy (2005)).

During the examination of thin sections, the origin of the cracks and the presence of AAR-gel in voids and cracks can be identified. Cracks perpendicular to the surface of the reactive grains are the most important diagnostic feature – see Figure 2.3-2.

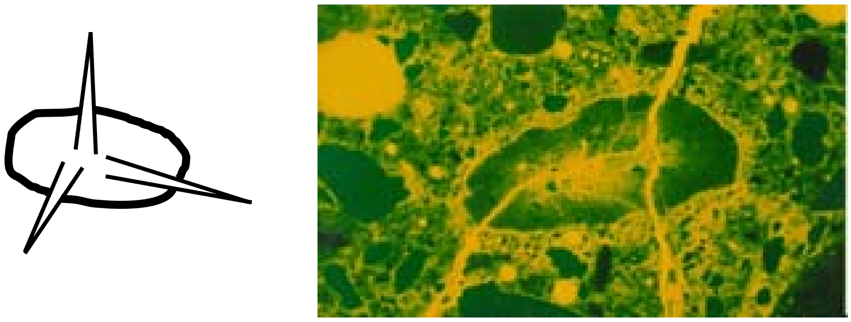

Figure 2.3-2: To the left: Cracks perpendicular to the surface of the reactive grains is the most important diagnostic feature. To the right: Fluorescence impregnated thin section showing cracking of the cement paste due to AAR in a sand grain of porous Danish flint. The size of the photo to the right is approximately 3.5 × 2.5 cm.

After determination of AAR as the main cause of damage, additional testing might be of interest to predict the future development of the AAR. By monitoring a structure, both the development of the expansion and the crack widths may be measured. In the laboratory, accelerated tests with concrete prisms exposed in water or salt solutions at high temperatures might be used (Strunge & Jensen, 1991). The test results can be used for predictions of the future development of the reactions (Thaulow & Geiker, 1992). However, the correctness of such expansion results has been disputed.

Techniques have also been developed to test and examine aggregates separated from sampled concrete (the rock composition may then be determined by use of standard petrographic methods), along with procedures to determine level of the alkali content in the concrete (Lindgård and Wigum, 2003).

An extended guide to diagnosis and appraisal of AAR damage to concrete structures has been developed by the RILEM committee TC 191-ARP (denoted “AAR-6.1”), Rilem (2006).

Bérubé et al. (2004) examined and discussed the value of expansion tests on cores from AAR-affected structures. It was concluded that the expansion results of tests performed in humid air were largely influenced by core diameter and pre-existing cracking. Results from tests in alkaline solution were largely influenced by pre-existing cracking, core diameter, alkali content, and concrete permeability (water/cement ratio).

3 Key parameters controlling AAR

There are several key parameters that influence the alkali aggregate reactivity in a concrete. As mentioned in chapter 2, basically four conditions have to be fulfilled before harmful AAR can occur. Beside this, other circumstances can influence the reactivity as well. This chapter deals with the following key parameters in relation to AAR:

- Temperature
- Humidity, moisture and degree of saturation
- Content of alkalis
- Role of calcium hydroxide (CH)
- Types and content of reactive rock types
- Aggregate particle size / grading
- Size of test prisms
- Air entrainment, paste porosity and water/cement ratio
- Storage conditions - leaching

In the following, each of the parameters mentioned above is reviewed separately, both based upon experiences from laboratory testing and experiences from real structures in the field. Evaluation of structural effects (i.e. effects depending on structure type, dimensions, reinforcement, etc.) are not included in the report.

3.1 Temperature

3.1.1 Laboratory experiences – reaction vs. expansion

One of the basic rules of chemistry is that increasing temperature usually accelerates a chemical process. As the alkali aggregate reactions are chemical reactions, it is expected that increased temperatures will increase the reaction rate. This has also shown to be true by many test results. However, dealing with alkali aggregate reactions in concrete the main concern about the reactions is the expansion caused by AAR. Thus, the expansion is very important when evaluating the effect of temperature of AAR. Experience shows that several reactions in a concrete are necessary for expansion to occur. If there is only a small amount of reactive material, the reaction will not be able to create stresses larger than the tensile strength of the concrete. This is illustrated in Figure 2.2-1.

Tests carried out with a fixed amount of alkalis have shown that a pessimum level of temperature can be found at which the expansion of the concrete is most severe. This pessimum value is found to be around 38^{o}-$40^{o}C$ (Locher, 1973). Tests carried out with an unlimited amount of alkali, for some particular aggregates, show that the total expansion decreases with increasing temperature (Chatterji and Christensen, 1990). Figure 3.1-1 and Figure 3.1-2 illustrates the two situations. The illustration to the right in Figure 3.1-1 is valid for ASTM type of tests, i.e. with limited alkali at a *fixed time*. However, the test results shown in Figure 3.1-2 and Figure 3.1-4 indicates, that the temperature corresponding to the maximum expansion might be lower if the tests are carried out for a longer time.

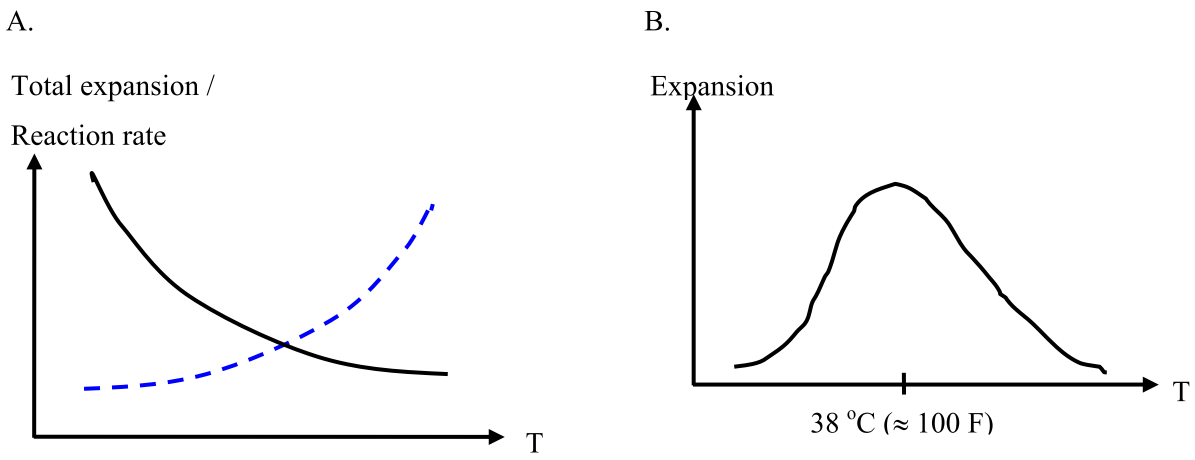

Figure 3.1-1: A: Illustration of the total expansion (solid line) and the reaction rate (dotted line) versus temperature. The total expansion decreases by increasing temperature. The reaction rate increases by increasing temperature – cf. Figure 3.1-2. The illustration is valid for unlimited alkalis. B: Expansion versus temperature at a fixed time with fixed alkali content (Locher, 1973).

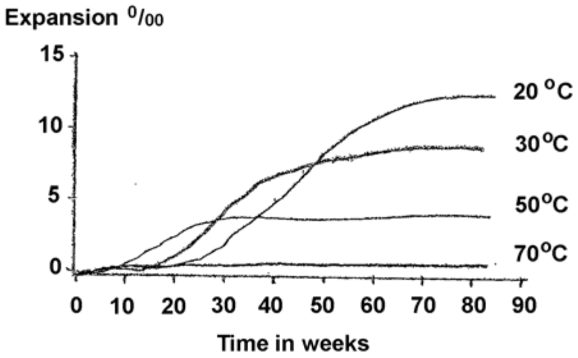

Figure 3.1-2: Test results showing the total expansion versus the time at different temperatures in test with an unlimited amount of alkalis. The total expansion decreases by increasing temperature. Mortar bar containing Danish Nymolle sand stored in a sealed container with 1 N NaCl. Extrapolated from results presented by Chatterji and Christensen (1990).

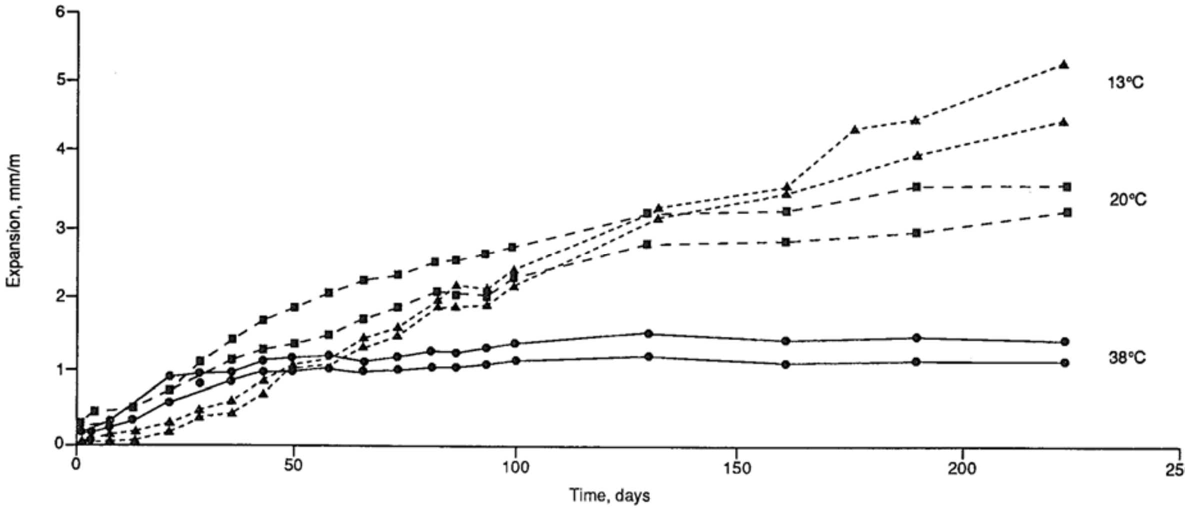

Figure 3.1-3: Core expansion at 100% RH and different temperatures. The tests are made with aggregate from southwest England, i.e. coarse limestone combined with sea dredged fines containing chert (Mott, Hay & Anderson, 1986).

The relation showed in Figure 3.1-2 and Figure 3.1-3 might be connected to the behaviour of calcium hydroxide as the solubility of $Ca(OH)_2$ is dependent of the temperature as well. The solubility of $Ca(OH)_2$ versus temperature is illustrated in Figure 3.4-2. The figure shows that the solubility of calcium hydroxide decreases by increasing temperature.

It has to be mentioned, that tests carried out by Òlafsson showed the opposite effect of the temperature regarding the maximum expansion (Ólafsson, 1988). The tests were carried out with mortar bars made of cement with an alkali content of 1.5 $NaO_{2,\ eqv}$ stored in sealed containers at a humidity level of 100% and temperatures at 38^{o}C respectively 23^{o}C. The test results showed higher expansion for the mortar bars stored at 38^{o}C after 12 months (corresponding to 365 days – cf. Figure 3.1-4) compared to the mortars bar stores at 23^{o}C (Ólafsson, 1988).

The temperature dependency shown in Figure 3.1-2 and Figure 3.1-3 illustrate the total expansion. There is, however, a difference between the expansion *rate* and the *extent* of expansion in relation to the temperature. As illustrated in Figure 3.1-4 (Poole, 1992) the reaction is increased by increasing temperature, while the ultimate expansion seems to decrease by increasing temperature.

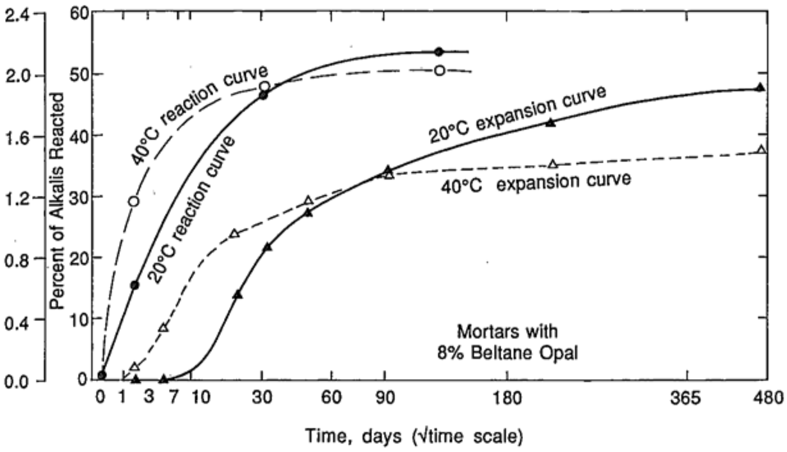

Figure 3.1-4: Comparison of reaction parameter curves versus expansion curves for sealed mortar specimens at 20^{o}C and 40^{o}C (Poole, 1992).

3.1.2 Experiences from real structures

Also for many field structures the influence of the temperature is obvious. The tendency is that the higher temperature, the higher rate of alkali aggregate reaction can be observed (Nixon et al., 1979). For instance in Denmark the effect of temperature dependence has been observed on bridges with alkali aggregate reactions in the beams bearing the bridge decks. Deleterious damage due to AAR has been observed in the beams on the south side of the bridges, while cracking has not yet been observed on the beams on the north side of the bridges. This is most likely due to the fact that the surface of the beams toward south is heated up from the sun during the day, while the beams on the north side are in the shade.

The same phenomenon has been observed on cross sections of some structure parts, for instance beams. Normally the outer parts of the cross sections are more heated up than the inner parts, leading to a more rapid AAR in the outer parts (provided the moisture content is sufficient high – see chapter 3.2).

For field structures it is, however, very important to keep in mind the results shown in Figure 3.1-2 and Figure 3.1-3 as these indicate that expansions could be expected on the cold side of the structures as well – and more important – by time these expansions may develop to be more severe than the expansions already observed on the warmest side of the structures.

3.2 Humidity, moisture and degree of saturation

3.2.1 Laboratory experiences

In most laboratory test methods a Relative Humidity (RH) of approximately 100% is prescribed – see Table 4.1. Thus the moisture content should be high enough to initiate a possible AAR – see Figure 3.2-2.

Regarding the influence of humidity, different propositions are made in the literature. Some authors claim that if the humidity exceeds a certain level, the viscosity of the gel will be so low that the gel penetrates the paste without causing any cracking (Nilsson, 1983). This assumption is based on test results where a decrease in pop outs was observed when the humidity exceeded approximately 90% - see Figure 3.2-1A. However, other test results show increasing expansion with time, even when the test specimens were submerged in water respectively in a concentrated NaCl-bath. The latter result is obtained from the Danish test method TI-B-51 – see Figure 3.2-1B.

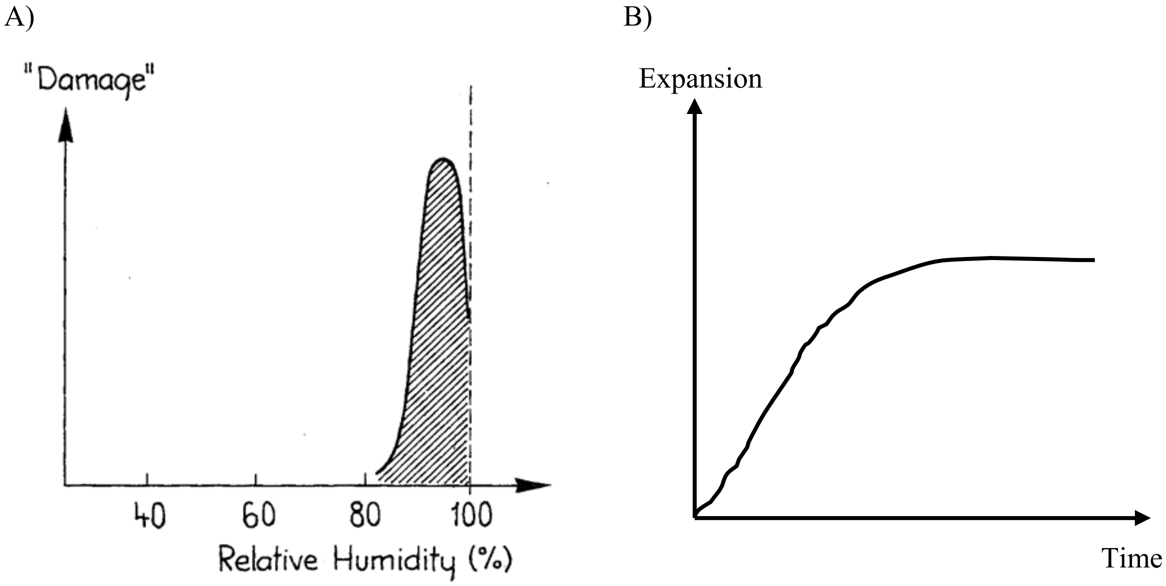

Figure 3.2-1: A: Test results showing the number of pop outs as a function of the humidity (Nilsson, 1983). B: Typical pattern of the expansion of a mortar bar in a saturated salt solution (NaCl) as a function of time by use of the Danish test method, TI-B51.

The diverging results from the two different test results presented in Figure 3.2-1 may be explained by differences in the test circumstances. In the Danish test method, TI-B51 the humidity of 100% is obtained by placing the mortar bar in a saturated salt solution (NaCl), enabling alkalis from the solution to penetrate into the concrete specimens. In the results presented in Figure 3.2-1A, expelled moisture could be observed on the surface of the test specimens when the RH was close to 100 % (Nilsson, 1983). This moisture will most likely contain some alkalis that have been withdrawn from the cement paste of the test specimens. As a consequence the alkali content of the concrete may have been reduced. The relation between damage and relative humidity is more likely to be as shown in Figure 3.2-2. This curve is also confirmed by the test results described in Chatterji et al. (1986) and Ólafsson (1988). Figure 3.2-2 illustrates that below a RH of 80% within the concrete deleterious damage due to AAR is unlikely to occur. But when the RH exceeds 80%, reactions due to AAR may appear provided that the other required components (i.e. a reactive aggregate and sufficient alkalis) are present.

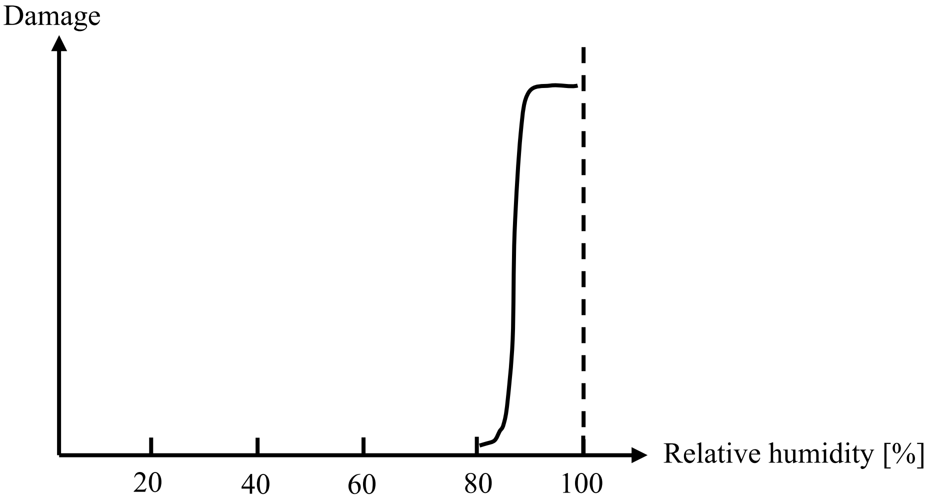

Figure 3.2-2: Illustration of the relation between damage due to AAR and relative humidity.

In an Icelandic study by Gudmundsson and Asgeirsson (1983) shown in Figure 3.2-3, a higher humidity gave a higher expansion (except for the combination 90% RH / 38°C that gave a somewhat higher expansion than 95% RH). Mortar bars stored at 23ºC and 100% RH did not expand much at this age (6 months) and temperature, and specimens stored at 83% and 90% RH even showed a small shrinkage when stored at a temperature of 23ºC. Results from another Icelandic investigation (Olafsson, 1986) showed that mortar bars stored at 95% and 100% RH, respecivety, revealed nearly equal expansions, but at 90% RH or lower the tendency was that a lowering of RH gave a decrease in expansion.

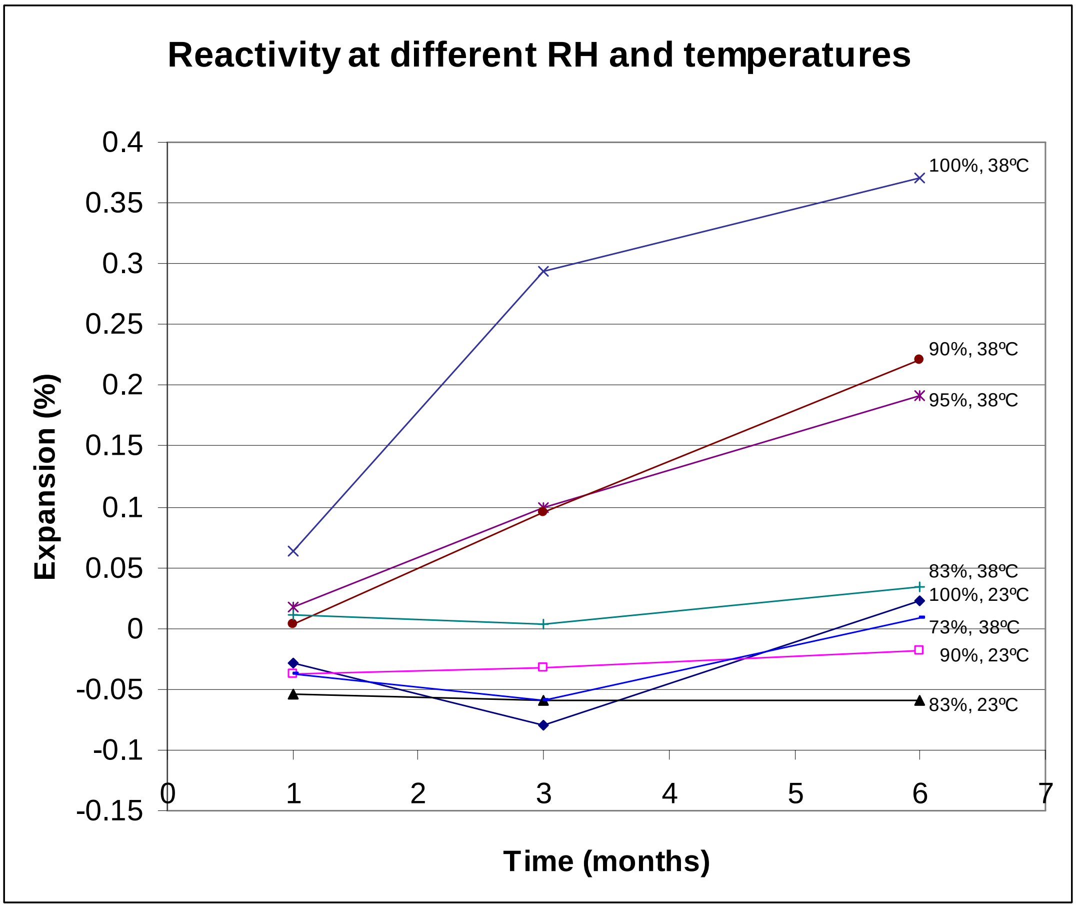

Figure 3.2-3. Mortar bar expansions at different RH and temperatures. Results from Gudmundsson and Asgeirsson (1983).

Larive et al. (2000) conducted laboratory experiments with a reactive siliceous limestone and a non-reactive limestone as a reference in order to investigate the role of water on concrete swelling. Four different curing conditions were used: immersed in water, storage at 100% RH, storage 95-99% RH, and sealed curing/storage by wrapping the prisms in aluminium foils (to avoid loss of water). All the specimens were stored at 38°C. The w/c-ratio was 0.48 for all the mixes. The experiments clearly showed a relation between expansion and weight increase for prisms stored in moist humidity or water. Prisms kept in 95-99% RH gave a maximum expansion higher than 0.20%, but even prisms wrapped in aluminium foils (sealed curing) gave an expansion higher than 0.1%. This shows that for this w/c-ratio the remaining water after cement hydration is sufficient to develop AAR and swelling. Another conclusion from this study is that access to water increases the concrete expansion during the formation of AAR-products. But after completion of the chemical reactions, any added water will not cause extra swelling in spite of the increase in weight due to water adsorption.

3.2.2 Experiences from real structures

Moisture is as stated important for the development of deleterious alkali aggregate reactions, and it is therefore of great interest to have knowledge of safe and unsafe levels of moisture state in concrete. In most literature dealing with moisture conditions in field related to AAR, relative humidity (RH) has been used as the only measured parameter. In a literature review carried out by Pedersen (2001) it is stated that it is better to measure moisture content in terms of degree of capillary saturation (DCS) or degree of saturation (DS). This is particularly the case for low strength concrete. For concrete of high strength RH might be considered as a more relevant parameter as it is more sensitive to the actual water content. Relling (1999) claims that the RH is a measure of the thermodynamic state of the pore water, and does not represent the amount of water. Thus the RH might be important for the redistribution of water within the concrete after alkali-silica gel is formed. At a specific moisture content the RH will be a function of the pore structure, the temperature and the chemical composition of the pore water. The relationship between RH and actual water content is given as adsorption/desorption isotherms. According to the hysteresis of the adsorption-desorption isotherms RH will not be related directly to the actual amount of pore water.

One of the most comprehensive investigations on RH in varying climates was carried out by Stark (1991). He showed that even in desert climates most of the concrete in highways and dams remain sufficiently damp to sustain expansive AAR (> 80% RH). And bridge decks and columns in dry climates are sufficiently damp on seasonal basis to sustain alkali reactions, and examples are given of alkali reactions on columns in arid climate. Massive concrete members in controlled indoor environment might maintain a RH of 80-85%, and might thus be able to react. In general he reported that the variations in moisture content seemed to lie in the outer 50 mm of the concrete, while the conditions in the inner parts were to a great extent a function of the w/c-ratio. A low w/c-ratio will give a lower resulting RH due to the self-desiccation caused by the cement hydration. Low w/c-ratios and consequently more cement with resulting higher alkali levels might thus under certain conditions be favourable to resist AAR because there will be a lack of water for the reaction to proceed.

Jensen (2000) performed tests on structures with documented AAR in Norway. Tests were performed with the wooden stick method, which when calibration data for each wooden stick is available, is reported to be a precise and stable method. Moisture data are reported from 3 cases, and there were in general monitored high RH in areas with visible damages (i.e. map cracking). Columns at a bridge gave very small variations over time, with 93-99% RH (average of 50 and 250 mm from surface) on the western faces, and 87-89% on the eastern faces (50 mm from the surface). Profiles of these columns gave values down to about 87% at a distance of 750 mm from the surface. Measurements from a dam structure gave variations between 97 and 100%, with no distinct variation between 50 and 400 mm from the surface.

Bligh (1991) performed tests on a highway structure damaged by AAR. In general he measured RH higher than 97%, and claimed that cracking caused by AAR resulted in higher RH due to the easy access to the inner parts of the member, while this water will not evaporate easily.

In a research project by Lindgård et al. (2004a,b, 2006), the degree of water saturation (capillary suction) was measured in the laboratory, after drilling out cores from the structure according to a special procedure (Lindgård and Wigum, 2003). The water content was then measured in the inner part of the cores in depths varying from about 250 to 400 mm from the surface. The experience has shown that the water content is rather stable in such depths. Later research has shown that the use of water in the drilling process only may lead to a minor increase in the water content in the concrete (maximum 1-2 volume-%), as long as the special drilling procedure (and packing of samples) is followed (Geicke and Dallager, 2003).

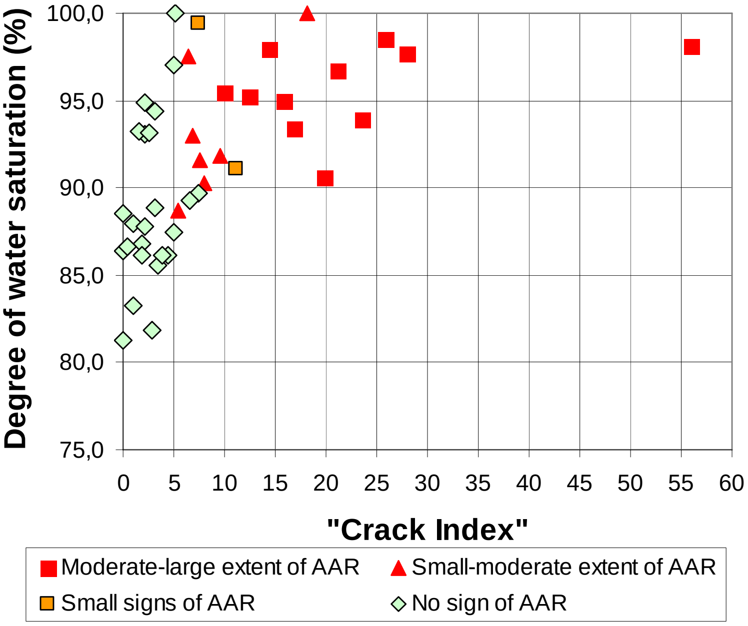

Figure 3.2-4. "Crack Index" versus degree of capillary saturation (DCS) in the drilled cores. From Lindgård et al. (2004 & 2006).

As shown in Figure 3.2-4 there is a rather good correlation between the presence of AAR, and thus a "high" Crack Index (CI), and the measured water content within the concrete. With one exception the degree of capillary saturation (DCS) of the concretes is higher than 90 volume-% for all the concretes with presence of AAR. Further one the water content of the majority of the concretes with no sign of AAR in the drilled cores (and thus a low CI) is lower than 90 volume-%. One reason for the observed high water content in the concretes with presence of AAR is that a high water content increases the risk for development of AAR. Another reason is assumed to be that the formation of an absorbing gel due to AAR leads to an increased water content.

As the alkali aggregate reactions are dependent of the presence of alkalis, calcium hydroxide, silica and water at the same time, the reaction is very much dependent of the moisture history. When the structure is exposed to water there will be a flow of water in the concrete by which the alkalis and the dissolved calcium hydroxide can be transported to the reactive aggregate from larger areas in the concrete. This will speed up the reaction rate as this transport of alkalis and calcium hydroxide by the flow of water will be much faster than the diffusion, which will otherwise control the reaction. Thus, if the concrete is subject to cycles of moisturizing and drying, alkalis and calcium hydroxide can be transported to the reactive aggregate and accumulate there resulting in an increase in the reaction rate. If, on the other hand, the moisture content is relatively steady, the reaction rate will be slower. For field structures this means that the extent of AAR is likely to be most severe close to the surface compared to the middle of the structure. Also a remarkable difference of the degree of damage due to AAR between the weather and the lee side of the structure has been identified for several structures (Poole, 1992 and Nixon et al., 1979). This is illustrated for a beam of an outdoor structure in Figure 3.2-5.

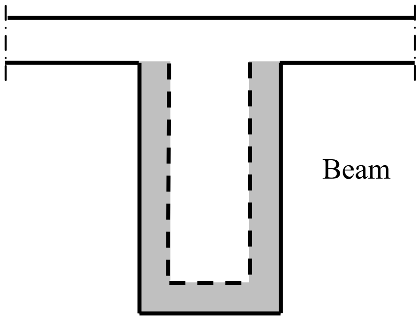

Figure 3.2-5: Cross section of a beam of an outdoor structure. The shaded area shows the part of the cross section where the moisture content is likely to vary most, and thus where the AAR reactions are expected to be most severe.

In field structures the side effects of high moisture content in the concrete is also of great importance, as possible leaching of alkalis and calcium hydroxide will decrease the reactivity.

3.3 Content of alkalis

3.3.1 Laboratory experiences

Any concrete contains a certain amount of alkalis. The majority (up to approximately 99%) of the alkalis contained in the concrete come from the cement. Minor amount comes from additions such as air entraining agent, fly ash, slag and micro silica, however dependent on the amount of these additons. Furthermore some specific aggregates may also contribute to the content of alkalis in the concrete. Regarding the addition of fly ash and micro silica the expansion may not necessarily increase because of the small increase in the alkali content, as these additives also reduces the pH value, which will decrease the reactivity.

Several tests have shown that alkalis from external sources may also contribute to the reactions (Chatterji et al. (1987), Okada et al. (1986), Kawamura et al. (1988), Sibbick & Page (1998), Sal'nikov and Ivanov (1968), Nixon et al. (1986)). Therefore the environment of the concrete structure ought to be evaluated before choosing the aggregate for the concrete. Potential risk of high alkali environments are swimming baths, structures near the ocean and structures near roads in countries where de-icing salt is used – for instance in the northern part of Europe. However, as it is seen from Appendix A, such an evaluation is now incorporated in the national standards regarding AAR in several countries.

Chatterji et al. (1987) and Sal'nikov and Ivanov (1968) have carried out several mortar bar tests evaluating the effect of adding alkalis to the reactive aggregate from external solutions e.g. nitrite, nitrate, chloride and hydroxide of sodium and potassium. The test results showed that the neutral alkali salts e.g. chloride, nitrite and nitrate, give higher expansion than corresponding hydroxides. This observation was explained by Chatterji et al. (1987) as due to the lowering of $Ca(OH)_2$ concentration in hydroxide solutions. Kawamura et al. (1988) came up with similar results in tests with mortar bars immersed in NaCl solution respectively NaOH solutions.

Some of the results from tests carried out by Chatterji et al. (1987) and (1989) with mortar prisms stored in 3N solutions of potassium (K) respectively sodium (Na) salt are illustrated in Figure 3.3-1. The results show that potassium (K) seems to accelerate the reactions more than sodium (Na). The research also showed that NaOH tends to result in dissolving the reactive grain rather than forming a gel, while the opposite is the case when the alkalis come from NaCl. Concrete stored in NaCl thus reveal a higher expansion compared to concrete stored in NaOH. In nature the main external sources of alkalis to concrete structures are de-icing salt and seawater with the constituent of NaCl. The expansions measured on tests specimens stored in NaOH may therefore be lesser than the expansions that can actually be achieved in nature under severe conditions.

A) B)

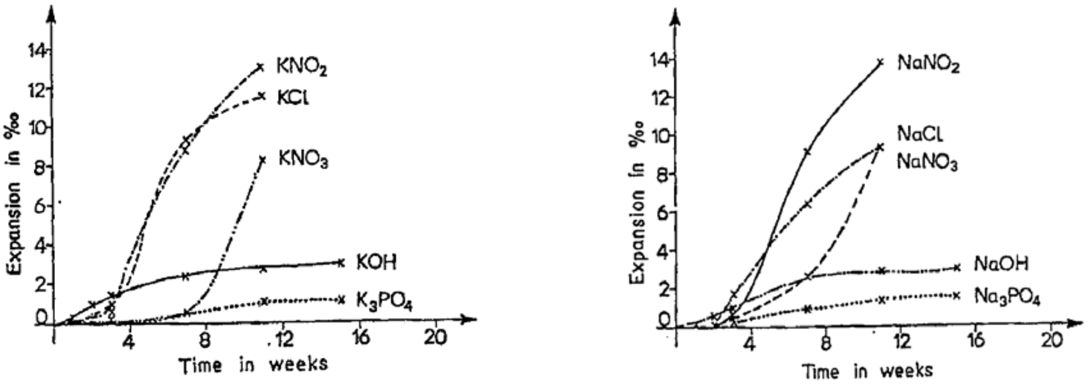

Figure 3.3-1: A: Test results showing expansion versus time from tests with mortar prisms stored in 3N solutions of potassium salts (Chatterji et. Al., 1987). Similar test results are found in Sal´nikov & Ivanov (1968).
B: Test results showing expansion versus time from tests with mortar prisms stored in 3N solutions of sodium salts (Chatterji et. al., 1987). Similar test results are found in Sal'nikov and Ivanov (1968).

Several other people have also tested the influence of salt on AAR, and it is concluded that alkali aggregate reactions is affected by the content of salt either in the aggregate or in the cement paste (penetration from outside sources), Okada et al. (1986) and Kawamura et al. (1988).

However, not all test results confirm the above-mentioned conclusion regarding the affect of sodium chlorides versus sodium hydroxides. Long term tests with mortar bars respectively concrete bars carried out by Duchesne and Bérubé (2003) show a much greater expansion of mortar bars immersed in 1 M NaOH than mortar bars immersed in 1 M NaCl. Another interesting observation made by them is that the initial alkali content (test with low-alkali (LA) respectively high-alkali (HA) cement) do not seem to affect the expansion of mortar bars immersed in NaOH, while the expansion of the mortars immersed in NaCl is affected by the initial alkali content. An affect of the initial alkali content was also observed for concrete prisms immersed in NaCl (greater expansion of prisms with HA cement), while no affect was observed for the concrete prisms immersed in NaOH.

3.3.1.1 Alkali release from aggregate minerals

The possibility that certain aggregate components release alkalis directly into the concrete pore solution has been considered for many years. Such alkali release from aggregates could enable AAR to occur even with low alkali cements. A considerable number of studies have reported results on this issue. Stark and Bhatty (1985) documented that some feldspar-containing American aggregates released significant amounts of alkalis into saturated CH solutions at 38 and 80°C. Similarly, Jensen (1993) found that a number of feldspar-bearing Norwegian aggregates released alkalis into saturated CH solutions at 40°C.

Based on results obtained from tests on aggregate particles, Bérubé (2002) found that very significant amounts of alkalis can be supplied with time by aggregates to the concrete pore solution, particularly by feldspar-rich ones. After 1.5 years, the amounts of alkalis released by the aggregates in a 0.7 M alkaline solution, which are close to the pore solution in normal concrete, were significantly higher than in water or a lime-saturated solution, even when using a solid excess of lime. Based on an aggregate content of 1850 kg/m^3, the alkali release varied from one aggregate to another, from < 0.1 to 12.7 kg/m^3 Na$_2$Oeq, for an average of 2.2 kg/m^3, being particularly high for a nephelinitic aggregate tested (phonolite).

Constantinera and Diamond (2003) performed direct measurement of alkali concentrations in pore solutions pressed from mortars with and without feldspars of several kinds. The results showed that: (1) feldspars (and presumably other alkali rich aggregates) embedded in a cement matrix do release alkali ions into the pore solution, and (2) that, if AAR-reactive aggregates are present, these released alkali ions can participate actively in AAR.

Bérubé and Fournier (2004) pointed out the fact that there is still no consensus about the absolute amounts that can be released by the aggregates in to the concrete. There is a need to develop a procedure to be used to estimate these amounts, and the way to take account for this alkali contribution in the standards or guidelines. RILEM has established that alkali release from aggregates can be a factor in some cases. A test method (AAR-8) is being developed. The method is only at an early stage, but it has been determined that the extraction medium will be an alkali hydroxide solution rather than the calcium hydoxide solution that has been most widely used previously (Sims & Nixon, 2006).

3.3.2 Critical amount of alkalis for European aggregates

The critical amount of alkali in the concrete differs according to the reactive material considered. This is illustrated Figure 3.3-2 considering two types of reactive sand: opaline silica respectively Thames Valley sand.

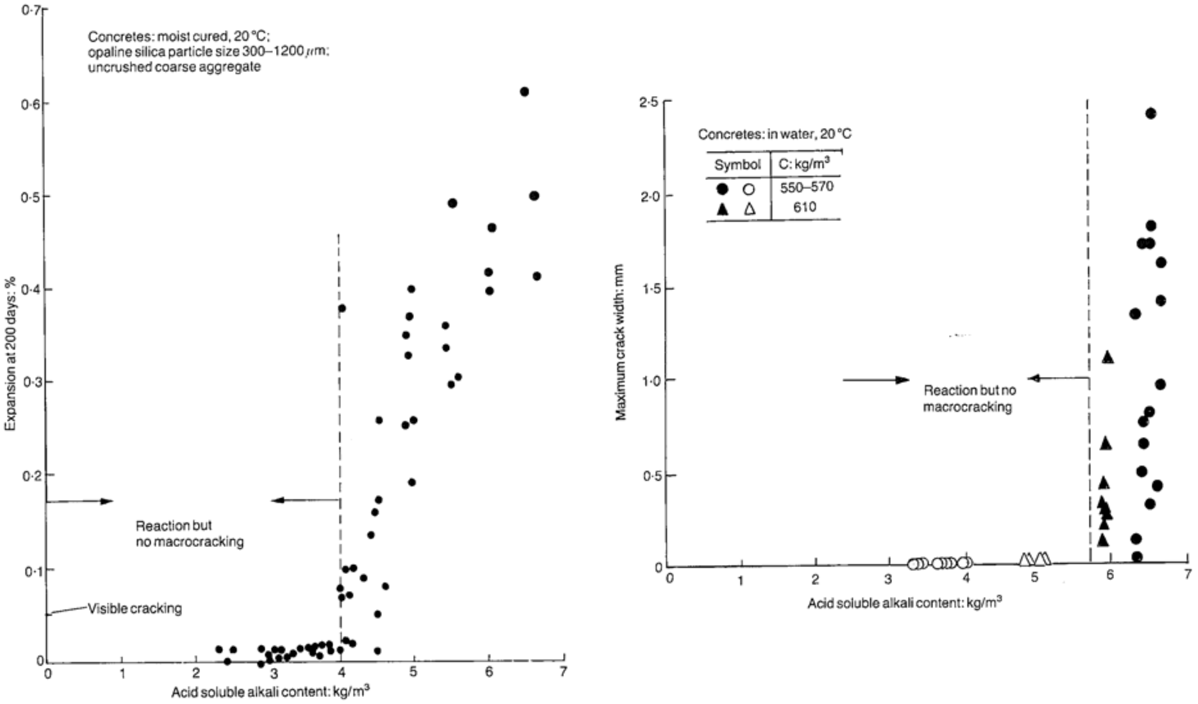

Figure 3.3-2: Test results from tests with concrete prisms stored at 20°C. To the left: Concrete made of reactive opaline silica and uncrushed coarse aggregate. Variation of expansion at 200 days with acid soluble alkali content. To the right: Concrete made of Thames Valley sand and Mendip limestone coarse. Variation of maximum crack width at an age of 14 years with acid soluble alkali content (expansion of cracked specimens was 0.15-0.30%) (Hobbs, 1986 &1988).

The Norwegian Concrete prism method was used to examine the threshold limit of alkali for three different types of Norwegian aggregates (Lindgård & Wigum, 2003). Both results from the "old" and the "new" version of the method are presented in Figure 3.3-3. The difference between the two versions is the size of the storage containers. The small containors are assumed to give a more stable high RH in the air surrounding the prisms, and thus a higher expansion. As shown in the figure a significant increase in the expansion is observed when the alkali level exceeds 3.0 kg/m^3.

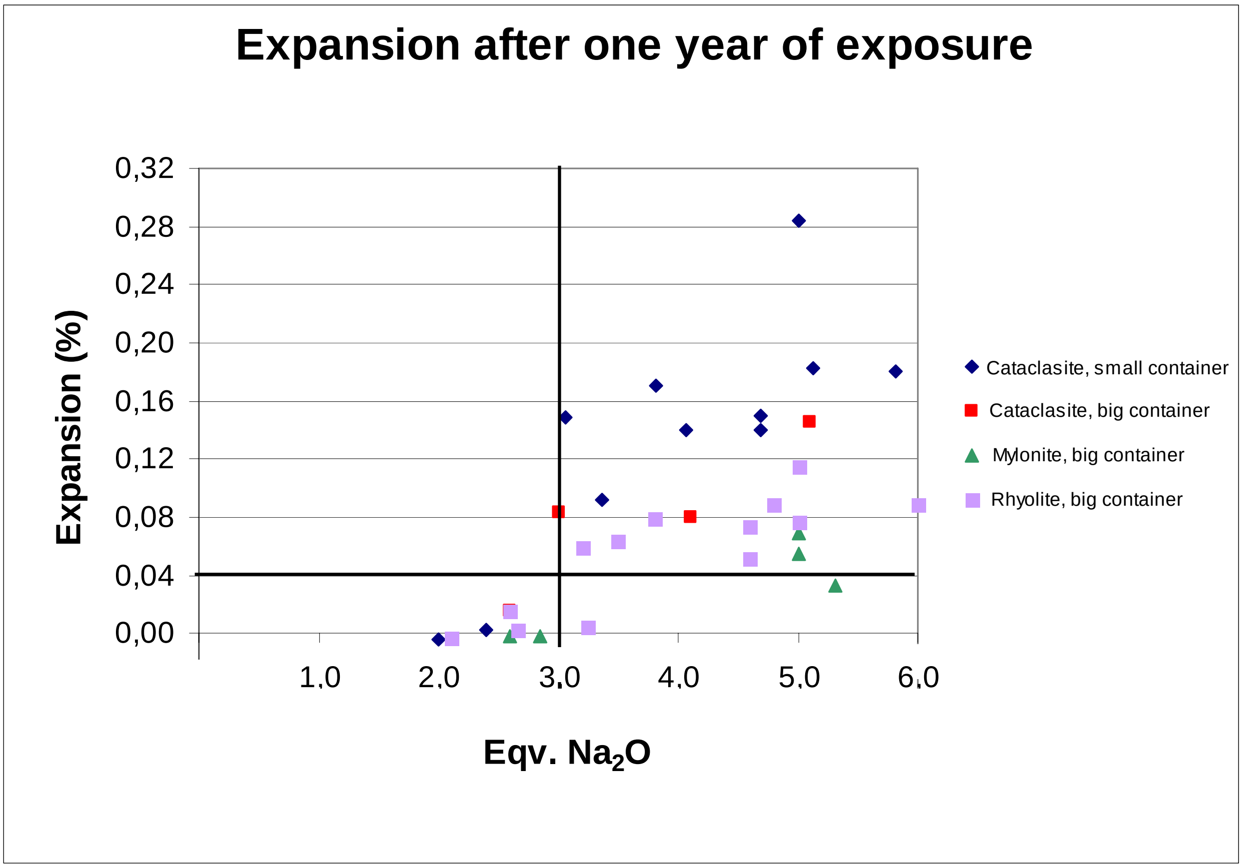

Figure 3.3-3: Threshold limit of alkali for three different types of Norwegian aggregates tested by the Norwegian concrete prism test. (Lindgård & Wigum, 2003).

The variation of the critical amount of alkalis in the concrete makes it valuable with national guidelines regarding critical parameters, as the reactivity of the aggregates differs from region to region. Various countries have also guidelines allowing for testing for critical amount of alkalies, for instant Norway (Dahl, 2004).

3.3.3 Experience with alkalis from external sources

Several tests have shown that alkalis from external sources increase expansions due to AAR (Chatterji et al. (1987), Okada et al. (1986), Kawamura et al. (1988), Sibbick & Page (1998).

One of the Danish examples is a railway bridge where the epoxy membrane on the top of the bridge deck is locally defected. Thus water has been supplied to the concrete in the bridge deck containing a relatively high amount of reactive material. As the bridge is a railway bridge no de-icing salt has ever been used, which means that no alkalis has been added from external sources. So far (after at least 5 years of defected membrane) only harmless AAR has been observed. With concrete cores from the bridge deck, tests of the residual reaction capacity has been carried out by Rambøll by placing one prism in a saturated NaCl solution and placing another prism in a sealed container with free water in the bottom and with the test prism wrapped around with a wet cloth. Both prisms are stored at 50°C. The test results presented in Figure 3.3-4 show that the alkalis from the NaCl solution has contributed to the reactions in the concrete, as the measured expansion of the prism stored in the saturated salt solution is much higher than the expansion of the prism stored in water (Rambøll, 2001).

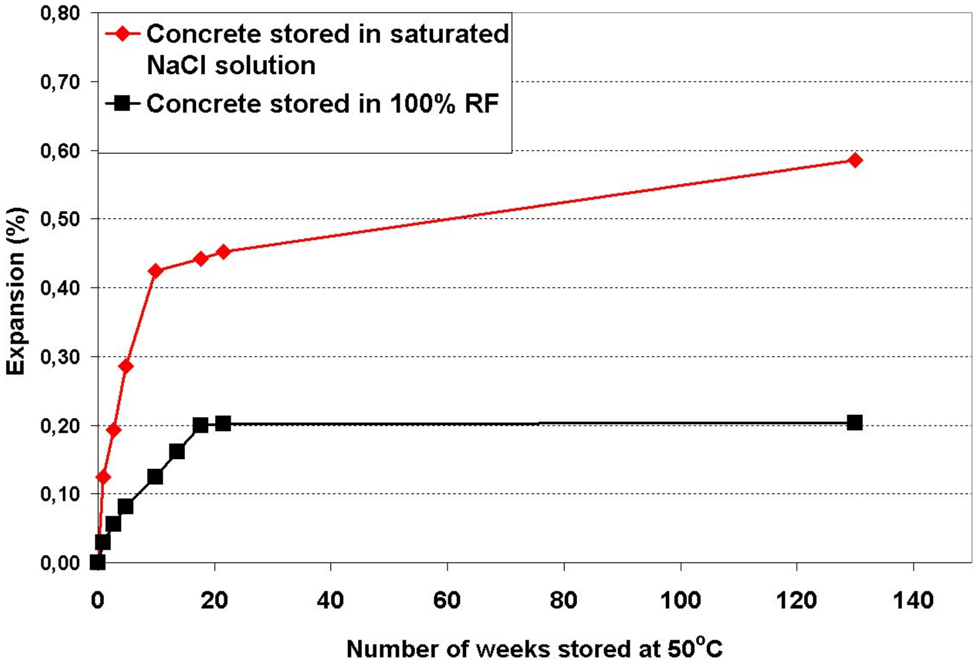

Figure 3.3-4: Test results from test of residual reactive capacity (Rambøll, 2001).

3.4 Role of calcium hydroxide (CH)

There exist different views on the role of $Ca(OH)_2$ (=CH). According to the hypothesis developed for opal by Powers & Steinour (1955), the ratio between lime (CH) and alkalis is a major controlling factor. A high enough amount of lime at the reaction site gives a non-expansive lime-alkali-silica complex, while a small amount of lime gives a swelling alkali-silica gel. The lime must diffuse to the reaction site, through the thin lime-alkali-silica layer initially formed at the silica particle. The thicker the reaction layer and the higher the alkali concentration, the more difficult it should be for the lime to reach the reaction site fast enough to produce a non-swelling complex. According to this hypothesis, the reaction of lime may release some of the initially reacted alkalis. Consequently, a regeneration of alkalis free to further reactions will occur.

Dent Glasser & Kataoka (1982) carried out experiments on model systems of silica gel, sodium hydroxide and calcium hydroxide. They found that if considerable amounts of calcium ions remained in solution, CSH forms. After that, the systems behave in the same way as model systems without calcium. The reaction between silica, calcium and water to form CSH is a pozzolanic reaction.

An alternative mechanism has been proposed by Chatterji et al. (1986 and 1992) where it is described that $Ca(OH)_2$ accelerates penetration of alkali ions, Ca^{2+}, OH^- as well as water in a reactive grain and it decelerates diffusion of silicate out of the reactive grain. If the concentration of alkalis outside the reactive grain is high and crystalline $Ca(OH)_2$ is present at the same time, more materials are pumped into the reactive grain than the diffusion of silicate out of the grain and expansive forces are created. It is concluded that due to the chemical reactions and transport of ions, the expansion decreases with decreasing amount of crystalline $Ca(OH)_2$ and the swelling gel should contain a substantial amount of lime. The mechanism is illustrated in Figure 3.4-1.

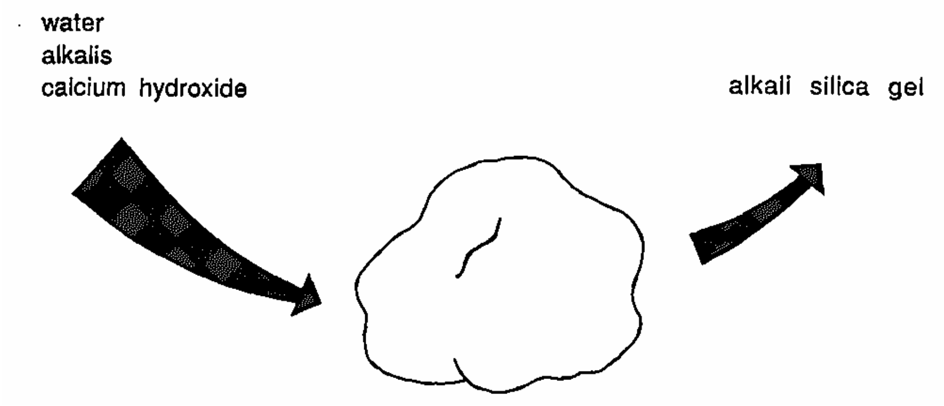

Figure 3.4-1: Schematic model of AAR. If the amount of water, alkalis and calcium hydroxide entering the reactive grain is larger than the amount of alkali-silica gel leaking out, the particle expands and causes cracks in the surrounding cement paste. Chatterji et al. (1992).

The solubility of lime (CH) decreases with increased temperature – see Figure 3.4-2. Thus the maximum rate of expansion is a compromise between increasing rate of reaction due to increasing temperature and decreasing rate of expansion due to decreasing concentration of $Ca(OH)_2$ in solution at higher temperature.

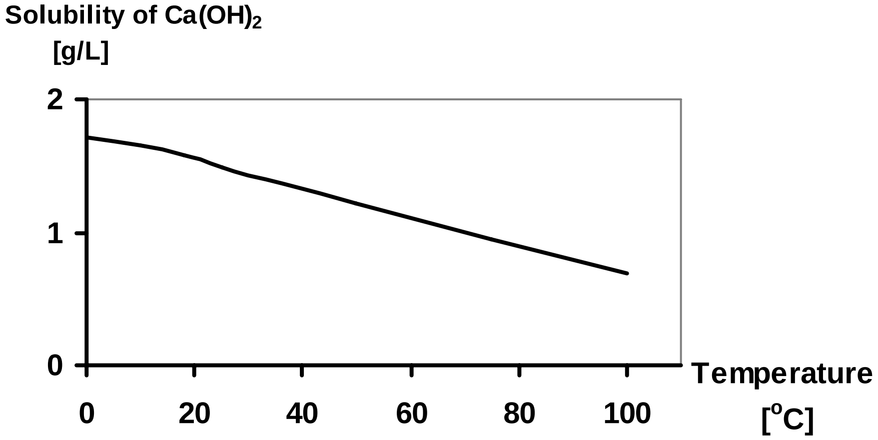

Figure 3.4-2: Illustration of the solubility of $Ca(OH)_2$ versus temperature.

The view of Chatterji et al (1986, 1992) is basically confirmed by the work of Wang & Gillott (1991). They concluded that the presence of CH worsens the AAR and increases the expansion of mortar bars containing opal. They stressed the importance of CH as a buffer to maintain a high concentration of hydroxide ions, i.e. high pH. Additionally, the calcium ions may exchange for alkali ions, then releasing alkalis being important to produce a reactive alkali-silica complex. However, the exchange of calcium ions for alkali ions in the swelling alkali-silica complex may produce a non-swelling lime-alkali-silica complex.

Struble 1987 (reported in Jensen 1993) made a set of experiments of opal, limestone and a solution being similar to the normal pore solution in concrete. No cement was added. The experiments showed that the opal simply dissolved, and in the absence of hydration products, no gel was formed. The conclusion from these experiments was that the calcium is necessary to produce an alkali-silica gel.

The role of CH seems confusing and complex, but there is clear evidence in the literature that at least some CH is essential to produce a gel of swelling nature. The similarity between the alkali-aggregate reaction and the pozzolanic reaction seems obvious. According to Diamond (2002), the gel may start as a labile alkali-silica gel, migrate, and pick up some calcium to become an expansive calcium-alkali-silica gel. Further, the gel may pick up more calcium and loose some alkali and end up as an alkali rich CSH, which is not expansive.

As the solubility of SiO_2 is very dependent of the pH the reaction only takes place at a practical rate at pH > 12.4, which is the pH of saturated calcium hydroxide solution. For field structures this means, that the alkali aggregate reactions stop if the cement paste around the reactive grains is carbonated.

3.5 Types and content of reactive rock types

As indicated in chapter 2 the types of reactive rock rocks / aggregates influence the reaction, as some reactive constituents are rapidly reactive, while other reacts slowly. The first group includes rock types such as porous flint, chert and opal, while the second group is characterized by having various amounts of microcrystalline quartz (VCJ, UK, ROA, 1995). In Figure 3.5-1 test results from tests according to the Danish test method TI-B51 is shown for two types of sands – a Danish sand, Nymolle, which is rapidly reactive and an Icelandic sand, Ísafjördur Pollur, which is more slowly reactive (Thaulow and Ólafsson, 1981). Several of the rapid reactive rock types show a pessimum behaviour – see chapter 3.5.1.

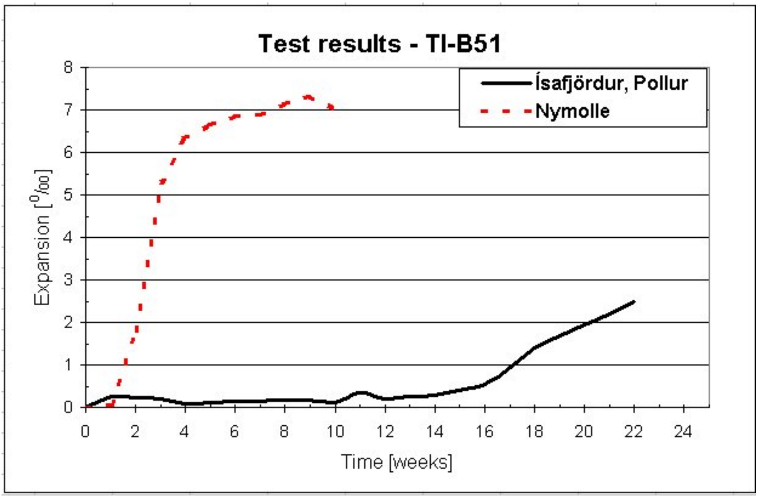

Figure 3.5-1: Illustration of test results extrapolated from the Nordtest project 173-79 (Thaulow & Olafsson, 1981). Expansion versus time for tests performed with a rapidly reactive sand (Nymolle) and a more slowly reactive sand (Ísafjördur Pollur).

Most slowly reactive rock types show a “normal expansion behaviour”, i.e. for a given alkali content an increased amount of the reactive material leads to an increased expansion. This is for instance the case for all known Norwegian reactive rock types. Results from testing of 60 Norwegian natural sand types both with petrographic analysis and according to the accelerated mortar bar test (80°C, prism size 40·40·160 mm) are presented in Figure 3.5-2 (Lindgård and Haugen 2000). Each sand type is denoted with the symbol for the dominating rock type (if the content is > 50 %). As shown in Figure 3.5-2, the overall tendency is an increased expansion with an increasing content of reactive- and potentially reactive rock types. However, when the content of reactive- and potentially reactive rock types exceeds about 40-60 %, a somewhat less expansion is revealed. This indicates a pessimum phenomena for the accelerated mortar bar test method, i.e. it is assumed to be a testing problem and not a pessiumum behaviour of the Norwegian reactive rock types. This assumption is confirmed by later results from a survey of a huge number of Norwegian concrete structures, mainly bridges (Lindgård and Wigum, 2003).

Other findings from the Norwegian research project (Lindgård and Haugen 2000) were that no significant difference in expansion rate could be found in the accelerated 80°C mortar bar test between different rock types or between natural sand and crushed coarse aggregate (gravel or crushed rock) with equal amount of reactive- and potentially reactive rocks (see Figure 3.5-2).

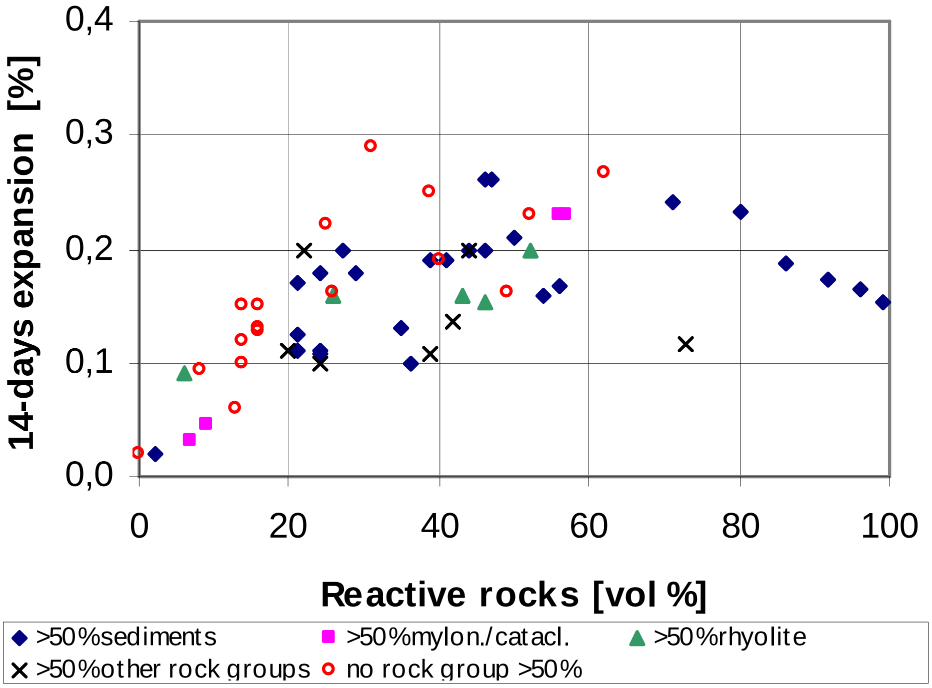

Figure 3.5-2: Results from testing of 60 Norwegian natural sand types both by petrographic analysis (abscissa) and by the 80°C accelerated mortar bar test (ordinate). Each sand type is denoted with the symbol for the dominating rock type (if the content is > 50 %). (Lindgård and Haugen 2000).

3.5.1 Pessimum behaviour

The pessimum behaviour is a term of illustrating that the maximum expansion will occur if a certain amount of alkalis is present for all reactive material to react. If less reactive material is present in the concrete for the same amount of alkalis, less expansion will occur. If on the other hand a lot more reactive material is present than the amount of alkalis needed for all reactive material to react 100 %, much lesser expansion can be observed. In the last mentioned situation a lot of reactive grains have to share the few alkalis, which leaves only little alkali for each grain resulting in little expansion.

Considering a concrete containing a rapid reactive aggregate many tests have shown that pessimum behaviour of the reaction as regards to the ratio between the alkali content and the amount of reactive silica (Hobbs, 1988 and Poole, 1992). The pessimum behaviour is not that significant with the slowly reactive aggregates as these aggregates usually consist of more types of constituents. Thus a reactive Norwegian rhyolite contains maybe 5-10% reactive constituents while a reactive Danish porous flint contains approximately 100% reactive constituents.

The pessimum behaviour is illustrated in Figure 3.5-3, which shows test results from tests with different amounts of alkali content in concrete with varying amount of reactive silica from Danish aggregates. The tests are made with mortar bars stored in sealed containers at 38°C.

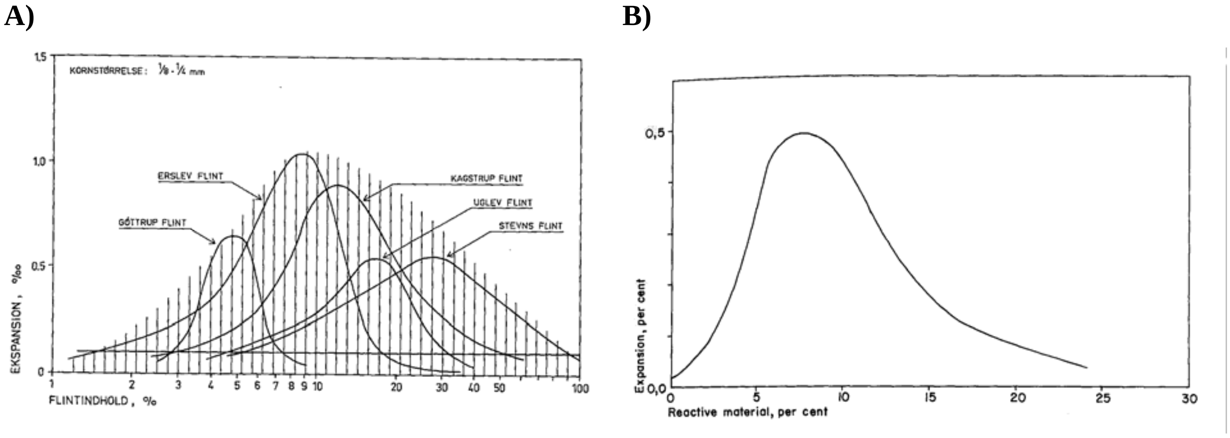

Figure 3.5-3: General outline of the dependence of mortar bar final expansions on the amount of reactive material in the aggregate for different Danish flint aggregates at a constant alkali content in the cement. The illustrations are based on several tests with Danish aggregates (Committee on alkali reactions in concrete, 1966 and Plum, 1961).

The curves shown above illustrates that for the Danish flint aggregates a content of reactive aggregate lesser than approximately 2% or larger than approximately 20% could considered to be "safe" as regard to AAR. As indicated by the illustration to the left in Figure 3.5-3 the exact shape and placement of the expansion curve depends on the specific type of aggregate. The "safe" criteria therefore differ a lot according to the aggregate considered. Furthermore the placement of the curve will be affected by the alkali content.

The pessimum behaviour is an important factor when evaluating the different test methods. In the method RILEM AAR-4, a fixed amount of alkalis is used. The same is the case with RILEM AAR-3 (which is based on the Canadian method), the German method and the Norwegian method. The original Canadian Concrete prism method was significantly modified in the mid 1990´s by raising the cement content (from 310 to 420 kg/m^3), the alkali content (from 3.88 to 5.25 kg/m^3) and the storage temperature (from 23 to 38^{o}C). The risk of blind use of this test method on rapid reactive aggregates is illustrated in Figure 3.5-3 as the pessimum value of expansion may not be achieved. On the other hand the test methods using an unlimited amount of alkali should be better able to identify if an aggregate is potential reactive as the alkali content will be sufficient to reach the pessimum value and thereby make sure that the maximum expansion will occur eventually.

3.5.2 Experience with pessimum behaviour in real structures

In Denmark several concrete structures built before 1960 contained a very high amount of porous flint, which is known to be highly reactive. The flint was contained in the fine as well as in the coarse fraction. Most of these structures do not suffer from deleterious damage due to AAR as the content of reactive material is much higher than the pessimum content of alkalis, and only harmless AAR has occurred – corresponding to the right part of the curve in Figure 3.5-3 B). However, in the 1960's one realized that the stone particles were very reactive and therefore inert granite aggregates were used. The sand was not considered to be reactive and during the 1960's to 1980's a lot of concrete structures were built with inert coarse material and very reactive fine material. A lot of these structures suffer from severe damage due to AAR, which corresponds very well to the new placement of the pessimum curve in Figure 3.5-3 B). In evaluating structures like this, one has to bear in mind the environment of the structure as the extent of damage in a structure like this could develop rapidly if alkalis are suddenly added to the structure from outside sources.

3.6 Influence of aggregate particle size

The size of the reactive aggregate particles influences both the rate of the alkali reactions, the expansions and the extent of damage on concrete structures due to AAR. It appears to be different effects dependent on various test methods, different reactive aggregate types, and differences in laboratory versus field experiences.

3.6.1 Laboratory experiences

Stanton first discussed the relevance of the particle size in 1940, where he demonstrated that the amount of expansion was related to the particle size (Diamond & Thaulow 1974). Reactive aggregates in the range from about 0.17 – 0.6 mm gave greater expansion than the coarser sizes. The aggregate in this case was a siliceous magnesian limestone, containing opal and chalcedony. Vivian found in 1951 a similar pattern when studying the same aggregate (Diamond & Thaulow 1974). However, he found that aggregate sizes smaller than 50 µm gave no expansion at all, while the fraction between 50 and 70 µm caused a delayed expansion.

Results from another study of Vivian (in Hobbs & Gutteridge 1979) used Australian opaline rock as the reactive constituent (5 %) together with a non-reactive aggregate. The period where no expansions occurred seemed to be prolonged for the particles below 150 µm. The expansions were subsequently high for all particle sizes down to 50µm. No expansion was observed for particles below 50 µm. The storing temperature was 20°C in this set of experiments.

Diamond & Thaulow (1974) carried out an investigation with opal of α-cristobalite type, which was crushed and sieved into fractions down to 20-30 µm. Standardised non-reactive sand was used as reference aggregate, while 5 or 10 % crushed opal of different sizes was added to the mortars. The specimens (cylinders with a diameter of 10 mm and a length of 20 mm) were stored above water at a temperature of 20°C. Results after approximately 300 days of exposure are given in Table 3.6-1.

Table 3.6-1 Effect from different ranges of particles on expansion. Data from Diamond & Thaulow (1974).

Size range (µm)	% expansion (5 % opal)	% expansion (10 % opal)
Appr. 150-125	0.65	1.61
125-90	0.52	1.74
90-74	0.77	1.49
74-53	0.57	1.77
53-44	0.89	2.08
44-30	0.54	2.81
30-20	0.13*	2.08

*Represents mean value of 3 cylinders, expansions for 2 of these were 0 or close to 0.

The results presented in Table 3.6-1 show that 10 % opal gave significantly higher expansions than 5 %. However, the results did not give evidence to any significant effect of particle size as all particle sizes were capable of producing large expansions.

Hobbs & Gutteridge (1979) examined Beltane opal of different fractions in the range from 150 µm to 4.8 mm. They made mortars with different amounts of reactive materials (from 1 to 20 %). Each fraction was tested separately in combination with non-reactive sand. The specimens (25 x 25 x 250 mm) were stored at a temperature of 20°C in water. For a given content of Beltane opal, the expansion increased with decreasing particle size. This effect was pronounced for an opaline content in the range between 4 and 6 %. They concluded that the cracking of the specimens was not a function of the particle size, but on the volume of reactive particles. A noteworthy difference between the investigation of Hobbs & Gutteridge (1979) and that of Diamond & Thaulow (1974), is that Hobbs & Gutteridge observed intensive cracking in the specimens, while Diamond and Thaulow observed much less cracking than could be expected from the large expansions. They found that expansion occurred suddenly and was completed in a relatively short time for fine reactive particles, while with larger sizes the expansion was slower and much more gradual. The large differences in specimen size may be responsible for some of the observed differences in cracking intensity.

Hobbs and Gutteridge (1979) showed that a higher reaction rate and a larger expansion were achieved with decreasing size of the reactive particles. They carried out the test with mortar bars stores in water at 20°C. The reactive aggregate was an opaline rock from California (Beltane opal) and the alkali content of the cement was 1.15% Na_2O_{eqv}. Besides the opaline rock (used in different gradings) Thames Valley sand (the four open symbols in Figure 3.6-1) or a limestone were used (the solid symbol in Figure 3.6-1). The measurements were carried out for 110 days.

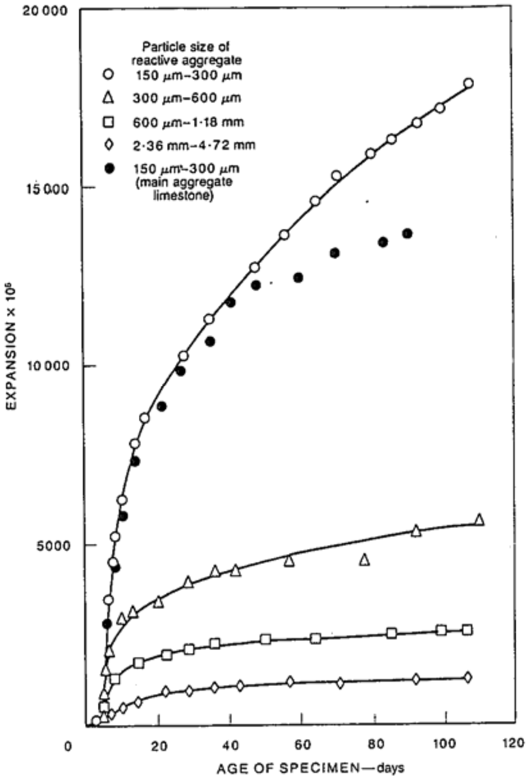

Figure 3.6-1: Expansion versus age for mortar bars containing 6% of reactive aggregate by weight of total aggregate for different particle sizes of the reactive aggregate Hobbs and Gutteridge (1979).

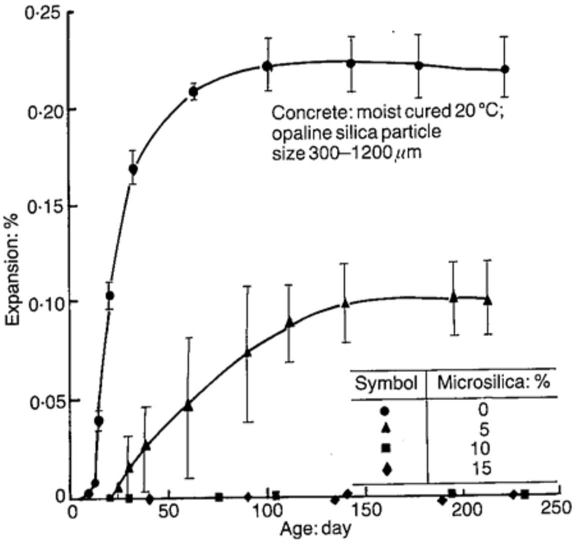

Figure 3.6-2: Influence of microsilica content on expansion; water/(cement + microsilica) and aggregate / (cement + microsilica) ratios are 0.41 respectively 3. Equivalent Na_2O content ≤ 5.0 kg/m^3 and 0.5% opaline silica by mass of total aggregate was used. Hobbs (1988).

However, more experiments show that this is only the case above a certain lower grain size limit. When the size of the reactive particles are smaller than a certain value, the expansion decreases. This corresponds to a pozzolanic effect.

The smaller grain the larger surface area and thereby also a larger reaction surface, which could explain why the reaction rate increases by decreasing size of the reactive grain. The advantage of this effect is achieved by replacing part of high alkali cement with microsilica. Test results showing the effect of microsilica are illustrated in Figure 3.6-2. (Hobbs, 1988).

Different explanations for the positive effect of microsilica have been made. According to Hobbs (1988) alkali aggregate reactions caused by the silica of the microsilica and the alkali in the cement is going on at a fast rate before the concrete is hardened. What happens regarding AAR in a concrete with high alkali cement and reactive aggregate, i.e. in the sand fraction, when part of the cement is substituted with microsilica, is that the small reactive particles (the microsilica) are more rapidly reactive than the larger size reactive particles (the reactive sand). The alkalis from the cement will then be used on reactions with the microsilica. These reactions happen so fast that the concrete has not yet gained its total strength. If the optimal amount of microsilica is used, the alkalis from the cement are consumed on these reactions and no alkalis are left for reactions to occur with the reactive sand. This indicates, however, that the concrete is still vulnerable to alkalis from external sources.

As it is seen from Figure 3.6-2 the effect on expansion due to microsilica is greatly dependent of the amount of microsilica by cement at a given alkali content. For a specific concrete an optimum amount of cement to be substituted by microsilica can be determined to eliminate deleterious expansion.

However, other claim that the effect of the microsilica (and other pozzolanes) are due to the fact that the microsilica binds the calcium hydroxide and thereby reduces pH, which reduces the reaction rate remarkably.

In Iceland experience with microsilica has been gained since 1979 – see appendix A.7.

A pozzolanic effect has also been observed with other substitutes than microsilica. Chatterji et al. (1986) showed the effect of using moler (diatomaceous earth) grounded to cement fineness in test specimens. Some of the test results are illustrated in Figure 3.6-3. From Figure 3.6-2 and Figure 3.6-3 it is seen that a larger substitution is needed of moler compared to microsilica to reduce the expansion.

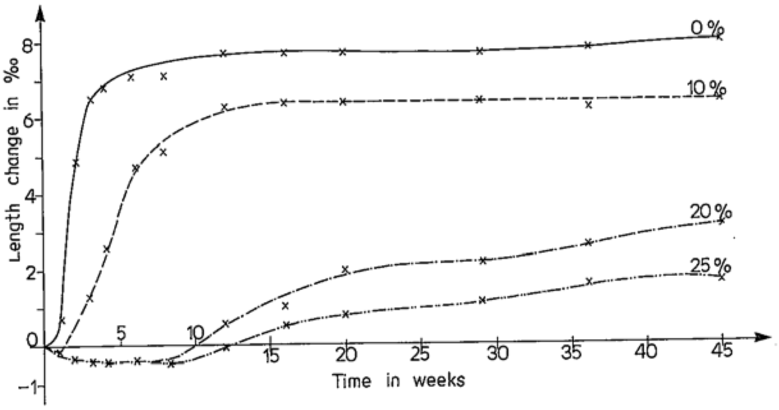

Figure 3.6-3: Test results from tests with mortar bars containing True sand and different amounts of moler stored in NaCl bath at 50°C. (Chatterji et al., 1986).

Qinghan et al. (1996) reported that use of a powder from an alkali reactive rock has proven to suppress the expansion caused by the alkali aggregate reaction. They used a reactive andesite with α-cristobalite as the primary reactive component, ground to Blaine-values in the range from 290 to 1133 m^2/kg. The prisms of size 40 x 40 x 160 mm were autoclaved before they were stored at a temperature of 45°C for long term measurements. The powders were used as cement replacers in this study, while NaOH were used to adjust the alkali level in the range from 0.5 to 3.0 % Na_2O eqv. Generally, all fillers gave a reduced expansion when replacing cement. The effect on reduced expansion was greater for the finest fractions.

Pedersen (2004a,b) investigated in his PhD-study the possible mitigation effect of alkali-reactive fillers (particles less than 0,125 mm) from two Norwegian cataclastic rocks, along with fillers of Icelandic glassy rhyolite and crushed bottle glass. Non-reactive reference fillers were included in the study, as well as silica fume and fly ash known to mitigate alkali-silica reactions. When testing the 0-20 µm fractions of the different fillers at 20°C, the materials could be divided into two distinct classes with respect to pozzolanity:

- The pozzolanic reactivity of fly ash, glass and rhyolite filler was distinct
- The pozzolanic reactivity of mylonite, cataclasite and quartz fillers was insignificant at the age of 28 days

All the materials being highly pozzolanic were found to have a distinct amorphous silica phase, while the silica phase of the non-pozzolanic materials was shown to be well crystalline quartz.

During the years, several studies of crushed glass utilised as concrete aggregates have been published. Generally, the glass is very deleterious, causing large expansions and damages. However, fine fillers of glass have been reported to cause the opposite effect, i.e. a reduced expansion tendency. Shao et al. (2000) have shown that glass fillers smaller than 38 μm are able to reduce the expansion relative to the control mortar containing no glass filler. This fine fraction of glass exhibits a pozzolanic behaviour that may explain this effect, and an increase in strength was also observed for this filler. The coarser fractions had less effect. The experiments were performed using the accelerated mortar bar test. Hudec & Ghamari (2000) reported a similar set of experiments, where glass fractions below 75 μm reduced the expansion. On the other hand, fractions larger than 75 μm gave a tendency of increased expansion of the mortar bars.

Various researchers (Hobbs and Gutteridge 1979, Diamond and Thaulow 1974, Lagerblad and Trägårdh 1992, Jensen 1993, Lindgård & Wigum 2003) have shown that AAR is dependent on the size of the reactive particles. Different experiences are observed in the laboratories versus in the field, and in addition, the influence appears to differ between rapid- and slow/late alkali-reactive aggregates.

An experimental investigation was carried out by Nishibayashi and Yamura (1992) to clarify the effects of reactive fine aggregates on the expansion characteristic of concrete due to AAR. By using accelerated test methods (prisms were stored at 100 % RH and 40ºC), it was observed that the expansion of concrete containing only reactive fine aggregate increased rapidly to a constant level during the early period of the reaction. On the other hand, the expansion of concrete containing only reactive coarse aggregate initially increased slowly, but continued for a longer period.

Sprung (1975) found that the pessimum expansion also changed as the particle size of the aggregates changed. It has also been demonstrated that there is some kind of pessimum size of particles for rapid reactive aggregates. Experimental work showed the pessimum size of opal particles to be about 0.2 mm when testing expansion according to the mortar-bar method (Kawamura et al. 1983).

Lagerblad & Trägårdh (1992) have reported that for Swedish slow reacting aggregates the particle sizes (1-2 mm) cause greater expansions than the coarser sizes. The expansion tests were performed using the Danish method (TI-B 51), where the prisms are stored in 50ºC NaCl solution. This contradicts observations in structures, where the larger fractions were found to be as deleterious as the sand fractions.

In a study by Mørtsell and Wigum (reported in Wigum, 1995), substitution of innocuous material with reactive materials showed that reactive material in the finer fraction (0.15 to 0.8 mm) gave the highest expansion in an accelerated mortar bar test (see Figure 3.6-4). The corresponding experiment with reactive material in the larger fractions (0.8 to 4.8 mm) gave significantly lower expansion. A similar test with the same total quantity of reactive material (50%) equally distributed in all fractions gave an even smaller expansion. The described differences were observed for all ages above 28 days, while there was no significant difference at 14 days of exposure.

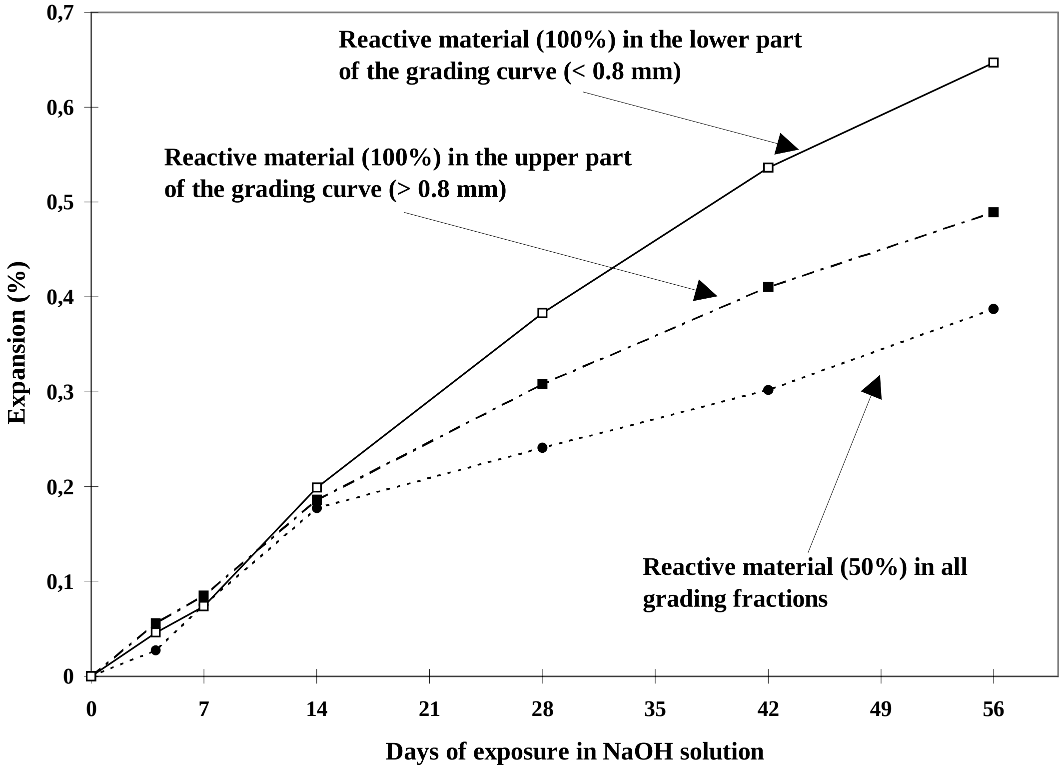

Figure 3.6-4 Expansion results from the Norwegian Modified NBRI mortar-bar test (80°C) with reactive materials substituted in different fractions in an innocuous aggregate. (Wigum,1995)

The influences of aggregate size and aggregate size grading on AAR expansion in the Chinese autoclave test were studied by Zhang et al. (1999). The result showed that the effect of the addition of coarse aggregate on the expansion of mortar is complex. It may be inhibiting expansion, may have no effect, or may be promoting expansion. Moreover, its effect will change with age. By theoretic analysis of micromechanics, it was showed that the effect of a coarse aggregate on the expansion of a mortar depends on the relation between the free expansion of the coarse aggregate and the mortar. From the results of experimental and theoretic analyses, the authors suggested that for the evaluation of the alkali-reactivity of aggregates, single-size aggregate gradings might be more reasonable.

The influence of the particle size of the reactive aggregate and the aggregate angularity on AAR were studied by Ramyara et al. (2005). An all-in natural reactive aggregate was used, while the coarse aggregate particles were crushed to obtain crushed fine particles. ASTM C1260 accelerated mortar bar test was conducted to compare the AAR expansion caused by various aggregate size fractions. The effect of the size of the particles on ASR expansion was studied by replacing each size fraction of the non-reactive aggregate with the reactive aggregate of the same size. In spite of similarity of the chemical and mineralogical compositions, the crushed aggregate caused higher AAR expansion than the natural aggregate in all size fractions. The summation of the expansions of individual reactive size fractions of both aggregates was found to be higher than that of corresponding control mixtures.

Foray et al. (2004) studied the influence of the size of the reactive particles and the distribution of the reactive particle sizes on the development of AAR. The experimental study confirmed that aggregate grading and particle size greatly influence the AAR development. It was claimed that each diameter of reactive aggregates has its own AAR gel formation kinetics. Simple relation does not exist between AAR expansion and aggregate specific surface. When several diameters of reactive aggregates are present together, competition phenomena appear, and result in coupling factors.

Kuroda et al. (2004) examined the effects of particle size, grading, and content of reactive rock types on the AAR expansion of mortar bars boiled in an NaOH solution inside an autoclave apparatus. From the experiments it was concluded that the expansions of the mortar bars increased as the size of the reactive aggregate decreased (the volume of the reactive rock types was constant). There was found a correlation between the expansion and the total surface area of the reactive rock types, independent of the particle size and the grading of the reactive aggregate. The expansions of the mortar bars could be expressed as functions of the total surface area and the volume of the reactive rock types.

It is evident that by using a concrete prism test, the real aggregate grading can be examined, which will more precise echoing the situation in real structures.

The mortar bar test (80°C) requires a specific grading of the aggregate. Various researchers have observed that for slowly reactive aggregates, certain particle sizes in particular contribute more to the deleterious reaction than others. Jensen (1993) reported that in Norwegian structures suffering from AAR, reacted aggregates have only been observed in the coarse aggregate and in the coarser sand fraction (>1-2 mm), while in the NBRI mortar-bar test with similar aggregates, all the particle sizes seemed to react.

Wigum & French (1996) studied the accelerated expansion of mortar bars (with real concrete grading, where samples were cut from the original concrete) and mortar bars with grading according to the accelerated mortar bar method, both with a reactive greywacke (see Figure 3.6-5). It was found that mortar bars with the fine grading expanded more rapidly in the beginning, while the identical sized mortar bars with coarser particles exhibited an ultimately higher total expansion. Further, for most of the samples, examined in thin sections, they found that the finest particles, below approximately 0.30 mm, did not react.

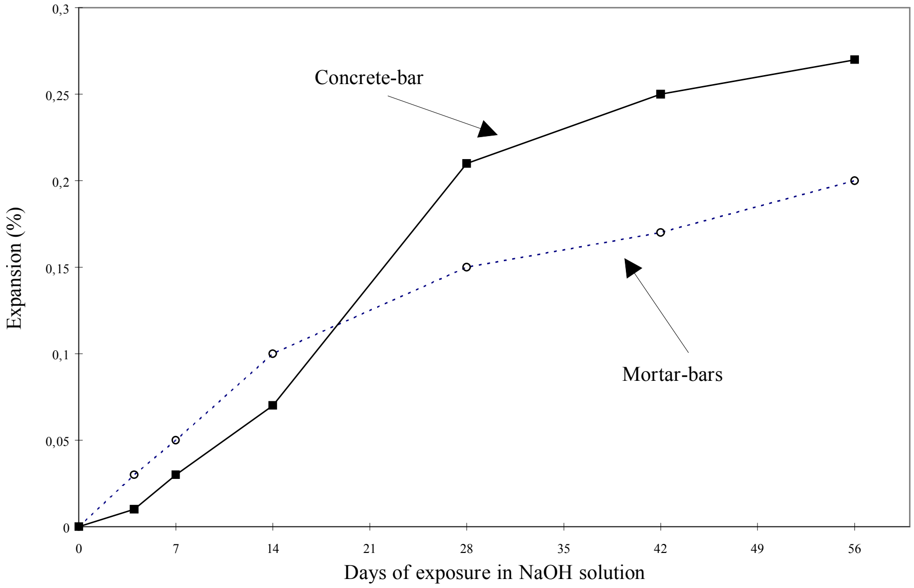

Figure 3.6-5 Average percentage expansion of mortar bars with mortar bar grading (named “Mortar-bars”) and real concrete grading (named “Conrete-bar”), respectively, both made with the greywacke aggregate (Wigum & French (1996).

It is an interesting question why small slow/late reactive aggregate particles do not seem to take part in the alkali aggregate reaction. It is commonly found that more serious cracking relates to clustering of reactive aggregate particles, and many cracks are developed as a consequence of the accumulation of chains of reaction particles. Coarser particles create longer cracks running out into the cement paste from the aggregate than finer particles, and consequently contribute to longer interconnected cracks along chains of reactive clusters. The expansion forces required to crack the concrete might be dependent upon this accumulation of reactive particles of a particular size. In real concrete, coarser particles will also have access to a greater concentration of alkalis due to the low surface area to volume ratios than would a similar volume of small particles because of the lower surface area to volume ratio. This might not be the situation in accelerated tests where there for many test methods are large excess of alkalis, but it is also evident that larger aggregate particles will have a greater potential for carrying the minute amounts of actually reactive lithic material and a greater potential for internal crack development than the smaller grains.

3.6.2 Experiences from real structures

According to French (1994) the alkali-reaction is most damaging for certain rock types when the reactive rocks occur in a particular size range. It has been observed that for rapid reactive rock types, such as chert and volcanic glasses, the most damaging size range appears to be between 3 to 7 mm. However, for slowly reactive rock types such as recrystallised sandstones and meta-quartzites the most damaging size range appears to be between 10 to 20 mm. Greywackes and argillites have been found to be most damaging when they occur as coarse aggregates, and damage has been observed in cases where the aggregate reaches several centimetres across (French 1994).

In Japan, only a few examples of deterioration due to AAR in structures containing reactive fine aggregate have been reported. Nishibayashi and Yamura (1992).

Lagerblad and Trägårdh (1992) and Jensen (1993) have observed that for slowly alkali-reactive aggregates, certain aggregate size fractions in particular contribute more to the deleterious reaction than others. It was observed that finer slow/late alkali-reactive particles do not seem to take part in the reaction in real structures. In Norway, thin section examinations of reacted concrete from structures seldom show particles sizes less than 1-2 mm to have reacted (Jensen 1993).

In most cases only aggregates particles larger than 8-10 mm lead to deleterious alkali aggregate reactions in Norwegian concrete structures (Lindgård & Wigum 2003). This is taken into account in the revised Norwegian regulations (Dahl et al., 2004). In these regulations (Norwegian Concrete Association, 2004) a “grain size factor” is applied in the calculations of the critical amount of the reactive rock types. For fine aggregates (0/4 mm and 0/8 mm) the factor is 1.0, while for coarse aggregates (8/16 mm and 16/22 mm) the factor is 2.0. For fine coarse aggregates (2/8mm and 4/8mm) the factor is 1.5.

For slow/late alkali reaction cracking occurs along discontinuous structures in the aggregate, such as grain boundaries and foliation. The ingress of alkalies and following reaction will especially be preferable where these discontinuous structures contain a high amount of finer grains and sub grains. To contain these discontinuous structures, particles have to be of a certain size. Particles finer than the width between parallel foliation or finer than original crystal sizes (monocrystalline) in the rock, will not contain any discontinuous structures. In such fine particles there will be no initial discontinuous structures which provide access to the alkalies.

3.6.3 Aggregate grading versus cement content

During the last decades much attention has been drawn to the alkali-reactivity of different aggregate types. However, in spite of the voluminous research performed, the effect of grading upon the reactivity and expansivity of aggregates, has not been considered to any great extent. The grading may influence in different ways the degree of alkali-reactivity for a concrete system containing alkali-reactive aggregate. The total amount of alkalis within the system will be dependent on the excess of cement in the system, which is governed by the density of the aggregate particles, which in turn is governed by the grading curve. The relative amount of alkalis available to take part in the reaction will be governed by the available surface area of the aggregates.

The principle of aiming a maximum density means specifying a grading curve which gives a minimum of open space (voids) between the particles. This volume of voids will stand in relation to the necessary amount of cement to be added to fill this space. It has been found that if the aggregates are graded to obtain a maximum density, a harsh and somewhat unworkable mix is normally the result. To improve the workability of the fresh concrete, an excess of cement above that required to fill the voids is added. This additional amount of cement will give an overfill and lead to an increasing spacing between the aggregate particles. One effect of this overfill, will be increased amount of alkalis, and consequently this will have an effect upon the alkali-reactivity of the concrete system. A grading curve for a typical structural concrete will give an open space of about 30%. Commonly, 33-35% of cement paste is added to the system, which gives an overfill of 3-5% of cement. Sometimes it is difficult to evaluate the effective level of alkali present because of its relationship to the grading curve (French 1994). When designing grading curves for massive dam concrete, extremely coarse particles will be used to avoid high heat generation and shrinkage due to the large volume of the concrete. If the whole concrete is considered, the alkali level might be found to be low, however, if the potentially reactive sites are considered, the alkali concentrations might have to be regarded as locally high.

Concerning the total amount of alkalis in a certain concrete system, the aggregate particle size and grading in smaller domains should be considered. To symbolise this, two theoretical types of concrete domains with exactly the same aggregate/cement ratio of 2.25:1 are shown in Figure 3.6-6. Both of the concrete domains will have the same total amount of alkalis due to same volume of cement. The concrete with a finer grading curve has consequently a higher ratio aggregate surface area/cement content than the concrete to the right with coarser aggregate particles. An aggregate particle of a given size in the coarser graded concrete (to the right) will be surrounded by more cement paste and hence have access to a higher amount of alkalis per unit surface area than a similar particle in the finer graded concrete.

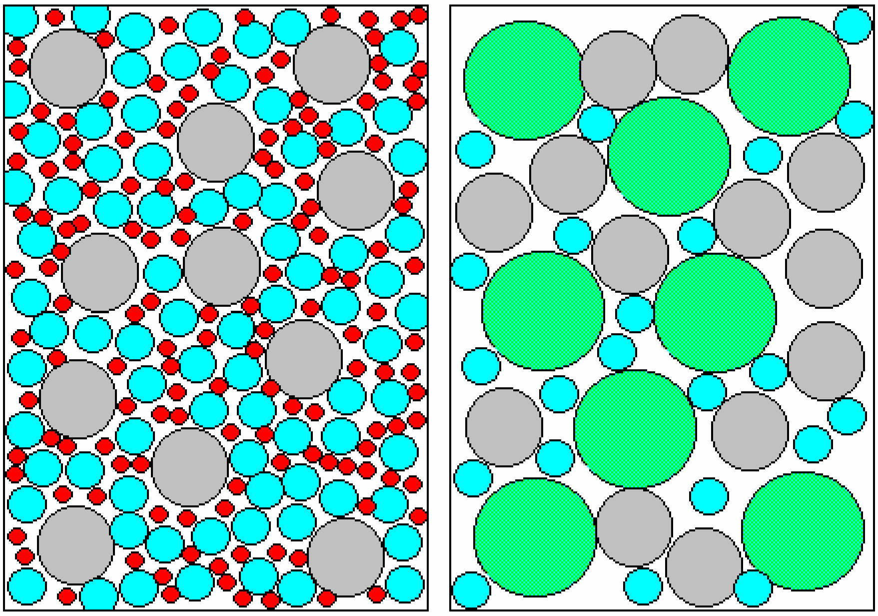

Figure 3.6-6 Two types of concrete domains with same aggregate/cement ratio (2.25/1) made of different grading. (Wigum, 1995)

3.6.4 Fracture mechanical approach to the effect of aggregate particle size

The following chapter is extracted from previous publication by Pedersen (2001 & 2004). The fracture mechanical approach to the size effect may be useful because fracture mechanical effects due may count for some part of the observed differences in expansion behaviour caused by particle size. Bache (1984) has treated the fracture mechanical aspects of relatively stiff aggregate particles embedded in a matrix of cement paste. The system of a shrinking matrix around a stiff particle or an expanding particle in the same matrix may be treated equally. Given that the expansion caused by the particle is greater than the strain capacity of the cement paste, a system with a large particle in a matrix will always lead to local fracture and cracking. On the other hand, if the particle is small, cracking may not be initiated even if the strain capacity of the matrix is exceeded. The reason for this is that cracking will only occur if the total deformation of the particle is greater than the cracking zone deformation needed to propagate a crack. In other words, the strain capacity of the system is higher than the strain capacity of the matrix.

According to Bache (1984), the controlling parameter for the increase in strain capacity is the inverse of the brittleness number B, which can be defined according to:

$$B = \frac{\sigma_0^2 D}{E\gamma_c}$$

σ_0	maximum tensile stress (N/m^2)
D	length, or diameter of a particle (m)
E	Modulus of elasticity (N/m^2)
γ_c	Fracture energy (N/m)

The brittleness number of a system will tell about the risk of crack propagation due to brittle behaviour. A higher number of B indicates a more brittle system with higher risks of crack propagation. The increase in B by increased particle diameter D will consequently lead to a higher risk of brittle behaviour. The fracture condition of an expanding particles in a matrix is illustrated in Figure 3.6-7. On the abscissa is the ratio of the actual particle size and the critical particle size, a ratio being proportioal to the brittleness number B. On the ordinate is the critical strain of the matrix, which is the ratio between the strain caused by swelling particles and the strain capacity of the matrix.

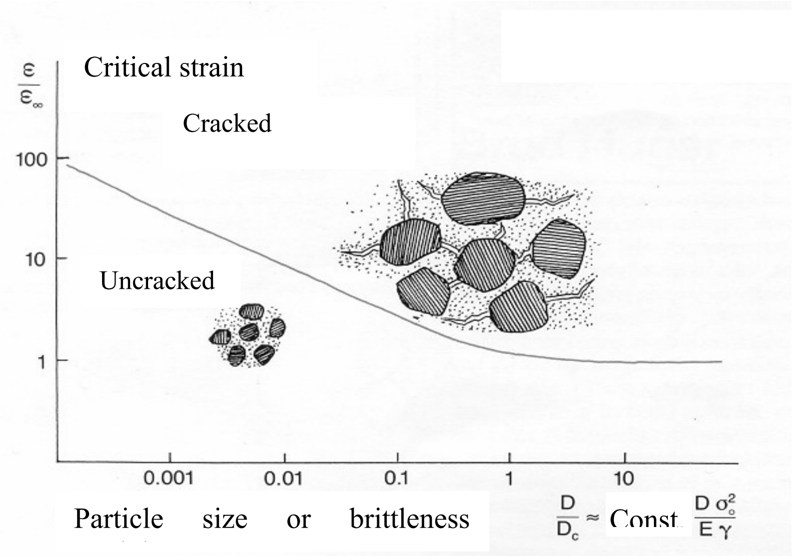

Figure 3.6-7. The critical shrinkage of matrix as a function of particle size or brittleness modulus. From Bache (1984).

An interpretation of Figure 3.6-7 is that for particles smaller than approximately 10 mm (only valid for this particular system and should not be generalized), the strain capacity of the system is equal to the strain capacity of the matrix. The strain capacity grows inversely proportional to the particle size for particles smaller than 10 mm. For this given system, the critical particle size is 10 mm.

Bazant et al. (2000) studied the effect of particle size of waste glass. They found that for large particles, a decrease in particle size caused a larger expansion. Their intuitive explanation of this was that given smaller particle sizes, a larger volume fraction of glass undergoes AAR due to the increased surface area per volume unit of glass. This causes larger pressures. However, they found a pessimum size of approximately 1.5 mm. Below this limit, the expansions were reduced, and below 0.15 mm the expansions was the same as for specimens with no glass. The stress intensity factor, or the energy release rate, is reduced with decreasing particle sizes. This causes less cracking.

Golterman (1995) also used fracture mechanics to explain why cracks were not formed from aggregates under a certain limit in the cases of expanding particles, or shrinking paste. According to fracture mechanics, cracks propagate if the released energy is larger than the fracture energy needed to propagate the crack. Further, according to Golterman (1995), the released energy for a swelling particle of radius R is proportional to R^3, while the necessary fracture energy is proportional R^2. Consequently, for a given system, there will be a critical particle size where no crack propagation will be caused by particles under this limit.

3.7 Size of test specimens

It is a fact that the size of the test specimens influences the test results in the way that it takes a longer time for the reaction to occur when the test specimens are enlarged.

Another factor of which the test results could be influenced by the size of the test specimens, is when only a few % of the fraction (either sand or stone) is reactive. In this case the representation of the reactive grains increases by increasing size of the test specimen and the deviation of the test results is expected to increase by decreasing size of test specimen.

Both in the European STAR-project and in a study carried out by Jensen & Fournier (2000) ultra accelerated mortar bar tests corresponding to the RILEM AAR-2 method were performed with different mortar bar sizes. The tests were carried out with prisms size 40×40×160 mm, respectively 25×25×250 mm. The correlation between the measured expansions with the two prism sizes is shown in Figure 3.7-1. Based upon these results it is claimed that the expansion measured for the thinner test specimens (ASTM) is found to be approximately twice the expansion of the thicker test specimens.

However, due to a lack of explanation of the various testing conditions in these results, a memo was compiled by Wigum (2004). Based on collected information, the original results (expansions) were recalculated, i.e. adjusted in relation to the “effective length” (i.e. the distance between the inner end of the studs) and for differences in w/c ratio (0,45 – 0,50). The memo presented the adjusted ratio of expansion results after 14 days for short bars (25x25x250 mm) versus long bars (40x40x160 mm) as shown in Figure 3.7-2 and Figure 3.7-3. It is evident that the ratio between expansions revealed by use of “short thick/long thin” prisms is low (0.35) for slowly reactive types of aggregate (eg. Phyllite), while more rapid and highly reactive types of aggregates approach a ratio close to 1 after 14 days of testing. After 28 days of testing the ratio has shown to be close to 1 for all types of aggregates, which also is recognized by Jensen and Fournier (2000).

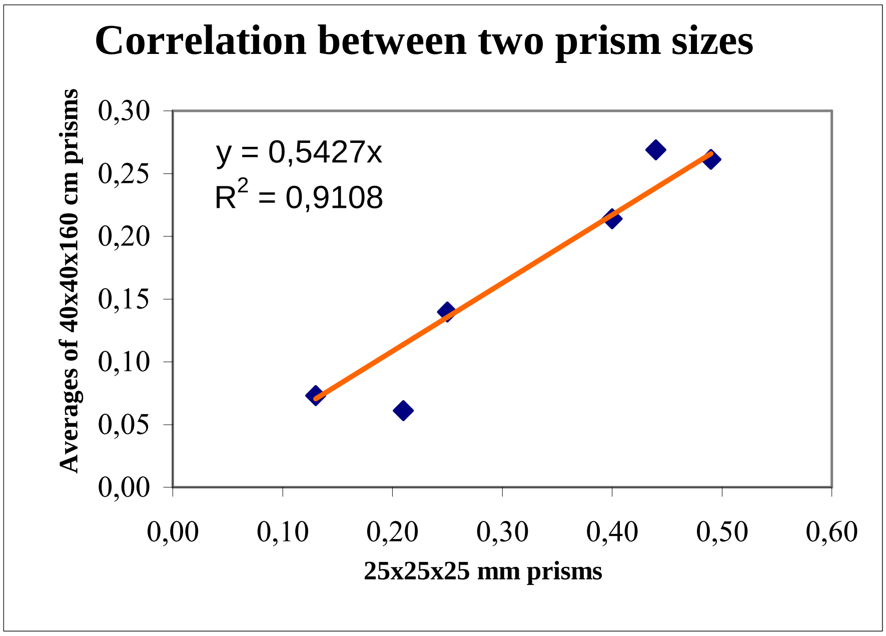

Figure 3.7-1: Correlation between prism sizes RILEM (40×40×160 mm) and ASTM (25×25×250 mm) (The STAR Partners, 1998).

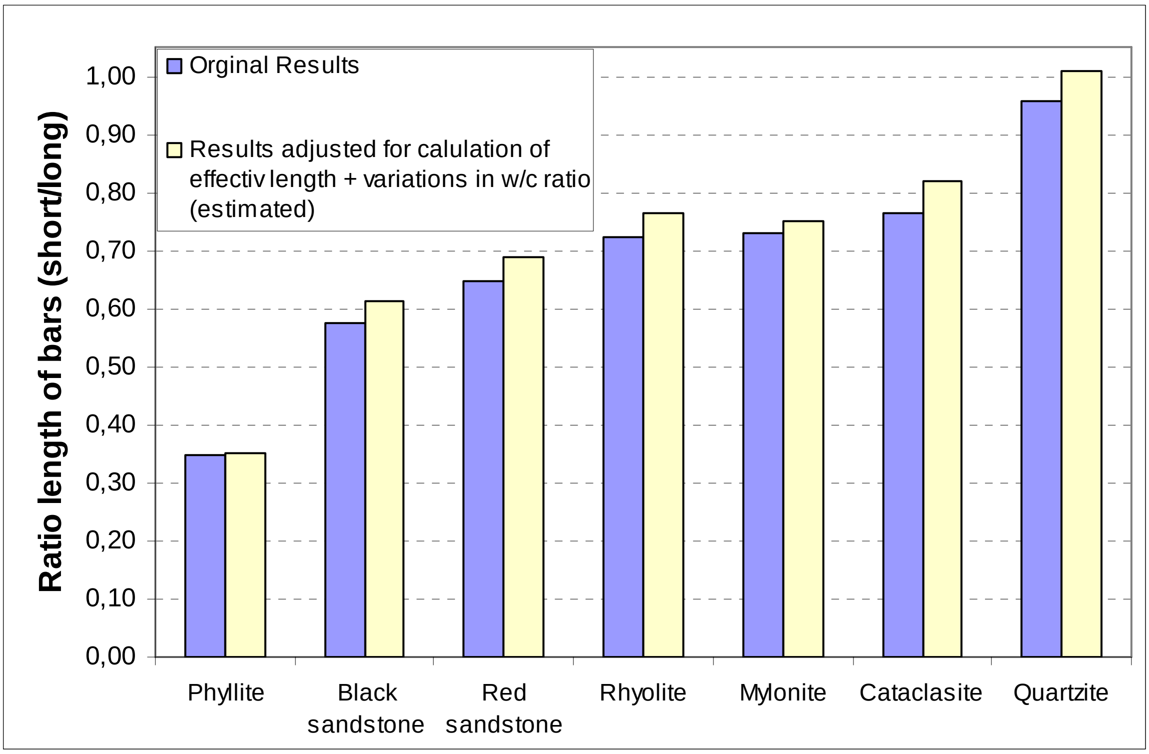

Figure 3.7-2 Ratio of expansion results after 14 days for short bars (25x25x250 mm) versus long bars (40x40x160 mm). Original results from Jensen and Fournier (2000), where adjusted results and graph are from Wigum (2004).

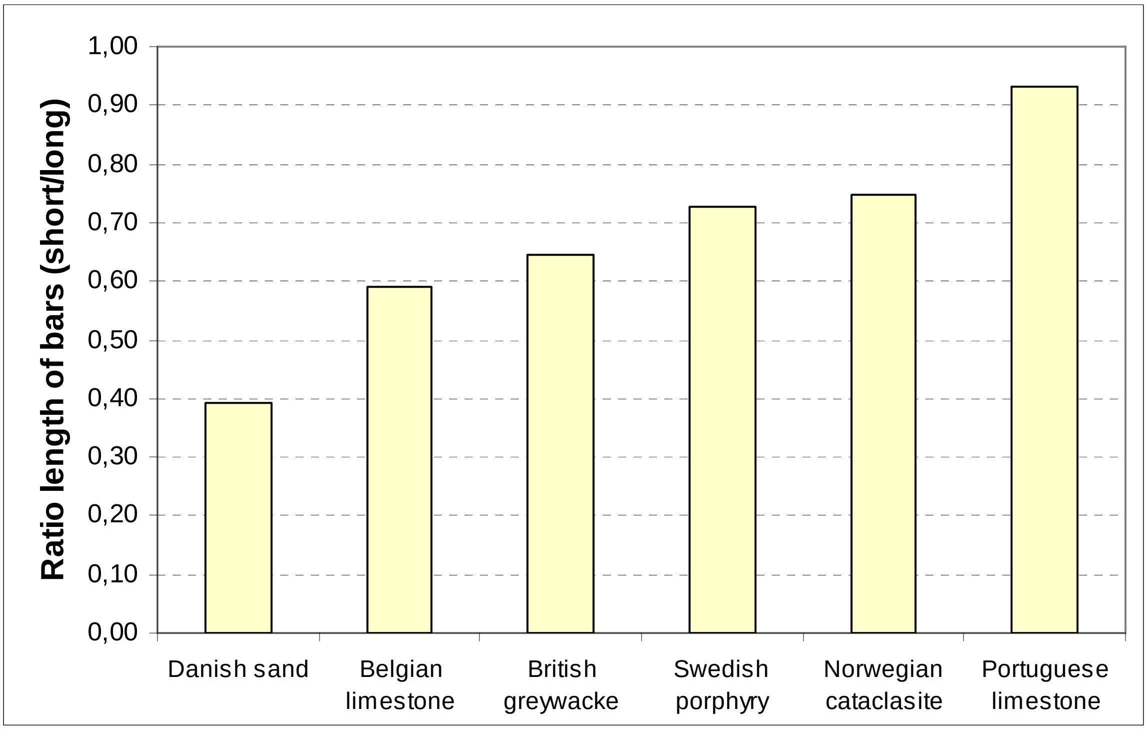

Figure 3.7-3 Ratio of expansion results after 14 days for short bars (25x25x250 mm) versus long bars (40x40x160 mm). Original results from the STAR-project (1998), where presented adjusted results are from Wigum (2004).

In a study carried out by Wigum et al. (1997) a non-reactive sand was adjusted by replacing some of the original sand with fused silica so that the amount of fused silica in total made 15% of the aggregate. Sufficient material was prepared to make more than 100 mortar bars both in the sizes

25x25x250 mm and 40x40x160mm, and accelerated mortar bar testing was carried out at two independent laboratories. A thorough statistical analysis of the expansion results for different mortar bar sizes did not show any significant differences in expansion results after 14 days of exposure between the two different mortar bar sizes.

Suzuki et al. (2004) found that the expansions of mortar bars by ASTM C 1260 were influenced by the size of the specimens. For a total of six different aggregates tested, the smaller cross-sectional bars (25x25x285 mm) showed 0,9-3,1 times higher expansion after 14 days when compared with the shorter and thicker bars (40x40x160mm). When the tests completed, the alkali ion content in the different mortar bar sizes was measured. The small-sectional bars contained higher alkali amount than the thicker bars. It is considered that the alkali ions become more easily permeated into the interior of the thinner bars.

Duchesne and Bérubé (2003) carried out a study where mortar bars and concrete prisms made with a very alkali–silica reactive limestone were stored at 38^{o} C in 1 M NaOH and NaCl solutions. Chemical equilibrium between the pore water and the immersion solution was much longer to obtain for the concrete prisms (near 3 years) than for the mortar bars (less than 3 months). The results obtained in this study show that the type of samples used (mortar bars or concrete prisms) strongly influence the measured expansion due to AAR.

The effect of the specimen size on threshold alkali content required to initiate expansion with a particular aggregate has been discussed by Thomas et al. (2006). Comparison was made between mortar bars, concrete prisms and concrete blocks (stored outside). It is evident that both mortar bars and concrete prisms reach an ultimate expansion after a certain period of time, while the expansions of the blocks continue to expand. They claim that this effect can largely be ascribed to leaching of the alkalies from smaller specimens (see chapter 3.9).

The effect of the prism size in the mortar bar test is also investigated in the PARTNER project – cf. report no. 3.2.

3.8 Air entrainment, paste porosity and water/cement ratio

3.8.1 Laboratory experiences

It is well known that a concrete, which is used in countries where the possibility of freeze-thaw cycles exists, has to have a certain amount of well-distributed air voids to be frost resistant. In this way there is room for growth in the volume of water contained in the concrete when it changes from the liquid phase of water to the solid phase of ice. The same principle can be used concerning the influence of the air content as regard to damage due to AAR. The alkali-aggregate reactions will exist regardless of the air content in the concrete, but the degree of damage is influenced by the air content. A certain amount of well-distributed air voids in the concrete makes room for the AAR-gel and thereby reduces the expansions. When the air voids are filled up to a certain level with AAR-gel, the expansion will increase. In a similar way the conclusion can be made that an air entrained concrete with AAR is likely to be less frost resistant when the voids are filled with gel (Hobbs, 1988).

The rate of reaction is independent of the air content, but as mentioned above the extent of damage is reduced. An illustration of the influence of the air content on the expansion is given in Figure 3.8-1 (Hobbs, 1988). The tests with mortar bars immersed in a salt solution at 50^{o}C carried out by Chatterji et al. (1984) show similar results. These test results show a reduction of the expansion by approximately 40% by introducing 4% air in the mortar (Jensen et al., 1984).

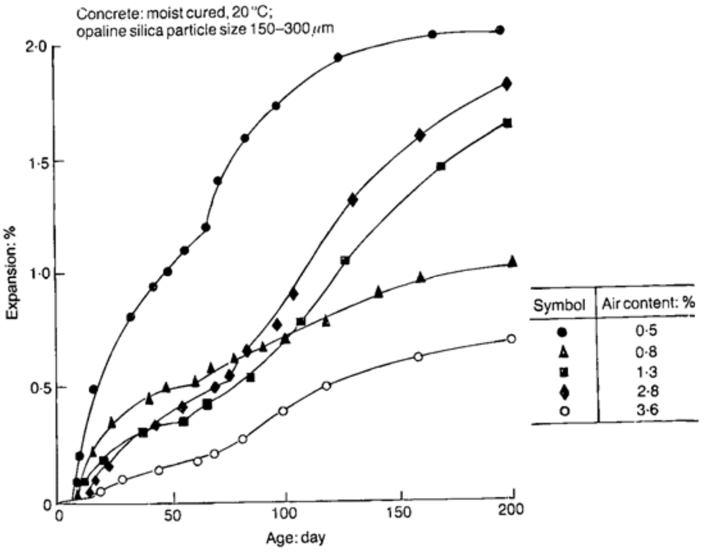

Figure 3.8-1: Influence of air content in hardened concrete on the relationship between expansion and age. w/c and aggregate/cement ratios were 0.41 and 3, respectively, sodium oxide 5.0 kg/m^3 (uncrushed aggregate) (Hobbs, 1988).

The paste porosity has an effect on the expansion similar to that of the air content. The more porous paste the more room for the AAR-gel in the paste and the less expansion will occur. In this case, the rate and extent of reaction will, however, increase by increasing porosity, as the transportation of ions to the reactive aggregates increases.

Decreasing the porosity leads to an increase of the expansion, as the room for the gel and the swelling of the reactive aggregate decreases. However, by decreasing the porosity, the reaction rate decreases as it takes a longer time for water and $Ca(OH)_2$ to penetrate through the paste to the reactive grains. In the 1970’s Thaulow and Idorn experienced expansion in concrete with w/c = 0.2. It took some time for the reactions to go on, but the expansion was very severe (unpublished data from Concrete Research Laboratory, Karlstrup, Denmark). Tests carried out by Collins & Bareham show a similar effect of the water-cement ratio with the expansion being 0.19% at a w/c-ratio of 0,325 respectively 0,10% at a w/c-ratio of 0,40 (Collins & Bareham, 1987). However the opposite affect is concluded by Shon et al. (2002) during experiment according to modified ASTM C1260 test methods. It has to be noted though, that the results given in Shon et al. (2002) only covers 28 days.

Changing the w/c ratio in mortar bars affects not only the alkali concentration of the pore solution, but also the physical properties of the mortar, in particular the paste porosity. These changes effect the expansion of mortar bars made with alkali reactive aggregates. It is suggested by Grattan-Bellew (1989) that the optimum w/c ratio for expansion of mortar bars containing alkali-reactive aggregates is probably in the range of 0.4 to 0.6. The exact value depends on the physical and chemical properties of the aggregates. Several researchers have observed that the expansion of mortar bars generally increase with increasing water/cement ratio (Davies and Oberholster 1987). Fournier and Bèrubè (1991) made mortar bars with different w/c-ratios and reported that the expansion generally increased with increasing w/c, but decreased when the w/c ratio exceeded 0.50. For this reason it is recommended by Bèrubè et al. (1992) to use a fixed w/c ratio of 0.50 for coarse aggregates and manufactured sands, and 0.44 for uncrushed sands. These criteria are now incorporated into the Canadian standard (CSA 1994) and the ASTM standard (ASTM 1994).

Wigum (1995) investigated the effect of different wc/ratios (w/c 0.45 and 0.50) in mortar bars made with innocuous aggregates. The results in Figure 3.8-2 showed that an increased w/c ratio exhibited an increased expansion with time.

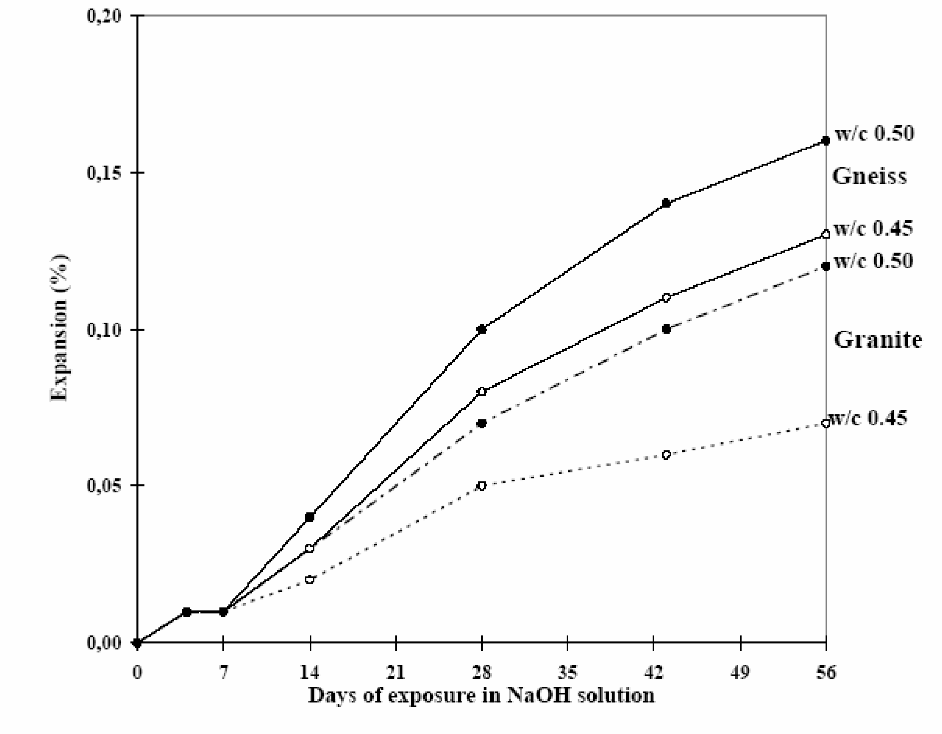

Figure 3.8-2 Effect of different water-cement ratios for the accelerated mortar bar expansion for the innocuous rock types gneiss and granite. From Wigum (1995).

Several authors have also reported that use of light weight aggregates in the concrete in combination with a reactive normal density aggregate suppress the expansion. The reason is that some of the gel produced may flow to the open porosity in the light weight aggregates instead of building up a pressure in the concrete.

As described above the air content, the porosity and the water/cement ratio of the concrete affects the expansion caused by AAR. The influence of a possible change in these parameters is important to bear in mind when testing the alkali reactivity of aggregates according to different testing methods. If for instance the aggregate to be tested has a high water demand and a certain workability of the concrete or mortar is aimed in a specific test method, one should reconsider before adding an air-entraining agent / plasticizer to the mix. The same caution must be taken before adding water leading to a change in the water/cement ratio of the concrete mix. It is also essential to cast and compress the concrete properly to avoid unwanted pores in the prisms.

3.8.2 Experiences from real structures

A concrete retaining wall in Norway was studied by Hagelia (2004). The wall exhibited map cracking in some segments, whilst other segments showed almost no surface cracking. Examination of cores from various segments revealed almost identical water saturation, and a small difference in the content of reactive rock types and cement (alkalis) in the concrete. The main difference between the two concretes was represented by the internal distribution of initial air voids, cement and aggregates. Map cracked concrete was characterised by few and irregularly distributed rather large air voids and a high cement content, notably within the interior. The concrete with intact surfaces was rich in well distributed small air voids displaying a homogeneous cement distribution. The author assumed that the difference in air content between different segments of the wall was the main parameter influencing the extent of damage due to AAR.

3.9 Storage conditions – leaching

If a moisture climate, i.e. Relative Humidity (RH) of 100%, is achieved in a laboratory testing method by having free water in the test container, there is a risk that some of the alkalis may be leached out from the concrete and thereby change the test conditions. If this is the case, it is very important for the test result whether the alkali content is constant or whether alkalis are continuously supplied to the test specimens. Another way (and a more consistent way) to achieve a humidity of 100 % RH is to use a damped cloth to wrap around the test specimen. In this case the risk of leaching of alkalis should be reduced.

However, Blanks and Meissner (1946) reported leaching from mortar bars stored over water in sealed specimens. A build up of alkali ions in the water at the bottom of the containers was detected. They explained their finding to be caused by water condensing on the surface of the bars, running down the bars into the reservoir below, providing transports of the alkalis.

Rivarda et al. (2003) showed that significant leaching occurred when performing accelerated expansion tests in the high moisture environment that prevails in the Canadian CSA A23.2-14 or American ASTM C1293 concrete prism tests, even though the test prisms were covered with plastic sleeves. After 52 weeks in high moisture storage, concrete prisms lost from 12% to 25% of their initial Na_2O_e content. The mass balance suggests that about 23% to 39% of the original alkalis in the cement were quickly adsorbed on the aggregate surfaces or rapidly migrated inside the aggregate particles.

Thomas et al. (2006) claim that the impact of leaching will be less for larger concrete prisms, but will still be significant. It was estimated that approximately 35% of the alkalis originally in the concrete find their way into the water reservoir after 1 year, and as much as 20% after just 90 days. As a consequence they claim that the need to boost the alkali content in concrete prisms means that the concrete prism method is not an effective means for determining the threshold alkali level necessary to produce deleterious expansion with a particular aggregate, nor is it suitable to evaluate the ASR potential for a specific combination of materials (i.e. a “job mixture”).

3.10 Summary of key parameters

The ultimate challenge when testing for AAR in a laboratory is to provide quick, reliable results regarding the reactivity of certain types of aggregate, or even more important assessment of specific concrete job mixes (i.e. performance testing). The results are required to mirror the durability behaviour in real structures designed for life time for up to 100 years.

As discussed in the previous chapters, many parameters will influence the alkali aggregate reactivity. Some of the parameters will only influence the reactivity in the laboratory, while others will have an overall contribution, both in the laboratory and in real structures. The following key parameters are discussed in the report in relation to AAR:

- Temperature
- Humidity, moisture and degree of saturation
- Content of alkalis
- Role of calcium hydroxide (CH)
- Types and content of reactive rock types
- Aggregate particle size / grading
- Size of test prisms
- Air entrainment, paste porosity and water/cement ratio
- Storage conditions - leaching

The authors have not made any effort of ranking the influence of the different parameters, because the influence of any changes in a parameter may vary a lot dependent of the situation, both when performing a laboratory test and in a real concrete structure. However, the experience has shown that in particular any variations in the humidity and/or the alkali content (due to leaching) in the test specimens lead to incorrect results. It is also very important to bear in mind the influence of the different parameters when performing a performance test that should reflect how a given concrete mix will behave in a real concrete structure for a long service life.

Evaluation of any structural effects (i.e. effects depending on structure type, dimensions, reinforcement, etc.) is not included in the report.

4 Test methods used across Europe today

Different test methods are used across Europe today, and within the last couple of years a new set of standardized RILEM methods has been established based on the methods used today and the state-of-the-art knowledge about AAR. Table 4.1 shows a list of the common test methods for alkali reactivity used across Europe today.

A laboratory test method has two, somewhat contradictory, objectives: (A) to ascertain alkali-aggregate reactivity of an aggregate in a reasonable short time and (B) to evaluate and set acceptable expansion limits, due to the alkali-aggregate reaction, of a reactive aggregate-cement combination (i.e. a performance test). This combination could be a specified sand-cement mortar or a specified concrete mix. In most national specifications these two objectives are combined in a single test method. The acceptance criterion is often subjectively chosen.

A. Like most chemical reactions, the rate of alkali-aggregate reaction is higher at higher temperature. It is therefore easy to detect alkali-aggregate reaction early at a higher temperature than at a lower temperature. However, long-term expansion due to alkali-aggregate reaction is higher at a lower temperature than at a higher temperature. Usually, in a laboratory experiment, the alkali-aggregate reactivity of an aggregate is evaluated indirectly measuring expansion of a specified cement-aggregate combination and specified alkali content. It is therefore necessary to make a compromise. The compromise accepted by ASTM is 37^{o}C. Some European countries have approved a similar temperature. This testing at evaluated temperature involves a risk that some very reactive and some slowly reactive aggregate may go undetected. One has to remember that 37^{o}C is not the most common temperature in Europe. However, in exposed concrete surfaces, especially facing south, the temperature can easily reach about 50^{o}C or more.

   In a few test methods, the alkali-aggregate reactivity of an aggregate is evaluated directly. In those methods, elevated temperature is often used. If the alkali hydroxide concentration is kept low in these tests, then the risk of false negative is low.

B. The objective (B) is best evaluated at the expected average temperature to which the concrete is to be exposed. However, in northern countries this may take unacceptable long time. Again, a compromise is called for. In this case, the acceptance criterion can be selected rationally if long term field experience lacks.

Table 4.1: List of test methods for alkali reactivity used across Europe today.

1) If a crushed aggregate is classified as potential reactive, tests with concrete prisms are recommended to be carried out to verify the reactivity (e.g. RILEM AAR-3).
2) The length of the prisms may vary from 250 mm to 300 mm.
3) If the alternative prism size is used – the expansion has to be adjusted.
4) If used in a severe aggressive environment: Reactive if expansion > 0.01% after 20 weeks.
5) The test period may be extended.
6) 104 weeks if pozzolanes are present.
7) Similar to RILEM AAR-2

Name	**Type of test**	**Testing**	**Type of particles**	**Size of sample or specimen [mm]**	**Temp (°C)**	**Alkali content [% Na_2O_{eqv}] L = Limited UL = Unlimited**	**Humidity**	**Criteria**	**Time**
RILEM AAR-1	Petrographic	Aggregate	Fine and coarse	-	-	-	-	The aggregate is classified as: Class I: Very unlikely to be alkali-reactive Class II: Alkali-reactivity uncertain Class III: Very likely to be alkali-reactive	3 hours
Chatterji	Chemical	Aggregate	Fine or crushed coarse	100 g	70	-	-	No in-built criteria – the method has to be calibrated against observed expansions	2 days
RILEM AAR-2	Mortar bar	Expansion	Fine (125 µm < x < 4 mm) – natural **or** crushed 1)	3 beams of 285×25×25 2) or 160×40×40 3)	80 ± 2	UL: immersed in 1M NaOH solution	Immersed in NaOH solution	Varying in the range of 0.08 – 0.20%	14 days
TI-B 51	Mortar bar	Expansion	Fine or crushed coarse	160×40×40	50 ± 2	UL: immersed in NaCl solution	Immersed in NaCl solution	For Danish aggregates: Reactive if expansion > 0.01% after 8 weeks 4)	20 weeks 5)
RILEM AAR-3	Concrete prism	Expansion	Coarse or fine	3 beams of (250 ± 50) × (75 ± 5) × (75 ± 5)	38 ± 2	0.9-1.2 L: (+ NaOH ~ 1.25%)	100%	Reactive if: Expansion rates >0.05%	52 weeks
RILEM AAR-4	Concrete prism	Expansion	Coarse or fine	3 beams of (250 ± 50) × (75 ± 5) × (75 ± 5)	60 ± 2	0.9-1.2 L: (+ NaOH ~ 1.25%)	As close to 100% as possible (reactor)	Reactive if: Expansion rates >0.03%	20 weeks
RILEM AAR-4 alternative	Concrete prism	Expansion	Coarse or fine	3 beams of (250 ± 50) × (75 ± 5) × (75 ± 5)	60 ± 2	0.9-1.2 L: (+ NaOH ~ 1.25%)	Wrapped in damp cotton cloth and further double wrapped in polythene.	Reactive if: Expansion rates >0.03%	20 weeks
Norwegian	Concrete prism	Expansion	Coarse or fine	3 beams of 100 × 100 × 450	38 ± 2	1.0 ± 0.2 L: (+ NaOH ~ 1.25%)	100%	Reactive if: Expansion >0.04% after 12 months without pozzolanes, >0.03% after 12 months or >0.04% after 24 months with pozzolanes	52 weeks 6)

Name	Type of test	Testing	Type of particles	Size of sample or specimen [mm]	Temp (ºC)	Alkali content [% Na_2O_{eqv}] L = Limited UL = Unlimited	Humidity	Criteria	Time
German	Concrete prism	Expansion	Coarse or fine	3 beams of 100 × 100 × 500 and one 300-mm-cube	40 ± 2	L: 1.3 ± 0.1	99% fog chamber	Non reactive if: expansions <0.6mm/m (‰) after 9 months	9 months
Canadian (ASTM)	Concrete prism	Expansion	Coarse or fine	275-405 × 75 × 75	38 ± 2	0.9 ± 0.1 L: (+ NaOH ~ 1.25%)	98%	Potential reactive if: Expansions >0.04% after 12 months	≥ 52 weeks
South African[7)]	Mortar bar	Expansion	Fine or crushed coarse	160 × 40 × 40	80	UL: immersed in 1M NaOH solution	Immersed in 1M NaOH solution	Innocuous if expansions after 14 days ≤ 0.1%, slowly expansive if 0.1% ≤ expansion after 14 days < 0.25% and rapidly expansive if expansion ≥ 0.25% after 14 days.	14 days
NFP 18-587									
TI-B52	Petrographic	Aggregate	Fine and coarse	-	-	-	-		3 hours
Chinese Method	Mortar bar	Expansion	Fine or crushed coarse	10 × 10 × 40					

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Appendix A: National standards and requirements

In this appendix, an overview is presented of various European national standards and requirements regarding Alkali Aggregate Reactivity.

A previous overview was carried out in the European project: STAR (Standard Test for Alkali-reactive Rocks), and are described in the report *“Star-project – final report, European Commission, DG XII, contract nr. SMT4-CT96-2128, 1995”*.

In the following, the descriptions from the above mentioned project is updated and supplemented by descriptions for additional countries. The chapters are compiled or updated and reviewed by representatives from the various countries.

1 Austria

The following Chapter has been reviewed by Stefan Krispel, VÖZ - Association of the Austrian Cement Industry.

1.1 Introduction

A set of studies and experiences on buildings and construction elements during the past years has shown that the potential for a possible damage caused by Alkali-Aggregate Reaction (AAR) depends on the local determining factors. Therefore national guidelines ensure that using local available raw materials, e. g. aggregates or cements, and with the observance of acknowledged rules of technology and methods of execution future damages on concrete structures could be prevented. Due to a systematic approach both owners and engineers could be supported what measures have to be taken to prevent harmful alkali-reactions in the concrete. Besides concrete technological and structural measures the methods of execution and maintenance are important to be looked at.

As a result of a possible presence of alkali-Silica reactivity in Austrian aggregate deposits a standard was necessary to minimize the residual risk and to prevent possible damages. Due to the geological variety of the Austrian aggregates it is not predictable which type of aggregate would cause AAR.

This standard (ÖNORM B 3100 [1]) is particularly adapted to the Austrian aggregates which could possibly contain a small amount of reactive grains.

1.2 Assessment

All so far known cases of AAR in Austrian concrete structures are individual cases. Regularly for concrete production used types of aggregates – especially for level of exposure 1 and 2 according to Table 1.2-1– are not affected. The question of alkali reactivity of aggregates exists therefore in general only for level of exposure 3 according to Table 1.2-1 and for new aggregates that have not been used for concrete production i.e. which have no long-term experiences.

Satisfactory long-term operating experiences are a satisfactory good base for conclusions than testing methods which only can simulate the exposure. Tests of aggregates according to their alkali-reactivity are an aggravated simulation of the actual existing conditions and the results may – if a negative result is taken into account – not correlate with the practical behaviour of this type of aggregate. This consideration is the reason why in the Austrian standard ÖNORM B 3100 a negative test result of an aggregate has not to be used for assessment if this type of aggregate shows positive long-term operating experiences.

If there are no positive long-term operating experiences for an aggregate and there exists also a negative test result it is not allowed to use this type of aggregate for constructions which are – according to Table 1.2-1 – classified in level of exposure 3. These materials can be used for structures which are classified in level of exposure 2 but additional measures according to the recommendations of an appropriate expert have to be considered

1.2.1 Level of exposure

For the assessment of the Alkali-Aggregate Reaction in the concrete the produced construction elements will be classified in levels of exposure according to Table 1.2-1.

Table 1.2-1: Levels of exposure

Level of exposure		Building component	Environmental conditions [1]
1	Low	Inside buildings Temporary building components (also level of exposure 2 and 3)	- dry to moderate moisture penetration - design life: max. 15 years
2	Moderate	All building components with the exception of building components of level of exposure 1 and 3) e.g. load-bearing structures of bridges which are not directly passable by vehicles	- exposed to weather conditions - moderate to highly moisture penetration - surface temperature: always < 20 °C
3	High	Concrete pavements (sub concrete and concrete topping)	- external alkali supply (e.g. de-icing agents) - moderate to highly moisture penetration - surface temperature: alternating and maximum temperature > 25 °C - dynamic loading

1) In each case one of the informative stated environmental conditions has to be met to divide building components in the levels of exposure.

According to Table 1.2-1 all concrete components and the methods of execution have to be assessed for the stated levels of exposure.

Therefore an assessment of

1. the aggregate
2. the cement
3. the admixtures
4. the additives
5. design and execution of the building components

are necessary for the particular level of exposure.

1.3 Assessment of the aggregate for the particular level of exposure

In the below-mentioned Flow-Chart is the assessment of aggregates described for the level of exposure 2 and 3. For level of exposure 1 the declaration “NPD” is acceptable.

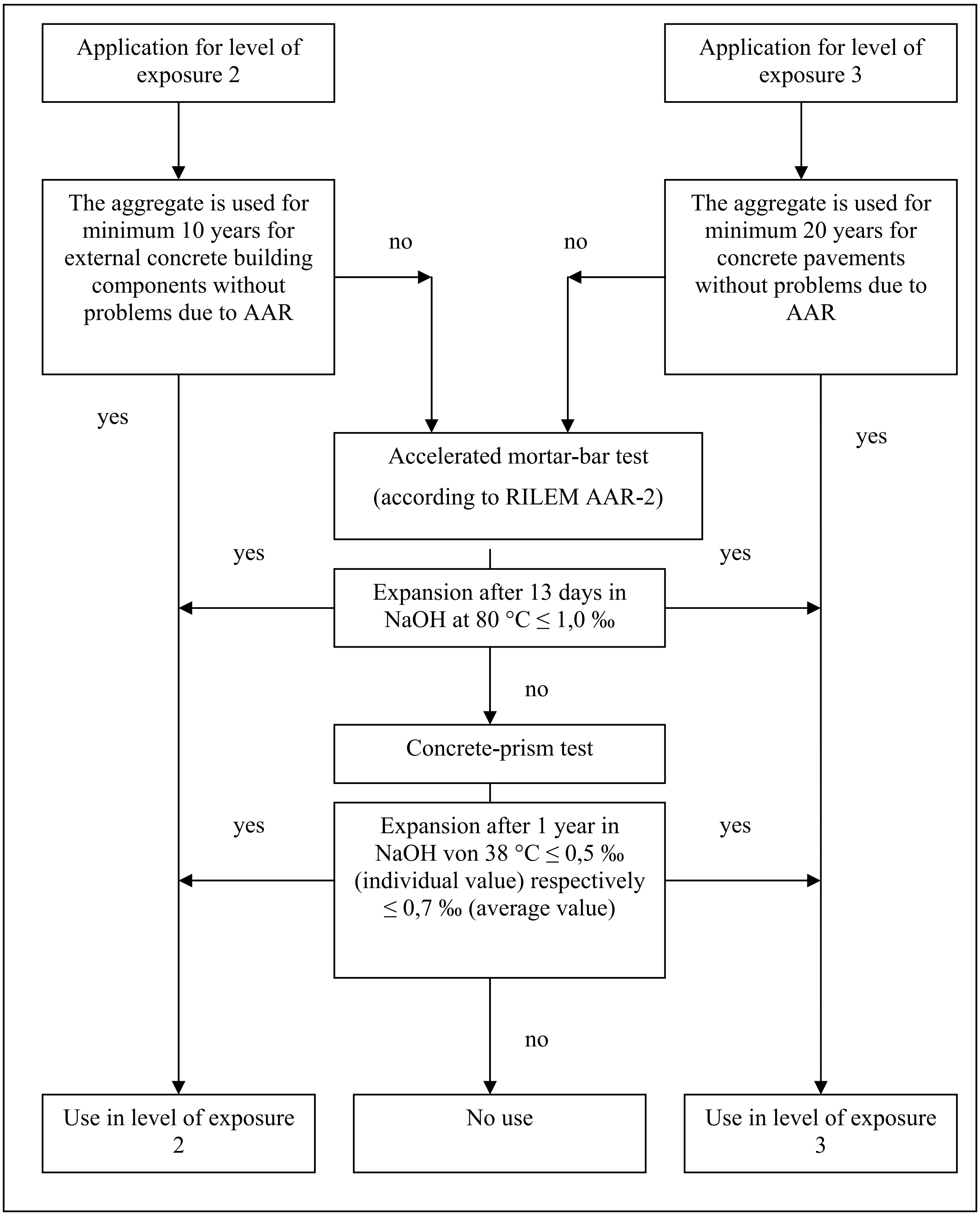

Figure 1.3-1 Flow Chart: Assessment of aggregates (level of exposure 2 and 3)

Above mentioned test methods correspond to below described measuring principles:

Accelerated mortar-bar test

Prisms (40 mm x 40 mm x 160 mm) are produced with the testing aggregate and CEM I "unit cement" (mixture of all Austrian CEM I cements). Reference studs are placed at the end faces. After 24 hours the prisms will be heated up in water from 20 °C to 80 °C and afterwards stored for 13 days in NaOH-solution at 80 °C. The expansion which occurs in the NaOH-solution will be used for assessment of the aggregate. This measuring principle is in accordance with the RILEM AAR-2 [2] test method.

Concrete-prism test

Prisms (10 cm x 10 cm x 40±4 cm) are produced with the testing aggregate and CEM I "unit cement" (mixture of all Austrian CEM I cements). Reference studs are placed at the end faces. After 24 hours the prisms will be stored for 6 days at 20 °C and min. 90 % r. H. and afterwards for 51 weeks in NaOH-solution at 38 °C. The expansion which occurs in the NaOH-solution will be used for assessment of the aggregate. The storage in NaOH-solution simulates the uptake of alkalis coming from the winter service (de-icing agents) on concrete pavements and produces significant higher expansions compared to the test results according to RILEM AAR-3 [4].

1.4 Assessment of the cement for the particular level of exposure

Level of exposure 1 and 2

All cements according to ÖNORM B 4710-1 [5] are appropriate.

Level of exposure 3

Cements have to meet the additional requirement "reduced expansion" according to ÖNORM B 3327-1:2005 Table A.1 [6] (tested with an aggregate which contains reactive grains).

1.5 Assessment of the admixtures for the particular level of exposure

Level of exposure 1 and 2

All admixtures which are approved for concrete production according to ÖNORM B 4710-1 are appropriate.

Level of exposure 3

Only admixtures according to ÖNORM EN 934-2 [7] with a declared alkali content < 1 % are allowed.

1.6 Assessment of the additives for the particular level of exposure

Level of exposure 1 and 2

All additives which are approved for concrete production according to ÖNORM B 4710-1 are appropriate.

Level of exposure 3

Only additives according to ÖNORM B 3309 [8] are allowed.

1.7 Assessment of design and execution of the building components for the particular level of exposure

Level of exposure 1 and 2

No additional design and execution measures are necessary.

Level of exposure 3

A penetration of water into the structure has to be prevented and if necessary drainage measures have to be designed.

1.8 References

[1] ÖNORM B 3100, Ausgabe 2006: Beurteilung der Alkali-Reaktion im Beton. Österreichisches Normungsinstitut, Wien.

[2] RILEM Recommendation TC 106-2: Method for detection of the potential alkali-reactivity of aggregates by ultra-accelerated mortar bar test, published in [3].

[3] RILEM TC 106 AAR – Alkali-Aggregate Reaction: International Assessment of Aggregates for Alkali-Aggregate Reactivity. Materials and Structures, Vol. 33, Nr. 226, S. 88-93, March 2000.

[4] RILEM-Recommendation TC 106-3: Detection of Potential Alkali-Reactivity of Aggregates – Method for Aggregate Combinations using Concrete Prisms. Materials and Structures, Vol. 33, S. 290-293, June 2000.

[5] ÖNORM B 4710-1:2004 – Beton Teil1: Festlegung, Herstellung, Verwendung und Konformitätsnachweis (Regeln zur Umsetzung der ÖNORM EN 206-1), Österreichisches Normungsinstitut, Wien 01.04.2004.

[6] ÖNORM B 3327-1, Ausgabe 2005: Zemente gemäß ÖNORM EN 197-1 für besondere Verwendung. Teil 1: Zusätzliche Anforderungen. Österreichisches Normungsinstitut, Wien.

[7] ÖNORM EN 934-2, Ausgabe 2006: Zusatzmittel für Beton, Mörtel und Einpressmörtel. Teil 2: Betonzusatzmittel – Definitionen, Anforderungen, Konformität, Kennzeichnung und Beschriftung. Österreichisches Normungsinstitut, Wien.

[8] ÖNORM B 3309, Ausgabe 2004: Aufbereitete hydraulisch wirksame Zusatzstoffe für die Betonherstellung (AHWZ). Österreichisches Normungsinstitut, Wien.

2 Belgium

The following Chapter has been reviewed by Gabriel Lorenzi, ISSeP - Institut Scientifique de Service Public.

2.1 Introduction

The ASR is a possible cause of deterioration of concrete structures. The cases recorded in Belgium until 2006 are numerous enough (more than 100 cases), to justify precautions. The official recommendations are significantly different as far as the Flemish or Waloon region is concerned.

Nevertheless on a general level, four approaches are considered as being effective in reducing AAR risk.

- To use a low alkali (LA) cement, together with a limit of alkali in concrete
- To use a non reactive aggregate
- If a non LA cement is used: to perform an acceptance test (*) on the proposed concrete mix design ((*) equivalent to AAR-2 but applied to concrete sample, AAR-4 or Danish method
- To use mineral admixture (fly ash or silica fume) together with the performing of an acceptance test (see (*)) on the so-designed concrete

2.2 Specifications against ASR

The Belgian concrete standard (NBN B 15-001(2204) (1)), which is the national adaptation of ENV 206, gives the following guidance:

Some aggregates containing particular varieties of silica are susceptible to attack by alkalis originating in the cement or from other sources. In humid conditions this can produce an expansive reaction leading to cracking or movement of the concrete. In such conditions one or more of the following precautions should be taken:

- Limit the total alkali content of the concrete mix, depending on the type of cement used,
- Use a low alkali cement (according to NBN B 12-109),
- change the aggregates,
- Limit the degree of saturation of the concrete, for example by using an impermeable membrane.

If the route of limiting the total alkali in the concrete is followed the following limits apply:

Table 2.2-1: Applied limits if the total content of alkali in cement is limited.

Category and class of the cement	Alkali limit in the concrete [kg Na_2O eq/m^3]
CEM I	≤ 3
CEM III A	≤ 4.5 (slag 50%)
	≤ 5.5 (slag 50%)
CEM III B and C	≤ 10
CEM V/A	≤ 7.5

These conditions are available only if the concrete is not exposed to other external or internal sources of alkalis. The external source of the alkalis is due to the de-icing salts (mainly NaCl), used every winter on the roads and bridges. The internal sources of alkalis are mainly due to additives or aggregates.

For the time being, the total amount of the alkalis in the concretes ranges from 1.7 kg/m^3 (CEM I cement types) up to 3.2 kg/m^3.

An additional instruction of the MET (Ministry of Equipment and Transportation) of the Walloon Region is to specify that when the « RILEM ASR methods » will be accepted and published, they will be automatically applied.

2.3 Low Alkali Cements

The first precaution against ASR damage is the use of low alkali cements according to the Belgian standard, NBN B 12-109 (2), which covers both Portland (CEM I) blast furnace slag (CEM III) and composite (CEM V) cements and defines the following limits:

Table 2.3-1: Limits defined in the Belgian standard, NBN B 12-109.

Category and class of the cement	Alkali limit in cement [Na_2O equivalent % by mass]
CEM I	≤ 0.6
CEM III A	≤ 0.9 (slag 50 %)
	≤ 1.1 (slag 50%)
CEM III B and C	≤ 2.0
CEM V/A	≤ 1.5

The Na_2Oeq amount (in %) of the cements (at the present time) is ranging from 0.2 to 0.8 %. In the past, the amounts of alkalis were higher: sometimes up to 1 % of Na_2Oeq.

2.4 Expansion Tests

When designing a new concrete mix, expansion tests can be carried out to obtain a mix with an expansion lower than a defined limit value, (according to the test used):

- Test NBRI modified (South-African test)
- Test NFP 18-587 modified (French test)
- Test TI – B5I (Danish test).

2.5 Non-reactive Aggregates

The nature and content of the reactive silica in the aggregates determines the reactivity of an aggregate (chert, flint, silicified limestone). The reactivity of an aggregate can be predicted by an expansion test or by a more rapid test: **petrographic analysis (RILEM-ASR1).** In this procedure, the aggregate is examined under a petrographic microscope (using, in some cases, the point counting method).

2.6 Mineral Additions

Mineral additions (fly ash, silica fume) can contribute more or less effectively to the prevention of damage by ASR. The maximum recommended amounts of these additives are as followed:

- Slag : 50 to 60%
- Fly ash: depends on their chemical composition
- Silica fume: 5-7.5%.

2.7 Pessimum behaviour

A pessimum behaviour has been observed with the dredged marine sands and gravels: it seems that the expansions over 0.1 % would be obtained with a content of cherts ranging from about 2% up to 35 % (tested by the Danish method).

2.8 References

1. NBN B 15-001 (2004) Title still to be defined.
2. NBN B 12-109 Cement - Low alkali limited cement. IBN September 1993.
3. MET - Circulaire 42-4-95-001 'Recommendations provisions pour la prevention des reactions alcalis - granulate'. February 1995.

3 Denmark

The following Chapter has been reviewed by Jørn Jensen, PC Laboratoriet A/S.

3.1 Introduction

In Denmark the Danish Committee on Alkali Reaction in Concrete carried out a large research work regarding ASR in the 1960's. Based on a lot of experiments the first suggested preventive measures were the use of low-alkali cement and/or the use of aggregates containing less than 2% reactive component. At this stage the basic assumption was that the alkali content of concrete is determined by that of the cement and the alkalis are evenly distributed /1/.

The second phase of alkali-silica research started with an investigation of concrete roads, which had deteriorated within 4 years of their construction. This investigation showed that extensive ASR had taken place although the coarse aggregate was non-reactive granite and the cement contained about 0.6% Na_2O_{epv}. It was the result of an interaction between the de-icing salt and reactive silica present in the sand. The explicit assumption of this phase of research was that the structures often receive alkalis from outside sources and that there is nearly always a concentration gradient of alkalis in any structure.

3.2 Specifications against ASR

In Denmark all concrete structures are to be considered with respect to the Danish Standards DS/EN206, DS 2426, DS/EN 12620 and DS427 (2000).

The development of ASR and cracking of concrete depends primarily on the following parameters:

1. The environmental condition (temperature, humidity, salt).
2. The alkali content of the concrete.
3. The type and content of the reactive silica in the aggregate.
4. The quality of the concrete (strength, w/c, air content).

The Danish specifications against ASR are given in DS 2426 and DS/EN 12620 and are based upon the above-mentioned parameters in 1, 2 and 3.

3.3 Environment classification

The requirement refers to three environmental classes, characterised by different degrees of aggressiveness commonly found in Denmark:

- Passive environmental class. Comprises dry, non-aggressive environment, i.e. particularly an indoor climate.
- Moderate environmental class. Comprises moist, outdoor and indoor environment and flowing and standing fresh water.
- Aggressive environmental class. Comprises environment containing salt or flue gases, seawater or brackish water and environments where de-icing salt is being used.
- Severely aggressive environmental class. Comprises environment where the humidity is high and large amounts of alkalis or chlorides could be accumulated in the concrete surface – for example swimming pools and bridge columns.

3.4 Aggregate classification

The aggregates are classified in 4 groups:

- Class P for use in passive environment.
- Class M for use in moderate environment.
- Class A for use in aggressive environment.
- Class E for use in severely aggressive environment (in Danish: "**E**kstra aggresiv").

The classification of the sand with regard to alkali silica reactive components is shown in Table 3.4-1. The methods used are the petrographic thin-section point count method TI-B52, the mortar bar expansion test TI-B51, the chemical shrinkage method (TK-84) and the accelerated mortar bar test (ASTM C1260). These 4 methods are described below. The standard distinguish between sand which contains micro porous flint and sand which do not. For the first group the demands for either chemical shrinkage, volume of reactive flint or the mortar bar expansion has to be fulfilled. For the second group the additional demand for accelerated mortar bar expansion has to be fulfilled as well. It the sand material is crushed testing results from another batch is allowed if the geological origin is the same /2/.

Table 3.4-1: Classification of Danish sand /2/.

Classification of sand					
	Class P	Class M	Class A	Class E	Test according to
Chemical shrinkage [mL/kg]	No requirement	Max. 0.3	Max. 0.3	Max. 0.2	DS 405.13
Volume reactive flint [Vol. %]	No requirement	Max. 2	Max. 2	Max. 1%	DS 405.14 (TI B-52)
Mortar bar expansion [%]	No requirement	Max. 0.1 after 8 weeks	Max. 0.1 after 8 weeks	Max. 0.1 after 20 weeks	DS 405.15 (TI B-51)
Accelerated mortar bar expansion [%]	No requirement	Max. 0.2 after 14 days	Max. 0.1 after 14 days	Max. 0.1 after 14 days	DS 405.16 (ASTM C1260)

For the coarse aggregate the amount of reactive aggregate is limited by the allowable amount of particles with a density below 2400 kg/m^3 for the environment classes P, M and A. For the environment class E, the restrictions are based on a density below 2500 kg/m^3. Furthermore, there is a limit on the water absorption of the flint with density larger than 2400 kg/m^3. This value is determined on those 10% of the flint with highest absorption /2/. The standard distinguish between coarse aggregate within the boarders of Denmark (or within the Danish sea territory) and coarse aggregate from abroad. For the first group the demand for the critical absorption has to be fulfilled. For the second group the additional demand for accelerated mortar bar expansion has to be fulfilled as well. It the coarse material is crushed testing results from another batch is allowed if the geological origin is the same /2/.

Table 3.4-2: Classification of Danish coarse aggregates /2/.

Classification of coarse aggregate					
	Class P	Class M	Class A	Class E	Test according to
Particle density below 2400 kg/m^3 [%]	No requirement	Max. 5.0	Max 1.0	-	DS 405.4
Particle density below 2500 kg/m^3 [%]	No requirement			Max. 1.0	DS 405.4
Absorption [%]	No requirement	Max. 2.5	Max. 1.1	Max. 1.1	DS 405.12
Accelerated mortar bar expansion [%]	No requirement	0.2 after 14 days	0.1 after 14 days	0.1 after 14 days	DS 405.16 (ASTM C1260)

3.5 Alkali content in the concrete

The alkali content in the concrete should be calculated as the sum of the acid soluble equivalent Na_2O content in the cement (obtained from cement supplier), the water soluble equivalent Na_2O content in the sand and coarse aggregate (obtained from the aggregate supplier) and the equivalent Na_2O content in the admixtures (obtained from the admixture supplier). The alkali content from fly ash (PFA) and mikrosilica are not included.

3.6 Requirements

The requirements depend on the environmental class and are given in the table below.

Table 3.6-1: Requirements of Danish environmental classes /2/.

Requirements				
Property	Passive Environment	Moderate Environment	Aggressive Environment	Severe aggressive environment
Alkali content in cement [%]	No requirements	≤ 0.8 1)	≤ 0.8 1)	
Alkali content in concrete 2) [kg/m^3]	No requirements	≤ 3.0 1)	≤ 3.0 1)	≤ 3.0 1)
Sand class	No requirements	M 3)	A	E
Coarse aggregate class	No requirements	M	A	E
w/c	No requirements	≤ 0,55	≤ 0,45	≤ 0,40

Notes to Table:

1) Alkali content in the cement may be higher than 0.8% and the alkali content in the concrete may be higher than 3.0 kg/m if the following conditions are fulfilled: Sand: Class A + expansion max. 0.1% after 20 weeks (or max. 1.0% reactive aggregates).
   Coarse aggregate: Max. 1.0% with density below 2500 kg/m^3.
2) The contribution from microsilica and fly ash is not included.
3) Class P sand is allowed if the cement is marked EA or LA and the total alkali content in the concrete is below 1.8 kg/m^3.

3.7 Assessment of aggregate reactivity

The following test methods are used:

1. **The alkali-silica reactivity of sand – TI-B51:**

   Principle: Prisms (40 x 40 x 160 mm) made from mortar, consisting of one portion by weight of cement and three portions by weight of the sand to be tested. The w/c ratio is 0.50. The prisms are water cured for 28 days and then put into a saturated sodium chloride solution at a temperature of 50 ^{o}C.

   The linear expansion of the prisms is measured for 8 weeks, or alternatively for 20 weeks, after they have been put into the saturated sodium chloride solution.

   Use: The method should be used for the relative comparison of the alkali-silica reaction of different sands tested, making it possible to choose the sand that will result in the smallest expansion.

   Test result: Mortar prism expansion.

2. **Petrographic investigation of sand – TI-B 52:**

   Principle: Fluorescent impregnated thin sections made from epoxy encased sand, which is to be tested, are analysed using a polarizing microscope to determine the mineralogy of the individual grains of sand.

   Use: The method is used to determine the mineralogical composition of sand for the specific purpose of determining the quantity of alkali reactive material, i.e. porous flint and opal lime.

   Test Result: Distribution of rock minerals and content of reactive materials.

3. **Accelerated mortar bar expansion test ASTM C1260 (DS405.16).**

   Principle: 40x40x160mm prisms are made according to ASTM C1260 and stored in 1 day at 80° water before being stored for 14 days at 80° 1N NaOH. During this period the expansion is measured and the expansion after 14 days is the result.

   Use: The method should be used for the relative comparison of the alkali-silica reaction of different sands tested, making it possible to choose the sand that will result in the smallest expansion. It is only used on sand that do not contain micro porous flint.

   Test result: Mortar prism expansion.

4. **Chemical shrinkage TK-84.**

   Principle: The method is based upon that reactive flint is dissolved in NaOH. A sample of the sand is subjected to hot NaOH and the shrinkage due to the dissolution of flint is measured.

   Use: The method is only used on sand containing micro porous flint.

   Test result: Dissolution pr kg material.

3.8 References

1. "The Alkali-Silica Reaction in Concretes. Edited by R.N. Swamy 1991, chapter 6 Alkali-silica reaction - Danish experience.
2. Danish Standard DS481, Dansk Standard, 1999.
3. Test Method TI-B51 "Alkali Silica Reactivity of Sand" (in Danish), Technological Institute, Copenhagen.
4. Test Method TI-B52 "Petrographical Investigation of Sand" (in Danish), Technological Institute, Copenhagen.
5. DS/EN 206-1 (05-09-2002): Concrete – Part 1: Specification, performance, production and conformity.
6. DS 2426 (10-05-2004): Concrete – materials – Rules for application of EN206-1 in Denmark
7. DS/EN 12620 (02-04-2004): Aggregates for concrete.

4 France

The following chapter has been reviewed by Bruno Godart, LCPC Laboratory.

4.1 Introduction

Detailed guidance is given by the Ministère de l'Equipement, du Logement, de l'Amenagement du Territoire et des Transports, in its recommendations (1). Reference to this guidance is made in the French National Annex of standard NF EN 206-1 of April 2004.

The first recommendations in this respect were published in France in 1985. They concerned both the alkali-aggregate reaction and sulphate reaction (2). They were issued by the Federation Nationale des Entreprises de Travaux Publics in response to a number of alkali-aggregate reaction problems encountered by French contractors at sites abroad. These recommendations were updated 5 years later (3).

In 1989, the Ministere de l'Equipement, du Logement, de l'Amenagement du Territoire et des Transports, faced with a significant number of structures affected by the alkali-aggregate reaction, decided to take preventive measures in the building of structures for which it is responsible. The Laboratoire Central des Ponts et Chaussees (LCPC) was put in charge of a technical committee bringing together representatives of the various organizations involved in the building process (suppliers of materials, contractors, operators and project supervisors, laboratories, engineering companies, etc.) to draft a document that would be applied as soon as possible. This document was published in January 1991 under the title "Recommandations provisoires pour la prevention des desordres dus a l'alcali-reaction" ("Provisional recommendations for the prevention of damage due to the alkali-aggregate reaction") (14). This document was then transformed in 1994 to an official guidance of LCPC (1).

4.2 Specifications against AAR

The basic principle underlying the specifications given in these recommendations is that the three conditions necessary for the existence of alkali-aggregate reactions are:

- the presence of reactive silica in the aggregates,
- a high alkali content in the pore solution of the concrete,
- a moist environment.

The recommendations define three levels of prevention as a function of both three categories of structures and four classes of environment derived from ENV 206.

Level A:

No special precaution is to be taken with respect to AAR, but special care is given to compliance with both standards and rules of good practice, among others AFNOR document P 18-011 on aggressive environments (4) where relevant.

Level B:

This level corresponds to the most general case, for which there are six equivalent possibilities.

Level C:

This level applies to exceptional structures; the use of non-reactive (NR) aggregates is recommended. But potentially reactive aggregates with pessimum effect (PRP) are allowed provided that the conditions specific to their use are satisfied.

For level B of prevention, the six equivalent possibilities are:

A. Use of non-reactive aggregates (NR)

B. Limit Alkalis in concrete

C. Performance criterion

D. Experience of use

E. Use of mineral additions

F. Use of PRP aggregates with specific conditions

Cf. A: Use of non-reactive aggregates:

The recommendations (1) explain the means conventionally used to classify an aggregate either as non-reactive (NR), as potentially reactive (PR) with respect to AAR, or as potentially reactive with pessimum effect (PRP) with respect to AAR.

The French Standard XP P 18-594 (February 2004) (7) mentions four different methods used to test the reactivity of aggregates to alkalis

These different methods are the following:

1 - Screening tests:

Mortar specimens, autoclave, 0.15 MPa, 127 °C, 3 days (reference method)

Mortar microbar specimen, autoclave, 3 days (alternative method)

Chemical test, NaOH, 80 °C, 4 days (alternative method)

2 - Long term Tests:

Concrete specimen 70x70x282 mm, 38 °C, 100 % HR, container in a reactor, 8 months

The French provisional Standard FD P 18-542 (February 2004) (8) gives the criteria for the qualification of natural aggregates with respect to alkali-aggregate reaction. The main points are:

- First, the identification and the petrographical methods have to be applied:
    - For limestone aggregates, if the chemical analysis shows that $Si0_2 < 4$ %, then the aggregate is NR
    - If the aggregates contain more than 70 % of flints, then the aggregates are PRP
- Second, the reference screening test or one of the two alternative screening tests has to be applied; if the result of the screening test falls within 10 % of the threshold, then it is necessary to confirm the information by doing a long term test.

If there is any controversy between the results of the petrographical analysis, of the chemical analysis or of the screening tests, then the long term test has to be carried out.

A producer of aggregates willing to classify his production with regard to reactivity to AAR shall apply within his quality system the requirements of a document (13) annexed to the recommendation (1).

Cf. B: Limit Alkalis in concrete:

When slag cement is used, and if the slag content of the cement is greater than 60%, only alkalis from the cement are taken into consideration with the following upper limits:

- For CHF cement (CEM III/A and B) (slag between 60 and 80 %): $Na_20_{eq} < 1.1$ %;
- For CLK cement (CEM III/C) (slag above 80 %): $Na_20_{eq} < 2$ %.

When other common cements (5) are used (i.e. CEM I, CEM II, CEM V), an assessment of active alkalis contained in the concrete is made. They may come from the cement, the aggregates, the additions, the admixtures, and the water. Active alkalis are calculated on concrete taking:

For the cement (see also Annex A of Standard NF P 18-454):

- 100 % of the alkali content of clinker and gypsum,
- 50 % of the alkali content of slags and calcareous fillers,
- 17 % of the alkali content of fly ashes and pozzolanas;

For the aggregates:

Actual proportion of alkalis from the aggregates soluble in boiling lime water (the principle of the test method is given in the recommendations; the detailed mode of operation is described in LCPC Method n° 37 (11)).

For the admixtures and mixing water:

- 100 % of the alkali content of admixtures and water.

Two cases are considered according to the knowledge of the variability of alkalis in the cement:

1) When statistical data on the active alkali content of the cement are available, the following rule for limiting alkalis in concrete is applied:

Mean effective value: $T_m < \frac{3.5}{1+2V_c} \frac{kg}{m^3}$

Maximum effective value $T_{max} < 3.5$ kg/m^3, where:

T is the sum of the active alkalis introduced by each constituents of the concrete; V_c is the coefficient of variation of the alkali content of the cement.

2) When the variability of the alkali contents is unknown, the following requirements must be fulfilled:

- $T_m < 3.0$ kg/m^3
- $T_{max} < 3.3$ kg/m^3

Cf. C: Performance criterion:

The performance criterion is based on a direct experimental characterisation of the concrete mix design used for the construction of the structure.

The French standard NF P 18-454 (December 2004) (9) describes the test which is based on the expansion of three 7 × 7 × 28 cm specimens of the actual concrete composition, cured at 60°C and 100 % RH.

The provisional standard FD P 18-456 (10) gives the criteria to interpret the results of the performance test made according to NF P 18-454:

- For concrete made with CEM I and without any additions:
    - With certain types of aggregates, the duration of the test is three months, and the threshold for expansion is 0.02 %
    - With the other types of aggregates, the duration of the test is five months, and the threshold for expansion is 0.02 %
- For all other concretes (including those with additions) :
    - The duration of the test is five months, and the threshold for expansion is 0.02 %
    - The monthly slope of the curve between 3 and 5 months is limited to 0.0025 %
    - If these two last criteria are not fulfilled, then the duration of the test may be delayed to 12 months with a threshold for expansion equal to 0.03 %

Cf. D: Experience of use:

Damage caused by AAR, when it occurs, often has a regional, even local, character. Conversely, some regions are free of it. It may therefore be worthwhile, on the strength of local or regional experience, to establish a framework in which some of the rules mentioned above may be relaxed.

Regional solutions can be used only if:

- The experience of use is for structures at least 10 year-old without any damage due to AAR, this being assessed by a specialised laboratory.
- Data on changes in the sources of cement, aggregates are available.

Since 1994, this possibility has rarely been applied.

Cf. E: Use of mineral additions:

Mineral additions can contribute more or less effectively, according to their type and proportion, to the prevention of damage by AAR. They can be incorporated in concrete in two ways;

- either as an integral part of the cement, or
- when the concrete is mixed.

There are two means of judging if the quantity of additions is sufficient:

- Verify that the concrete mix design satisfies an analytical criterion (as for cements).
- Verify that the concrete mix satisfies a performance criterion (this later is the most often used).

Cf. F: Use of PRP aggregates with specific conditions

Concrete mix designs using PRP aggregates are suitable for use when the two following conditions are satisfied:

Condition 1:

- either the concrete mix designs contains only PRP aggregates
- or the aggregates (NR, PR or PRP) entering the mix design are globally characterized as PRP

Condition 2:

- either the granular mixture contains more than 60 % flint by mass, as determined by counting
- or the expansion of concrete specimen according to the long term test is less than 0,04 % at 8 months

4.3 References

1. Recommendations for the prevention of damages by the alkali-aggregate reaction, Laboratoire Central des Ponts et Chaussées, June 1994. (French and English version)
2. Recommandations Alcali-reactions - Federation Nationale des Entreprises de Travaux Publics-1985.
3. Les alcalis-reactions - Recommandations FNB-FNTP-SNBATI. Annales de l'Insthtut Technique du Bidment et des Travaux Publics - 1990.
4. AFNOR P18-011 Concretes - Classification of agressive environments – May 1985.
5. AFNOR NF P l5-301 Hydraulic binders – Common cements: Composition, specifications, and criteria of conformity, april 1994.
6. AFNOR XP P 18-540 Aggregates for concrete – Definitions, Conformity and Requirements
7. AFNOR XP P 18-594 Aggregates - Tests methods on reactivity to alkalis (February 2004)
8. AFNOR FD P 18-542 Les critères de qualification des granulats naturels pour béton hydraulique vis a vis de l’alcali-reaction, février 2004.
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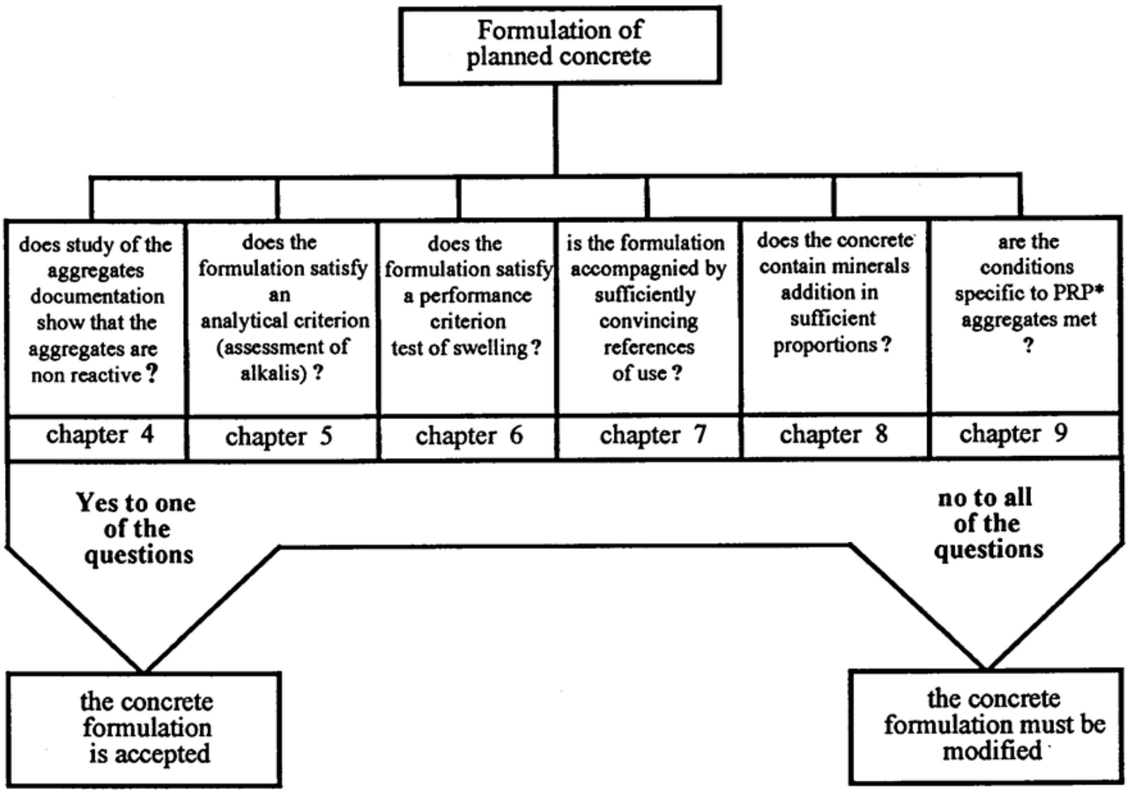
12. B. BOLLOTTE: Development of an accelerated performance test on concrete for evaluating its resistance to AAR - 9th International Conference on AAR in concrete (London 27-31 July 1992).
13. Granulats pour betons hydrauliques - Guide pour l'élaboration du dossier carriere - Document annexe aux recommandations pour la prévention des désordres dus à l’alcali-réaction -Ministere de l'Equipement du Logement, de l'Amenagement du Territoire et des Transports- LCPC, Juin 1994.
14. Provisional recommendations for the prevention of damage by the alkali-aggregatereaction -Ministere de l'Equipement, du Logement, de l'Amenagement du Territoire et des Transports -1991.



Methodological diagram for level of prevention B

5 Germany

The following Chapter has been reviewed by Ingmar Borchers, VDZ - German Cement Works Association.

5.1 Introduction

To get some idea of the possible risk that an alkali-aggregate reaction may constitute to concrete constructions in Germany extensive tests were performed in the 1970's. Because of the complex nature of the alkali-aggregate reaction, a great number of aspects had to be considered, such as the influence of type, quantity and grain size of the aggregate as well as that of the alkali content. (1) Extensive concrete tests and investigations into the behaviour of structures in Germany have been carried out.

The research into the behaviour of structures was also necessary because concrete tests in laboratories do not cover all practical construction factors, such as concrete production under field conditions, the influence of the dimensions of the building units, loading and stress conditions of the reinforcement and design details.

5.2 Specifications against ASR

The German Committee for Structural Concrete (Deutscher Ausschuss für Stahlbeton, DAfStb) gives detailed guidance in its recommendations. The first preliminary draft of the Alkali Guidelines was published in 1974 and the latest version in 2007 (2). The Guidelines are divided into 3 parts. Part 1 includes general aspects like the scope and types of aggregates that have to be tested and the environmental conditions. Part 2 describes the testing of aggregates with opaline sandstone including siliceous chalk and flint and the measures that have to be taken. This part must be used in the whole north of Germany (see Figure 5.2-1).

Part 3 deals with the assessment and the measures in case of listed types of crushed alkali sensitive aggregates, which have to be tested in Germany. In the last years damage occurred in a few concrete pavements on motorways 10 to 12 years after construction. In these cases aggregates like crushed gravel from the Upper Rhine Valley, rhyolite and greywacke had been used. With respect to this damage the Guidelines were revised in 2007.

There is no differentiation between fine and coarse aggregates in the Alkali Guidelines. The measures against ASR depend on the type of aggregate, the alkali reactivity, the cement content of the concrete and the environmental conditions of the concrete construction/element.

5.2.1 Limits on alkalis in cement

The following cements are classified as low alkali cements (NA) according to the German standard DIN 1164 (3).

Table 5.2-1: Limits for low alkali cements (NA) according to DIN 1164 (3)

Cement	Total Na_2O_{eq} of cement [wt.-%]	
CEM I – CEM V without CEM II/B-S	≤ 0.60	
Cement with ground granulated blastfurnace slag (ggbs)	Content of ggbs [wt.-%]	Total Na_2O_{eq} of cement with ggbs [wt.-%]
CEM II/B-S	≥ 21	≤ 0.70
CEM III/A	≤ 49	≤ 0.95
	≥ 50	≤ 1.10
CEM III/B and CEM III/C	66 - 80 and 81 - 95	≤ 2.00

5.2.2 Assessment of aggregate reactivity

Gravel with opaline sandstone and flint (Alkali Guidelines Part 2):

The alkali reactive rock types occurring in north Germany are opaline sandstone and to a lesser extent porous flint. As a consequence of various tests the alkali-reactivity of aggregates is assessed in Germany on the basis of their content of opaline sandstone and reactive flint. As table 5.2-2 and table 5.2-3 show, there are categories of "non-reactive", "conditionally useful" and "reactive" aggregates. There are special test procedures in the German specifications. (2)

If the cement content is below or equal to 330 kg/m³, only opaline sandstone has to be considered to assess the alkali-reactivity of an aggregate or a special grading range of an aggregate. There are three classes of alkali-reactivity depending on the percentage of reactive opaline sandstone in the aggregate. The limits for the reactivity classes E I-O to E III-O are listed in table 5.2-2. Preventive measures for these cases are summarised in table 5.4-1.

Table 5.2-2: Classes of alkali-reactivity concerning aggregates with opaline sandstone with a cement content ≤ 330 kg/m³. *⁾ Including reactive flint 1 to 4 mm.

	Limits in % by weight for the reactivity groups		
	E I-O (Non-reactive)	E II-O (Conditionally useful)	E III-O (Reactive)
Opaline sandstone + other opal-containing rocks *⁾	≤ 0.5	≤ 2.0	> 2.0

If the cement content is higher than 330 kg/m³, opaline sandstone and flint have to be considered to assess the alkali-reactivity of an aggregate or a special grading range of an aggregate. If the aggregate contains opaline sandstone in critical proportions, it could under adverse conditions cause damage, no matter whether the grain size is 0/32 mm or 4/32 mm. However, under otherwise equal conditions the highest degree of damage is produced when the size range 2/8 mm contains the reactive aggregate and the remaining aggregate is made up of inert material. There are also three classes of alkali-reactivity depending on the percentage of reactive opaline sandstone and flint in the aggregate. The limits for the reactivity classes E I-OF to E III-OF are listed in table 5.2-3. Preventive measures for these cases are summarised in table 5.4-2.

For the assessment of aggregates that contains gravel with opaline sandstone test methods according to the Alkali Guidelines Part 2 must be used. This includes:

- Petrographic classification of non-reactive constituents, opaline sandstone and flint.
- Boiling of opaline sandstone in 90 °C NaOH-solution (4 % NaOH for grain sizes up to 4 mm, 10 % NaOH for bigger grain sizes) for one hour, determination of loss of mass, calculation of alkali-sensitive amount of opaline sandstone.
- Determination of density of flint, calculation of alkali-sensitive amount of flint.
- Assessment of alkali-reactivity of the whole aggregate.

Table 5.2-3: Classes of alkali-reactivity concerning aggregates with opaline sandstone and flint with a cement content > 330 kg/m3. *) Including reactive flint 1 to 4 mm.

	Limits in % by weight for the reactivity groups		
	E I-OF (Non-reactive)	E II-OF (Conditionally useful)	E III-OF (Reactive)
Opaline sandstone + other opal-containing rocks *)	≤ 0.5	≤ 2.0	> 2.0
Reactive flint over 4mm	≤ 3.0	≤ 10.0	> 10.0
5 x (opaline sandstone + other opal-containing rocks) + reactive flint	≤ 4.0	≤ 15.0	> 15.0

<u>Crushed alkali sensitive aggregates (Alkali Guidelines Part 3):</u>

According to Part 3 of the Alkali Guidelines (2) the application of precautionary measures is required if the following types of crushed alkali sensitive aggregates are used:

- crushed greywacke,
- crushed rhyolite,
- crushed gravel from the Upper Rhine Valley,
- recycled aggregates,
- gravel that contains more than 10 % by weight of crushed fractions of the above-mentioned aggregates,
- crushed aggregates, if no experience exists in the application area of the Alkali Guidelines and
- other reactive crushed aggregates if damage has occurred.

The preventive measures, which have to be applied, depend on the alkali-reactivity class of the aggregate, which is the result of the assessment according to the Alkali Guidelines. Three different test methods could be used to find out if an aggregate is potentially reactive or not.

Crushed alkali sensitive aggregates are tested according to the scheme in figure 5.2-2. At first an aggregate is tested with the ultra accelerated mortar bar test. If it passes the test, the aggregate is non-reactive (E I-S) and no further tests or measures are necessary. If the aggregate fails, a categorisation is not possible and a concrete prism test with storage at 40 °C in a fog chamber has to be done. Endeavours are being made to apply the accelerated concrete prism test at 60 °C as an alternative to the 40 °C fog chamber storage method in the future in order to accelerate the testing of aggregates in concrete prism tests. The method is described in an informative annex to the Guidelines. If the expansion of the prisms does not reach the limit no further test is necessary and the aggregate is non reactive (E I-S). If an aggregate fails the concrete prism test it is reactive (E III-S).

The limits for the different test methods are given in table 5.2-4, 5.2-5 and 5.2-6.

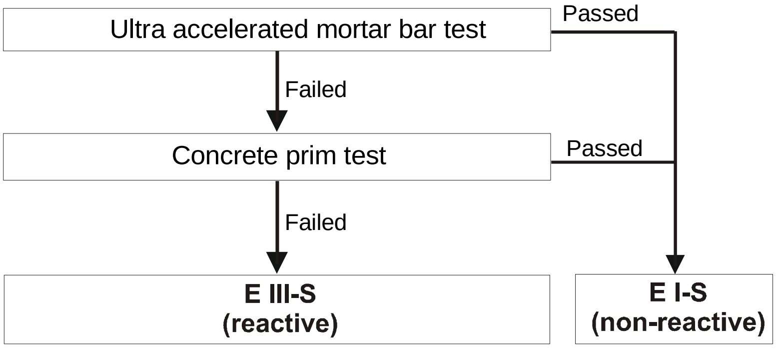


Figure 5.2-2: Flow chart for the aggregate test described in Part 3 of the Alkali Guidelines

Accelerated mortar bar test at 80 °C (Reference method)

- Test is like RILEM AAR-2.
- Assessment of alkali-reactivity after 13 days of storage in 1 M NaOH.

Accelerated mortar bar test at 70 °C (Alternative method)

- Preparation of mortar bars is similar to RILEM AAR-2.
- Crushed and sieved aggregate (0.5/1 mm and 1/2 mm) and non-reactive quartz sand (0.1/0.5 mm).
- Addition of NaOH to the concrete mix water to increase the alkali content of the binder to 2.5 wt.-% of sodium oxide equivalent.
- Storage of mortar prisms (4 cm x 4 cm x 16 cm) at 70 °C above water.
- Determination of length change of the beams at 20 °C.
- Assessment of alkali-reactivity after 28 days.

Concrete prism test in the 40 °C-fog chamber (Reference method)

- Concrete prism test, 3 prisms (10 cm ×10 cm × 50 cm) and one 30 cm cube are stored in a 40 °C fog chamber for nine months – cf. the German test method described in the main report in section 7.
- Determination of length change of the beams and cracking of the cube.
- Assessment of alkali-reactivity after nine months.
- (Additionally petrographic investigations of thin sections, especially for tests with aggregates, which are not known to be reactive).

Accelerated concrete prism test at 60 °C (method is described in an annex to gain experience)

- The test is like RILEM AAR-4 with a different concrete composition.

Table 5.2-4: Limits for the expansion for the assessment of aggregates with the accelerated mortar bar test at 80 °C (Reference method)

	Reactivity class[1)]	
	E I-S Non-reactive	No classification possible [1)]
Limits for the expansion of mortar bar in mm/m after 13 days in NaOH	≤ 1.0	> 1.0

[1)] For classification a concrete prism test could be used.

Table 5.2-5: Limits for the expansion for the assessment of aggregates with the accelerated mortar bar test at 70 °C (Alternative method)

	Reactivity class[1)]	
	E I-S Non-reactive	No classification possible [1)]
Limits for the expansion of mortar bar in mm/m after 28 days over water	≤ 1.5	> 1.5

[1)] For classification a concrete prism test could be used.

Table 5.2-6: Limits for the expansion for the assessment of aggregates with the concrete prism test in the 40 °C-fog chamber (Reference method)

	Reactivity class[1)]	
	E I-S Non-reactive	E III-S Reactive
Limits for the expansion of concrete beams in mm/m[2)]	≤ 0.6	> 0.6
Cracking of cubes	None	Severe[3)]

[1)] The more unfavourable assessment is relevant

[2)] After 9 month storage in a 40 °C fog chamber including expansion through elevated temperature and swelling

[3)] With a crack width of w ≥ 0.2 mm

5.3 Environmental conditions

Besides a sufficiently alkaline solution and an alkali-reactive aggregate there must be a sufficient supply of water for an alkali silica reaction that can cause damage. If the external moisture contains salt (with Na and K) it is possible that the risk of damage from ASR is increased. In the specifications the following four environmental conditions are defined:

Dry = WO

Humid = WF

Humid and alkalis from outside = WA

Humid, alkalis from outside and high dynamic loads (only for concrete pavements) = WS

5.4 Precautionary measures

Gravel with opaline sandstone and flint (Alkali Guidelines Part 2):

On the basis of the present state of knowledge the precautionary measures set out in table 5.4-1 and table 5.4-2 are recommended in Germany as being on the safe side.

Table 5.4-1: Precautionary measures against deleterious Alkali-Aggregate Reaction in concrete (apply only to opaline sandstone and reactive flint) of concrete with a cement content ≤ 330 kg/m^3.

	Environmental Conditions			
Alkali-reactivity of the aggregate	WO (Dry)	WF (Humid)	WA (Humid with additional alkalis from outside)	WS (Humid with additional alkalis from outside and dynamic loads)
E I-O (non-reactive)	None	None	None	Cement according to table A5.9
E II-O (Conditionally useful)	None	None	Low-alkali cement according to DIN 1164	Substitution of aggregate
E III-O (Reactive)	None	Low-alkali cement according to DIN 1164	Substitution of aggregate	Substitution of aggregate

Table 5.4-2: Precautionary measures against deleterious alkali-silica reactions in concrete (apply only to opaline sandstone and reactive flint) for concrete with a cement content > 330 kg/m^3.

	Environmental Conditions			
Alkali-reactivity of the aggregate	WO (Dry)	WF (Humid)	WA (Humid with additional alkalis from outside)	WS (Humid with additional alkalis from outside and dynamic loads)
E I-OF (Non-reactive)	None	None	None	Cement according to table A5.9
E II-OF (Conditionally useful)	None	Low-alkali cement according to DIN 1164	Low-alkali cement according to DIN 1164	Substitution of aggregate
E III-OF (Reactive)	None	Low-alkali cement according to DIN 1164	Substitution of aggregate	Substitution of aggregate

Table 5.4-3: Required characteristic value for the alkali content of cements for constructions elements exposed to environmental condition WS

Cement	Content of ground granulated blastfurnace slag (ggbs) [wt.-%]	Na_2O_{eq} of cement [wt.-%]	Na_2O_{eq} of cement without ground granulated blastfurnace rsp. burnt shale [wt.-%]
CEM I + CEM II/A	-	≤ 0.80	-
CEM II/B-T	-	-	≤ 0.90
CEM II/B-S	21 - 29	-	≤ 0.90
CEM II/B-S	30 - 35	-	≤ 1.00
CEM III/A	36 - 50	-	≤ 1.05

Crushed alkali sensitive aggregates (Alkali Guidelines Part 3):

On the basis of the present state of knowledge the precautionary measures set out in table 5.4-4 are recommended in Germany as being on the safe side.

Table 5.4-4: Measures to prevent deleterious alkali silica reactions in concrete when using aggregates described in Part 3 of the Alkali Guidelines (2)

Reactiv-ity class	Cement content in kg/m³	Preventive measures for humidity class			
		WO (Dry)	WF (Humid)	WA (Humid with additional alkalis from outside)	WS (Humid with additional alkalis from outside and dynamic loads)
E I-S	Without limit	None	None	None	Cement according to table A5.9
E III-S 1)	c ≤ 300	None	None	None	Cement according to table A5.9
	300 < c ≤ 350	None	None	Performance-test 2) or Low-alkali cement according to DIN 1164	Cement according to table A5.9 and Substitution of aggregate or expert report 3)
	c > 350	None	Performance-test 2) or Low-alkali cement according to DIN 1164	Performance-test 2) or Substitution of aggregate	Cement according to table A5.9 and Substitution of aggregate or expert report 3)

1) Also applies for aggregates that have not been assessed.

2) The performance testing will be described in a future Part 4 of the Alkali Guidelines. Until further notice the stipulation of preventive measures will be based on an expert report 3).

3) Experienced technical experts must be called in for issuing expert reports.

5.5 Conclusion

The basic principles underlying the German specifications are similar to those of the UK. For a damaging alkali silica reaction the following three conditions must be met simultaneously:

- there must be a sufficiently alkaline solution in the pore structure of the concrete
- the aggregate combination must be susceptible to be attacked by this alkaline solution
- there must be a sufficient supply of water

5.6 References

1) Bonzel J. und J. Dahms: Alkalireaktion im Beton, beton 23 (1973) S. 153 - 188.

2) Vorbeugende Massnahmen gegen schädigende Alkalireaktion im Beton. (Alkali-Richtlinie): Deutscher Ausschuss Für Stahlbeton (Februar 2007).

3) DIN 1164-10 : 2004-08. Zement mit besonderen Eigenschaften.

Figure 5.2-1: Aggregate regions with opaline sandstone and flint in Northern Germany (2). Dark grey area: area of application for opaline sandstone and flint according to Alkali Guidelines, Part 2; Light grey area: adjacent area.

6 Greece

The following Chapter has been reviewed by Katerina Pavlou, TITAN Cement Company, S.A.

Almost 90% of the aggregates that are used for concrete in Greece are calcareous. There has never been any alkali silica reaction reported officially in the cases that concrete constructions have been manufactured with siliceous aggregates. According to the current Greek Standard ELOT 408 “Aggregates for concrete” aggregates containing the following mineral phases are considered to be alkali silica reactive and therefore should be avoided for use in concrete:

- Opal
- Chalcedony
- Tridymite
- Cristobalite
- Rhyolite
- Andesine

7 Iceland

The following Chapter has been reviewed by Børge Johannes Wigum, Hönnun Consulting Engineers.

7.1 Introduction

In the late sixties and seventies research on preventive measures against ASR damage in concrete was carried out at the Icelandic Building Research Institute, (IBRI) [1, 2, 3, 4 5, 6, 8, 9 & 10]. Natural pozzolanas found in Iceland proved to be effective as well as the use of silica fume. Due to this research, effective countermeasures could be taken once ASR damage had been proven.

Interest in silica fume in Iceland dates back to 1972, when the first samples of the material were tested as soon as a ferrosilicon plant was conceived in the country. Silica fume proved to be a very effective suppresser of alkali-silica expansion. Based on these results the Icelandic building authorities enforced the strict limit on allowable mortar bar expansion mentioned below. In line with this the State Cement Works has since 1979 only produced cement intermixed with silica fume, replacement at first being 5%, but since 1983 7,5%. Research at IBRI has shown that even such a small amount is highly effective in preventing deleterious expansion. Such a level of silica fume has none of the serious negative side effects, such as higher water requirement or increased drying, shrinkage, associated with higher levels [15].

Prior to the use of silica fume, finely ground rhyolite pozzolanas was intermixed with OPC, starting in 1973 with 3% and increasing to 9% in 1975-79 [15]. Measurements over a period of 10 years show that silica fume reduces ASR expansion drastically but it has to be kept in mind that it also delays expansion that will continue in some degree over a longer period of time.

In the 80's considerable research was carried out on how to stop pre-existing ASR in structures (houses) and how to maintain the structures. Full-scale experiments were made using: claddings, mortar-insulation systems, hydrophobic impregnation, other surface treatments, and shotcrete [7, 11, 12, 13, & 14]. On the bases of the results IBRI has published recommendations in this field.

The situation during the last years is now however more complicated. Since import of cement started some years ago, there are various types of cement on the marked. The original 7,5% adding of silica fume in the Icelandic cement is now reduced to 4-6% depending on type of cement. A special Icelandic cement with 10% silica fume and 25% grounded rhyolite is not longer available. Cement, aggregates and various admixtures for concrete are imported on a large scale.

7.2 Specifications against ASR

The principal test method used for evaluating alkali-silica reaction has been the well-known ASTM - C227 mortar bar method. The ASTM 6-month criterion of 0,1% expansion was at first used for guidance. However, in 1979, the Icelandic Building Code demanded a stricter criterion of 0,05% in 6 months and 0.1 % in 12 months [15]. Other methods such as the ASTM quick chemical test, ASTM C289, and petrographic analyses have not been found suitable for Icelandic aggregates. These methods were however previous used for preliminary information prior to the mortar bar measurement (8).

Another building code restriction introduced in 1979 is that unwashed sea-dredged reactive aggregates are now banned. The reason for this ban is that NaCl from the sea may exchange ions with $Ca(OH)_2$ liberated during the cement hydration and form NaOH, which increases the alkalinity of the concrete and its alkali reactivity [15].

The Building regulations from 1998 [16] requires that all aggregate for concrete shall be tested with respect to AAR. The aggregate producer shall test his aggregates on a regular basis, and obtain a certificate from an independent laboratory, classifying the aggregate as reactive or innocuous. The testing shall be carried out according to ASTM C-227 and or ASTM C-1260. Critical limits for assessment of reactivity are presented in Table 7.2-1. In the cases where aggregates are classified as reactive, it is the responsibility of the aggregate producer to prove that the prospected mix of aggregates and cement is non deleterious. This has recently been criticised, because no concrete performance tests are available in Iceland, and because this responsibility should belong to the concrete producer.

Table 7.2-1: Critical limits for assessment of reactivity [16].

Test methods	Type of cement	Critical expansion limits		
		14 days	6 months	12 months
ASTM C-227	Icelandic Portland cement without pozzolanas		< 0,05%	< 0,1%
ASTM C-1260	Icelandic Portland cement without pozzolanas	< 0,2%		
ASTM C-227	Cement applied in concrete project			< 0,1%
ASTM C-227	Cement with silica fume			< 0,06%
ASTM C-1260	Cement applied in concrete project, or cement with silica fume.	< 0,1%		

7.3 Icelandic research in progress

In order to maintain the security against deleterious AAR it has been found necessary recently to take up again the research in Iceland, particularly in accordance to the new test methods developed internationally. Particular attention has been paid to the two concrete prism methods (RILEM AAR-3 & AAR-4) and the competence of the mortar bar test (ASTM C 227) and the accelerated mortar bar tests (ASTM 1260 & RILEM AAR-2) to mirror the effects of various types of cements and pozzolanas, particularly silica fume. [17].

The main conclusions from the project are as following:

- The use of silica fume and cements with low alkali content exhibit little influence on the results of the accelerated mortar bar expansion according to the method; RILEM AAR–2. This method is similar to the method ASTM C 1260 as described in the Icelandic building regulations. It is proposed that results from RILEM AAR-2 should only be used for assessing the reactivity of aggregates, and not as an assessment of the effects of the additives and types of cement. Mortar mixes exhibiting expansion greater than critical limits in the test will not necessary develop deleterious expansion in real concrete mixes.
- When using cement with 4% silica fume in the test method ASTM C 227, the expansion decrease by a factor of approximately 0.5, compared to same aggregate with cement with no silica fume. Comparison of results from the two mortar bar tests; RILEM AAR–2 and ASTM C 227, shows a poor correlation.
- By testing Icelandic materials, it is believed that the concrete prism test; RILEM AAR-3, is the best method reflecting real field conditions. However, based on the shorter testing period for the accelerated concrete prism test; RILEM AAR–4, this test is considered to be the most suitable test in the future. It is the ambition to continue the development of this test in Iceland.
- It is realised a future need to make connection with requirements of critical limits of the laboratory tests and what is happening in real concrete structures. A new field study is required as it is already 15 years since the last study was carried out in Iceland.

It is found that the accelerated mortar bar test, RILEM AAR–2, exhibits significantly higher expansion than the other test methods examined. The two new concrete prism tests from RILEM appear to reflect well the effects of types of aggregates, types of cements and additives of pozzolanas. The mortar bar test ASTM C227 reflects well the effect of pozzolanas, however, it appears to exhibit to little expansion for mixes with cements with low alkali content. One explanation for this could be the effect of leaching which will be more in long and thin mortar bars.

Based upon the results from this recent project it is considered necessary to review the current Icelandic building regulations, both regarding the test methods described and requirements of critical expansion limits. Resent results, along with continuous research will enable the development of an Icelandic national document providing guidelines for production of durable concrete regarding AAR.

7.4 References

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8 Ireland

The following chapter is based upon a review by Mark Richardson (2005) [1], School of Architecture, Landscape and Civil Engineering, University College Dublin, Ireland. He was chairman of the national working party in Ireland that updated guidance first published in 1991 as a revised 2003 version {2].

8.1 Background

National guidance on minimising the risk of alkali-silica reaction was first published in the Republic of Ireland in 1991 and was revised in 2003 [2]. The introduction to practice of European Standard EN206-1 and the pending withdrawal of conflicting national standards prompted a review. This has recently concluded and revised national guidance on minimising the risk of deleterious reaction was published.

8.2 Developments in testing

The IEI/ICS guidance published in 1991 could not be definitive in relation to testing. It referred to petrographic assessment such as ASTM C295 [3], the quick chemical test ASTM C289 [4], and an accelerated mortar-bar expansion test, ASTM C227 [5]. The practitioner was advised that these tests were not definitive but could be helpful in making a judgement on the acceptability of an aggregate.

Development of expansion tests has been the focus of greatest interest in the last decade and work continues. The ASTM C227 mortar-bar test has been effectively superseded by the development of ultra-accelerated mortar-bar expansion tests. These include ASTM C1260 [6], BS DD249 [7], and RILEM AAR-2 Ultra-Accelerated Mortar Test [8]. There is doubt about reliance on accelerated mortar-bar tests for definitive findings but they may serve as rapid screening tests.

Long-term expansion testing of concrete prisms may develop to the stage of being definitive. These include the RILEM AAR-3 Concrete Prism Test [9] and BS 812 Part 123 [10]. Prisms of 75 mm•75mm•250mm are cast with an alkali load of about 7 kg/m^3. A further method, in which concrete prisms are stored at 60^o C, is being developed to reduce the period of test.

None of these tests are definitive, particularly in places such as the Republic of Ireland, where there is limited knowledge of their applicability in Irish conditions, since there is insufficient evidence of AAR occurrences in practice on which to build a comparative database. Indeed, a significant number of Irish aggregates fail to remain below the 0.10% limit for innocuous classification in the mortar-bar test and the 0.05% limit in the concrete prism test, despite satisfactory service history to date. However, the specifier is advised that such test results, in conjunction with petrographic examination, may be helpful in deciding if the aggregates are classifiable as unlikely to be alkali-reactive.

8.3 Revised guidance for the Republic of Ireland

The national Joint Working Party considered the recent national and international developments and updated the 1991 guidance in 2003. The opportunity was used to streamline the advice as much as possible. Judgement was exercised to strike a careful balance between minimising the complexity of the guidance and maximising the effectiveness of risk management. It will be appreciated that it is difficult to legislate for a problem that has yet to manifest itself. The possibility of being too lenient is as likely as being too conservative until one is wise after the event! A summary of the updated advice is presented in flowchart form in Figure 8.3-1 and the specific guidance in respect of alkalis contributed by ggbs and pfa is presented in Table 8.3-1.

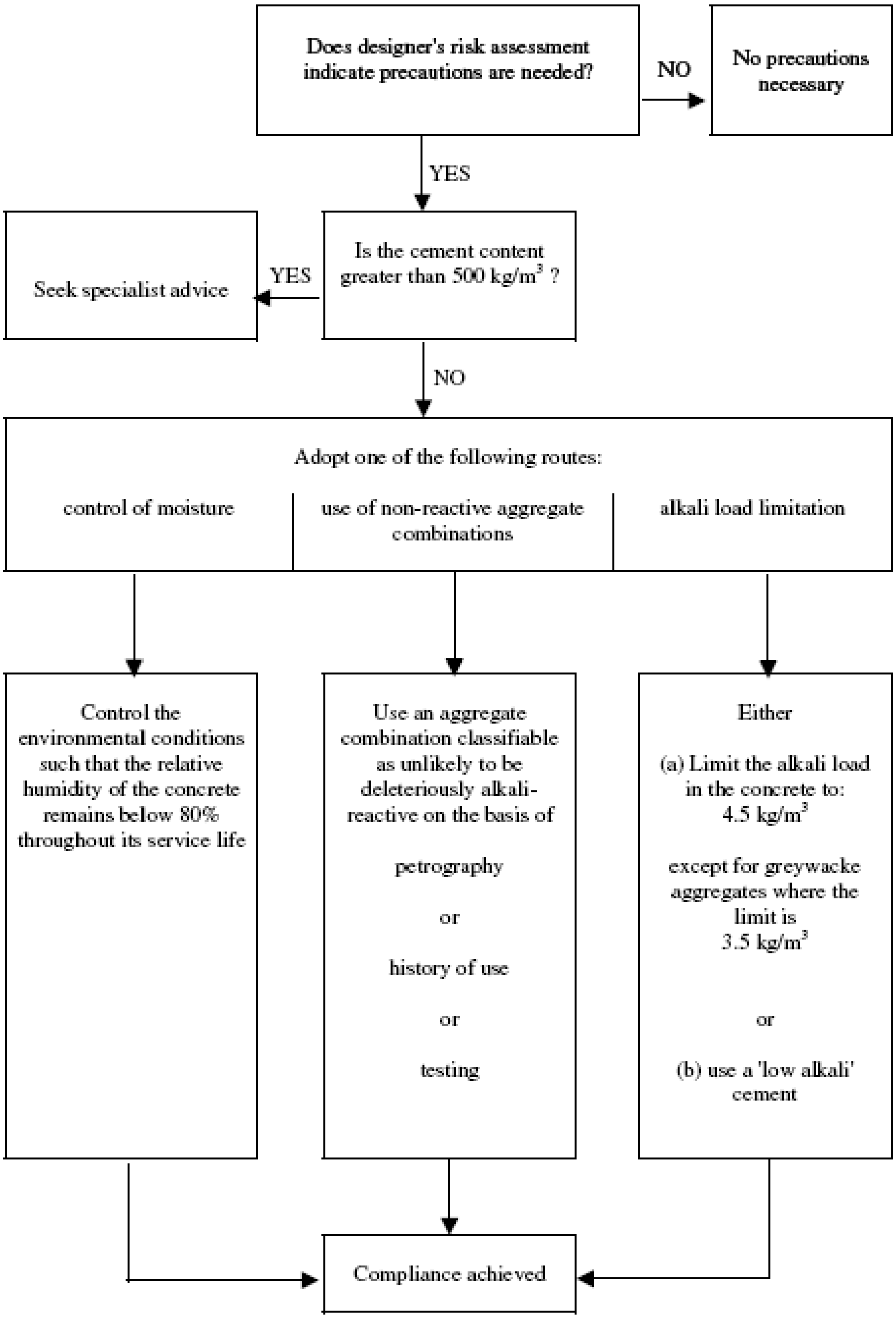

Figure 8.3-1. Flowchart summarising proposed recommendations for future Irish practice [1].

Table 8.3-1. Proposed future Irish practice on alkali contribution of slag and ash [1].

Contributor	Content	Determination
Ground granulated blastfurnace slag	Less than 40%	ggbs content × (average Na_2O_{eq} + 1.64 SD)
	Equal to or greater than 40%	ggbs content × 0.5 (average Na_2O_{eq} + 1.64 SD)
Pulverised fuel ash	Less than 21%	pfa content × (average Na_2O_{eq} + 1.64 SD)
	Equal to or greater than 21%	pfa content × 0.2 (average Na_2O_{eq} + 1.64 SD)

The principal features of the revised guidance are as follows:

- the advice is limited to current experience, defined as concrete with cement contents less than 500 kg/m^3;
- the flowchart presents the control routes in three parallel streams of equal status, whereas the original guidance prioritised the routes with alkali load limitation being a last resort;
- the alkali load limit is standardised at 4.5 kg/m^3; for all but greywacke aggregates, whereas the previous guidance had different conditions for carboniferous chert;
- a safety margin of 1.0 kg/m^3 is imposed on the alkali limitation route where greywacke aggregates are used that have not been declared innocuous;
- the alkali contribution of ggbs and pfa is determined on a more conservative basis than heretofore where the replacement level is at the lower end of the scale.

8.4 References

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9 Italy

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9.1 Specifications against ASR

In the UNI Standard 8981 - part 1 (under final approval in 2006) “Durability of concrete works and precast concrete elements – Instructions for obtaining the resistance to aggressive actions” the alkali-silica reaction is quoted as cause of concrete deterioration in the presence of reactive aggregates.

Recommendations to ascertain the risk conditions for the reaction and to prevent it are given in UNI Standard 8981 - part 2, (under final approval in 2006) “Durability of concrete works and precast concrete elements – Criteria for the avoidance of alkali-silica reaction”).

To promote the alkali-silica reaction the presence of the following simultaneous factors are considered necessary:

- Reactive silica in the concrete aggregates
- Sufficiently alkaline pore solution in the concrete
- Exposure conditions of the structure such as to provide a sufficient supply of water to the concrete

Moreover, the rate at which this reaction develops is considered to depend on the environmental humidity and temperature and on the possibility of further alkali supply from the environment.

In Italy there is a concentration on:

- avoiding reactive aggregates or
- limiting the alkalinity of the concrete pore solution, mainly through the identification of safe cement-aggregates combinations.

For normal building and civil engineering structures and in absence of external alkali supply, a limit of 3.0 Na_2O_{eq}/m^3 for the Total Effective equivalent Sodium oxide (SET) has also been proposed.

The European Standard UNI EN 206 –1 (Concrete - Specification, performance, production and conformity), has some additional provisions for its application in Italy (UNI 11104 - 2004“Additional provisions for the application of EN 206-1”). In order to ascertain the suitability of the concrete aggregates for alkali-silica reaction (according to European Standard on aggregates UNI EN 12620), these additional provisions refer to a procedure described in the Italian Standards UNI 8520-2 and 8520-22 (*see Assessment of the aggregate reactivity*). A partial mapping of the critical areas where potential reactive minerals and rocks are present has been proposed and some experimental activities have been undertaken (1, 2, 3). Different classes of aggregates, according to their different flint contents, were identified, although no complete thorough evaluation of their reactivity in concrete has been made so far.

The identification of blended cements able to prevent expansion phenomena due to aggregate reactivity, is to be carried out according UNI 8520-22 through the mortar bar test or, according to UNI 8981-2, through a concrete test method (*see Use of blended cements and mineral additions as preventive measures*).

9.2 3Assessment of the aggregate reactivity

The assessment of the aggregate reactivity has to be carried out according to the UNI 8520-2 “Aggregates for concrete - Additional provisions for the application of EN 12620 – Requirements” (2005).

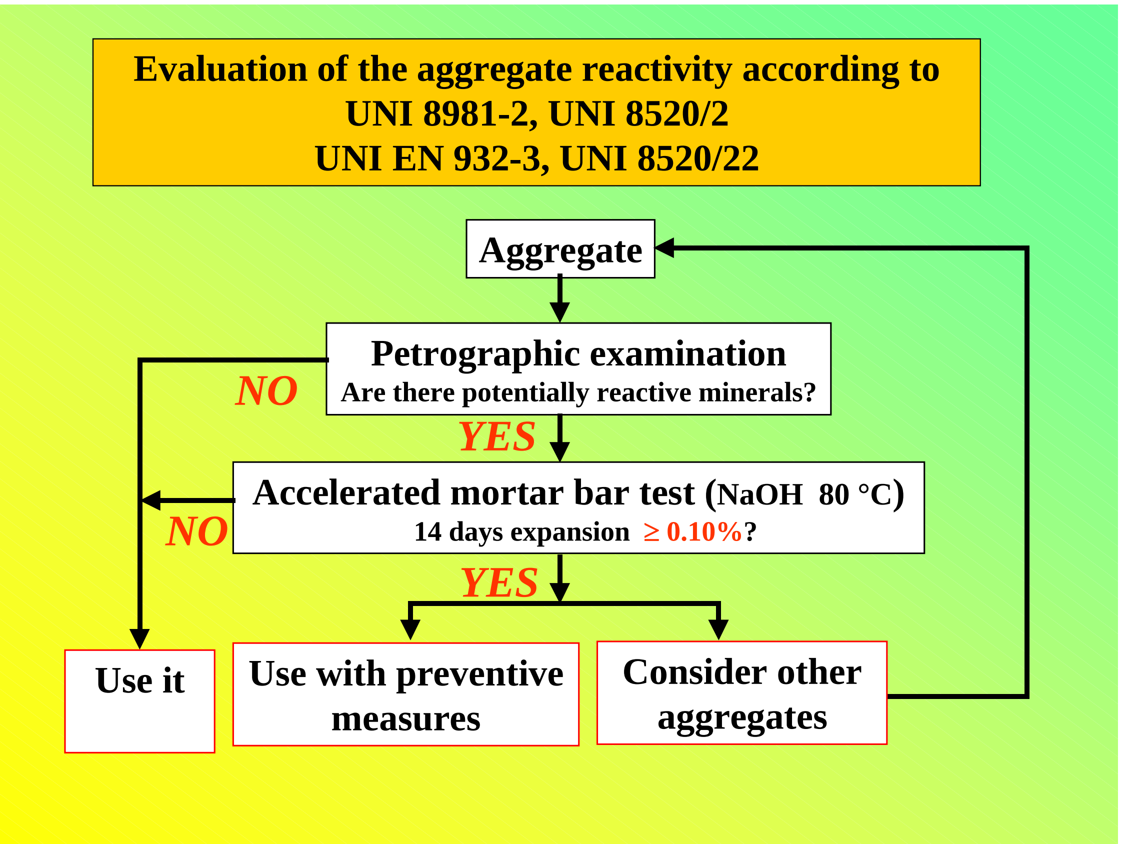
If the petrographic examination UNI EN 932-3 “Tests for general properties of aggregates – Procedure and terminology for simplified petrographic description”(2004) does not reveal any reactive minerals, the aggregate is considered not reactive to alkali and suitable for use in concrete.

In the contrary case (presence opal and amorphous silica, volcanic glass, microcrystalline fibrous quartz such as chalcedony, strained quartz) the aggregate is tested according to UNI 8520-22 “Aggregates for use in concrete – Determination of potential reactivity of alkali in aggregates (2002), and in particular to the proposed accelerated mortar bar test in 1 N NaOH solution at 80°C, using 25x25x285 mm size mortar bars. If the 14 days expansion at the accelerated mortar bar test is less than 0.10% the aggregate is considered as non reactive.

In the contrary case, according the same UNI 8520-22, long term expansions on mortar bars at 38°C and 100% R.H. should be considered. For this last test 0.08% expansions at 3 months and 0.10% at 6 months are the suggested limits and, if they are exceeded, preventive measures based on suitable cement-aggregate combinations are to be adopted.

However, the new criteria reported in the UNI 8981 - part 2, under final approval in 2006, the long term mortar bar test should be disregarded and the evaluation criteria of the aggregate should be based only on the accelerated method.

The general procedure is schematically reported in the following flow chart.



9.3 Use of blended cements and mineral additions as preventive measures

The use of blended cements as well as mineral additions (natural pozzolanas, fly ash, blast furnace slags, silica fume) in concrete is a normal practice in Italy, particularly in those regions where there is a high risk of alkali-silica reaction. The aim is to produce safe cement/aggregate combinations for the concrete, able to prevent expansion phenomena, independently of their total alkali content.

According to the UNI 8520-22 the effectiveness of blended cements and mineral additions is assessed using the job aggregate in the long term mortar bar test at 38°C and 100% relative humidity. The tested combination is considered suitable for concrete if the 3 and 6 months expansion values remain below the same limits above mentioned for the assessment of the aggregate reactivity (0.05 and 0.10% respectively).

Nevertheless, because of the drawback of the long operating time of this method, the application of the accelerated mortar bar test at 80°C in NaOH solution is being developed (4, 5, 6, 7).

According to the new UNI 8981-2 (under publication in 2006), in the case of reactive aggregate a concrete test should be used to verify the cement-aggregate suitability. At the moment there is no a concrete test in Italy and, as reference, the RILEM test method B-TC 106-3 (2000), now under revision as AAR-3(2006), or the ASTM 1293 (2005) are suggested.

However, the Standard Italian Organisation (UNICEMENTO) recently promoted a pre-standard research with the aim of find a suitable method to evaluate the efficiency of mineral additions in cement to prevent the alkali-silica reaction in concrete.

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10 Netherlands

The following chapter is excerpted and updated by Maarten A.T.M. Broekmans, Geological Survey of Norway, Department of Mineral Characterization, from the 'General introduction' in the PhD-thesis of Broekmans 2002 [1].

10.1 The Dutch history of ASR

Before 1991

A thorough literature survey reveals that the first Dutch contribution to ASR knowledge dates from 1957, from the hand of Bosschaert [2]. He arguments against the general Dutch attitude back then that a different aggregate material mixed with a different cement would provide adequate protection for ASR. Instead, Bosschaert warns that it is not so much about individual materials being susceptible to deleterious ASR, but a particular *combination* of materials. The paper also urges for caution in selecting (aggregate) materials, and he strongly recommends collaboration between civil engineers and mineralogists [3]. Furthermore, though Bosschaert does not use thin section petrography, this is most probably the first publication to recognize ASR in Dutch concrete, and the author provides credible arguments that the damage described in another earlier article may in fact have to be attributed to ASR too [4].

The next Dutch contribution to ASR-literature dates from 1962 [5] and is also listed in Diamonds' bibliography [6] on worldwide ASR. The article bears the more general character of a review. Of the European countries, only Denmark is referred to as having alkali-reactive aggregate, whereas Great-Britain is mentioned to be free of such deleterious materials. Furthermore, the article provides first experimental expansion data on Dutch aggregate materials.

The authors explicitly state that only opal significantly expands, and that the expansion of all other investigated materials (chalcedony, flint, silex) is negligible, even after two years of exposure to Portland cement. Expansion maximum (pessimum) is reported to occur at around 2.5% content of opaline silica and was also found to correlate with grain size, in good agreement with previous results by others. Fluviatile sands from the Meuse river were found innocuous. In their final summary, the authors state "...the crypto-crystalline forms of SiO_2 are inaccurately marked as deleterious...". Their final conclusion "It is highly unlikely that ... the Dutch aggregate material contains (alkali) reactive constituents." must be held responsible for the fact that ASR has been neglected in the Netherlands for the three subsequent decades.

In 1980, the PhD-thesis from Dutch author Bakker discussing the influence of blast furnace slag cement on ASR and sulphate corrosion is published at the Technical University of Aachen in Germany [7]. Most of the positive ameliorating effects of slag are attributed to the immobilization of alkalis as well as to the different porosity distribution of slag cement. In 1988, more than thirty years after the first Dutch article by Bosschaert, Flemish author De Ceukelaire reports ASR as the most likely cause for the deterioration of a then 20-some year old structure [8]. Alkali-silica reaction of porous chert in the fine aggregate is held responsible for the damage. The pre-tensioned viaduct at Kontich in the peripheral highway around Belgian capital Brussels had hand-breadth cracks lengthwise, and was demolished in the night of April 27, 1995 using explosives.

The proceedings of the 8th International Conference on Alkali-Aggregate Reactions in Concrete held in Kyoto, Japan in 1989 contain two contributions by Flemish authors Soers and Meyskens [9] and Van Gemert [10], as well as two papers co-authored by Larbi [11,12], at that time a PhD-student at the Technical University of Delft. With the same (co-) author Hudec, Larbi published two more papers on rapid testing of the alkali-reactivity potential of rocks [13,14], in 1989 and 1990, respectively.

In 1990, Flemish authors Soers and Meyskens published a general overview on Belgian ASR in the Dutch periodical Cement [15]. At the time of writing, ASR had been recognized in some twenty-five Belgian structures, including both in-situ poured and prefabricated concrete, and some structures had been replaced already. The subtitle of "Extending quality control necessary" in fact reaffirms the original suggestions published in the same periodical in 1957 [2]. However, none of the papers mentioned above confirms a Dutch occurrence of ASR.

1991: first time recognition of deleterious ASR in Dutch concrete

The first time that ASR was unequivocally confirmed in Dutch concrete by petrography dates from 1991 [16]. This article is not listed in [6] and nor are [8,15], somewhat strange as they were all published in the same periodical as [2] in 1957 and [5] in 1962. The article describes ASR as the main cause for the deterioration in a two-lane viaduct in a rural road near Schoonhoven. The damage consisted of intensive surface cracking and leaching of the concrete with the concomitant formation of lime stalactites. The reluctant attribution of the damage to ASR as the primary cause was partly due to limited access to the structure for structural assessment and sample extraction.

Porous chert is reported as the main alkali-reactive constituent in the coarse aggregate, though alkali-silica reaction products also have been observed in cracks in sericitic sandstone (i.e. containing fine-grained muscovite). Macroscopic cracks were assumed not to run very deep from the surface (essentially until the reinforcement mesh), and a more thorough assessment of the macro crack fabric was considered useful and said to be included in further research [p.11 in 16]. However, later publications by the same author with various co-authors do neither provide nor refer to such data [17,18].

Until some time after its discovery, the Schoonhoven ASR-case was considered an isolated and incidental curiosity, the proverbial exception confirming the rule that ASR was non-existent in the Netherlands. The original article furthermore reaffirms the general conviction of the immunity of Dutch concrete for deleterious ASR [p.11 in 16] that apparently had persisted despite suggestions made in 1957 [2]. In 1962, this immunity was attributed to the absence of alkali-silica reactive constituents in Dutch aggregate material as well as the use of different cement types [5; see above]. The article describing the Schoonhoven ASR-case does not subscribe to this argument and rather attributes the general immunity mainly to the extensive use of cements with a high slag content together with the less extensive use of high water/cement-ratio concrete mixes. However, data supporting these statements have not been published yet [16,17].

1991: establishment of a national research committee

In March 1991, the same year as when the Schoonhoven ASR-case was first published, the Dutch legislative organization ‘Centre for Civil Engineering Research and Codes’ (in short: CUR) installed committee B56 named ‘Alkali-silica reaction in concrete’, with initial objectives defined as [19]:

1. Identification of aggregate materials for Dutch concrete which are potentially ASR-deleterious, depending on winning location, through literature investigations and interviews,
2. To draft procedures for the assessment of the alkali-reactivity potential of aggregate materials for Dutch concrete,
3. To draft guidelines for the production of durable concrete with the use of potentially alkali-reactive aggregate materials, and finally
4. Communicating knowledge acquired with bullet points 1,2 and 3.

After “...undisputed occurrences of Dutch ASR had become known to the committee...” [ipse dixit p57 in 19], these were changed into the much more pragmatic objectives:

1. What causes the damage, and
2. How to prevent future damage.

In 1994, the B56 CUR-committee published CUR Recommendation 38 titled ‘Measures to prevent damage by the alkali-silica reaction (ASR) in concrete’ [20]. The procedures mentioned in the recommendation are not supposed to be used for assessment or appraisal of existing structures, though this application is not explicitly excluded [§2 in 20]. In addition, a supplementary background article published simultaneously clearly states that CUR Recommendation 38 is based upon just that kind of data [p.60 in 19].

CUR Recommendation 38 provided four methods for the assessment of aggregate material prior to use in concrete, including:

1. Long-term experience with the use of that material in concrete,
2. Microscopic analysis of the mineralogical composition,
3. Experiments with mortar bars or concrete prisms, and/or
4. Chemical determination of the reactivity.

However, while mentioning assessment methods (rather than describing), CUR Recommendation 38 failed to provide references to accepted standards for such methods. Even worse, it did not provide acceptance/rejection criteria. To correct these omissions, CUR Recommendation 38 was later (as effective per 1995) supplemented with two documents from the Ministry of the Flemish Community, Department of the Environment and Infrastructure [21,22]. The first document from 1991 [21] is a circular setting criteria for the maximum content (2vol%) of potentially alkali-reactive grains including porous chert, chalcedony and opal, differentiated for both coarse and fine aggregate materials, and outlining the petrographic point counting procedure. The later document from 1994 [22] is a code of conduct for ongoing construction works.

Together with the criteria given for the maximum allowable alkali-content of the cement as well as the final concrete mixture [both in 21, and even in 20], this should prevent deleterious ASR to occur in concrete produced following these guidelines. The Flemish circular explicitly states that ‘coarse-grained sandstone’ is considered innocuous to ASR. However, earlier observations on Dutch concrete [16,17], even from before the circulars were supplemented to CUR Recommendation 38 already mention that certain types of sandstone appear to be alkali-reactive too. Later, this has been confirmed on several occasions by the current author [23,24,25,26].

10.2 Recent Dutch developments in structure assessment

In June-July 1992, three viaducts named Heemraadsingel, Hoogeindse Rondweg and Wolput respectively, and one tunnel named HertogJanstraat, all in or over Dutch main road A59 were subject to a regular maintenance inspection. The main road had been constructed through the 1960's, in three different contracts, all using locally available aggregate materials. The four structures inspected all displayed extensive map cracking and exudations [27]. Petrographic analysis of cores extracted from structure Heemraadsingel confirmed the presence of deleterious ASR, whereas the concrete in the tunnel at HertogJanstraat *possibly* contained some alkali-reactive grains [see 27]. The other two structures were not petrographically assessed.

Structure Wolput was reinvestigated early summer 1994, when the Dutch road authorities were planning partial rehabilitation of the structure. Ten cores were extracted from both cracked and uncracked areas spread over the entire structure, for petrographic thin section analysis. The surface cracking was observed to be confined to certain parts of the structure, leaving large areas seemingly unaffected. However, all thin sections contained either reactive aggregate grains and/or silica-gel filled cracks, without exception, a sign that alkali-reactive aggregate grains at least were close by [28]. Thus, the one justified conclusion had to be that ASR was present in all main parts of the structure. As a consequence, the strength of the concrete and the structure as a whole had been reduced by ASR-induced cracking to some degree which could not yet be quantified.

According to personal communication with the Dutch road authorities, Wolput was not the worst (ASR-) affected structure [29]. There were quite a few more significantly affected structures, and more importantly, with the knowledge available at the time, it was impossible to make a reliable estimate of the remaining strength. In 1995, this led to the installation of an entirely new CUR-committee C106 on "Structural aspects of the alkali-silica reaction in concrete structures". Its main objectives were defined as:

1. Collecting knowledge on and experiences with ASR regarding structural aspects and (remaining) service life,
2. Drafting of procedures for the assessment of potentially ASR-affected structures,
3. Publication of these procedures as an official CUR Recommendation.

Technical input for CUR-committee C106 was provided by a number of pilot studies on structures Heemraadsingel, Wolput and Vlijmen-Oost, commenced in 1996. These included detailed structural assessments [30,31], as well as detailed petrographic analysis and subsequent geochemical characterization [23,25]. Data on other structures with confirmed deleterious ASR (e.g. KW5 at Zaltbommel) and their respective concrete compositions were made accessible to the committee as they became available. Some of the obtained results have been published at the 11th International Conference on Alkali-Aggregate Reactions in Concrete (ICAAR) that was held June 11-16, 2000 in Québec, Canada [32, 33, 34].

The 1992 ISE-procedure for the assessment of potentially ASR-affected structures [35] was taken as a starting point by the C106 committee, and compared with Dutch building and construction practice, and ASR-experience [36]. In fact, much of the research done by this committee C106 contributed to the data on existing structures for the revision of CUR Recommendation 38 on ASR-prevention (see below). Additional experimentation included four point load bend testing on full scale beams cut from actual ASR-structures scheduled for demolition, expansion testing, petrographic assessment of a range of aggregate materials, crack fabric assessment of a great number of extracted cores, etc, etc. All acquired results were collated and transformed to official guidelines for the assessment and appraisal of existing ASR-suspect structures, published in June 2005 as CUR Recommendation 102 [37], with a supplementary background report [38].

10.3 Recent Dutch developments in aggregate assessment

Recent developments in the assessment of aggregate materials, with respect to petrographic methodology and the drafting of procedures and criteria, are based on data on concrete from existing structures that became available through th3e activities of CUR committee C106 (see above). In March 2000, CUR established a new research committee VC62 entitled "Alkali-silica reaction in concrete", for the revision of existing CUR Recommendation 38 on ASR-prevention, with a supplementary report providing background details, and initially scheduled for publication before June 1, 2001.

The committee's project plan was certainly ambitious and was to address a range of ASR-related issues:

1. Draft an overview of available ASR-related knowledge, to help specify further research topics;
2. Characterization of the alkali-reactivity potential of Dutch aggregate material,
3. Assessment and characterization of field structures in terms of CUR Recommendation 38,
4. ASR-mitigation by added blast-furnace slag and/or pulverized fly-ash, and
5. Alkali-attribution from internal and/or external resources.

The overview as intended in the first point has been published in a report [39].

In February 2001, a one-day workshop followed by a one-day seminar was held at the Technical University of Delft, organized by the Dutch Ministry of Transport. The workshop critically assessed the reliability of (point) counting methods and counting statistics, reliability of sample extraction and sample size procedures and methods, and the identification of potentially alkali-reactive aggregate constituents. The subsequent seminar informed authorities and executives from the aggregate producing industry on the outcomes of the technical workshop sessions the day before, indicating the need for further research and discussion to reach general consensus. A second two-day workshop was organized on 2-3 October 2001, with participants from all over Europe, USA and Canada. The workshop also included a half-day guided tour to a number of Dutch concrete structures with ASR. Discussions addressed the applied different criteria for the minimum content of blast-furnace slag and/or pulverized fly-ash and/or the maximum alkali-content, and their combinations.

The knowledge acquired by VC62 has been collated in CUR Recommendation 89 to prevent future ASR, published in June 2002 [40]. The supplementary background report [41] provides suggestions for further research, reflecting a number issues from the discussions of the VC62 committee for which no consensus was reached. The issues directly pertaining to CUR recommendation 89 include the reliability of the RILEM AAR-3 concrete prism expansion test for structures with a design service life of over fifty years, the ingress of external alkalis from de-icing salts or sea water, and whether the use of limestone powder in e.g. self-compacting concrete mortars, has any significant effect on deleterious ASR. Apart from these, a number of further suggestions are provided referring to more fundamental issues from mineralogy and geochemistry [p46 in 41].

A number of the issues specified above are dealt with in the upcoming 1st revision of CUR Recommendation 89, expected to appear in print medio December 2006.

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11 Norway

The following Chapter has been reviewed by Børge Johannes Wigum, Hönnun Consulting Engineers.

11.1 Introduction

During the years since 1988, AAR has been recognised as a concrete durability problem in Norway. The reaction is of the slow/late-expansive type developing over decades and takes a minimum of 15-20 years to appear in concrete structures. AAR in Norwegian structures has also been caused by a high content of alkalis in Norwegian cements (during the course of decades). AAR in Norway has been observed under humid conditions in several dams, hydro-electric power plants and bridges [1, 2].

Wigum [3] presented in 2006 a review of the research carried out in Norway during the last 15 years, parties involved and the results and conclusions achieved. An assessment of the state-of-the-art is made along with considerations about future path to follow for the Norwegian research community.

11.2 Current Guidelines

The Norwegian Concrete Association produced in 1996 a publication, NB21 [4], providing guidelines to combat AAR in Norway. The publication was not refereed to by any standard or other formal regulation. Consequently, it had no formal status, and the use of it was voluntary. Despite of this, NB21 was refereed to in most of the Norwegian construction projects and served in general as the Norwegian regulations with respect to production of non-reactive concrete.

Based upon national- and international research work since 1996, a revision of the NB21 publication started late 2002 and was finalized in 2004 [5]. In addition, the Norwegian test methods along with requirements to laboratories were published in a new publication, NB32 [6].

An English summary of the NB21 publication has been presented by Dahl et al. [7]. The new version of the NB21 publication has now a formal status as a harmonised normative reference document to the new European concrete materials standard, NS-EN 206-1 [8]. The NB21 is considered as a key element in the Norwegian system for preventing AAR.

In an article by Jensen [9] the NB21 publication is assessed. By statistical evaluation he claims that some of the requirements in the publication are very restrictive and conservative and even higher than the uncertainties of the methods. This is particularly the case for the petrographic analysis. Consequently, questions are raised about the significance of the method in order to classify Norwegian aggregates regarding AAR. He argues that the concrete prism method is probably the best method to assess Norwegian concrete aggregates according to AAR, and provides the best correlation with field experience. The latter assertion is in agreement with the opinion of the Norwegian revision committee for NB21, and thus this method overrules the other methods described in NB32 and refereed to in NB21. However, the committee is of the opinion that the Norwegian petrographic method is a good and safe tool for assessing potential reactive Norwegian aggregates, under the supposition that the requirements to the petrograph and the laboratory given in NB32 are followed.

11.3 Current test methods and critical limits

Evaluation of material parameters regarding effect of AAR in Norway is since 2004 based upon three different test methods; the Norwegian petrographic analysis, the Norwegian accelerated mortar bar test and the Norwegian concrete prism test [5].

The Norwegian petrographic analysis – This method is a compulsory first step to evaluate the reactivity of aggregate types. The test is carried out by sieving a sand sample into two fractions (1-2 mm and 2-4 mm), respectively by crushing and sieving a coarse aggregate sample into one fraction (2-4 mm). The sieved samples are embedded in an epoxy resin, which allows the preparation of thin sections for petrographic examination. Two thin sections (25 x 35 mm) are made with particles in the fraction 2-4 mm and one in the fraction 1–2 mm. Approximately 1000 points are counted in each fraction. The content of potentially alkali reactive rock types in an aggregate is obtained by calculating an average of the results from both fractions, and is given as volume percentage of alkali reactive rock types, ambiguous rock types and innocuous rock types (see Table 11.3-1), whereas the critical reactive component is the summation of alkali reactive rock types and ambiguous rock types. According to the method description the reactivity of the particles as a whole are evaluated. However, there are some exceptions from this procedure, e.g. if a mylonite zone occurs in a granite grain. Then the mylonite is regarded as a mylonite, while the rest of the particle is regarded as a granite. The petrographic analysis should be performed by an experienced and approved petrographer [6]. This is important because Norwegian rocks are very varied and hence often difficult to identify and classify correctly.

The Norwegian petrographic method is in agreement with the RILEM AAR-1 method [10, 11]. The accuracy of the method has been examined by Wigum et al. [12].

In order to make judgment regarding AAR of the aggregates tested by the petrographic analysis, some adjustments of the results are required in the new publication [5]. A comparative value, *Sv*, is calculated. The calculation includes:

- Use of a "serial factor", i.e. a weighted average is obtained from all the 6 last individual petrographic analyses.
- In order to take into account the fact that coarse aggregates have proven to be more harmful than sand aggregates, a "grain size factor" is applied. For fine aggregates (0/4 mm and 0/8 mm) the factor is 1.0, while for coarse aggregate (8/16 mm and 16/22 mm) the factor is 2.0. For fine coarse aggregate (2/8mm and 4/8mm) the factor is 1.5.
- Finally a safety margin is added in order to take into account the number of analyses that form the basis for the weighed average value.

If the calculated *Sv* is less than the critical limit (see Table 11.3-2), no further documentation is required, i.e. the aggregate is considered to be non-reactive and may be used in any concrete mix.

The Norwegian accelerated mortar bar test – The test is carried out using mortar bars (40•40•160 mm) exposed in 1N NaOH at 80oC for 14 days. The method is mostly in agreement with the RILEM AAR-2 method, but European standards (NS-EN) are followed for sieving, conditioning and moulding. As the mortar consists of a given grading the method is not able to evaluate the reactivity of different aggregate fractions, i.e. the experience is that a sand and a coarse aggregate from the same deposit give similar expansion values. This has been accounted for by differentiating the critical limits as shown in Table 11.3-2, i.e. since the coarse aggregates have proven to be more harmful than sand aggregates a lower limit is applied for coarse aggregates.

The Norwegian concrete prism test - The test is carried out using concrete prisms with dimension 100•100•450 mm. The prisms are stored in 100 % RH at 38^{o}C in small containers, similar as described in the Canadian (CSA) and American standard (ASTM). The critical expansion is measured after one year. The test may be used for testing a sand, a coarse aggregate or a combination of both. When a potential reactive fine or coarse aggregate is tested, it shall be combined with a specified non-reactive coarse or fine aggregate, respectively, in a 60/40 mix representing the practical "worst case", i.e. 60 % of the potential reactive aggregate shall be applied.

Table 11.3-1: Classification chart for alkali-reactivity of Norwegian rock types [12].

Class 1. **ALKALI REACTIVE ROCK TYPES** *(Documented in structures)*	**Class 2.** **AMBIGUOUS ROCK TYPES**	**Class 3.** **INNOCUOUS ROCK TYPES**
1. SEDIMENTARY ROCKS **Sandstone** **Arkose** **Quartz sandstone** **Claystone** *(including shale)* **Siltstone** *(including shale)* **Marlstone** *(including schistose and/or metamorphic)* **Greywacke** *(also metamorphic)* ***Sedimentary features should be observed.***	**5. AMBIGUOUS** *Examples*: **Quartzite/quartz schist** **Rock types with quartz** *(Modal quartz >20vol%)* **Limestone** *(contaminated with dispersed fine grained quartz)* **Hornfels** *(quart-bearing)* **Mylonites** low in free quartz (1-5vol%)	**6. MAFIC ROCK TYPES** **Basalt** **Greenstone** **Gabbro** **Amphibolite** ***All types of variations of the rocks, also metamorphic***
2. MYLONITE/ CATACLASITE **(Containing free quartz)** **Mylonites** **Cataclasites** **Mylonite gneiss**	*All quartz-containing rock types could be potentially reactive. This however depends on petrological parameters such as grain size of quartz, degree of deformation and other microstructural features.* *Various types of quartzites have reacted in concrete.* *Microcrystalline quartzite (quartz grains <60 µm) should be classified as alkali reactive.* *Quartzite with quartz grains <130 µm, should be classified as ambiguous.* *Quartzite with quartz grains > 130 µm, should be classified as innocuous, even if the quartzite contains "strained" quartz.*	**7. ROCK TYPES CONTAINING QUARTZ** **Granite/Gneiss** **Quartzite/quartz schist** **Mica schist**
3. ACIDIC VOLCANIC ROCKS **Rhyolite** **Quartz keratophyre**		**8. FELDSPATHIC ROCK TYPES**
4. OTHER ROCK TYPES **Microcrystalline quartzite** **Phyllite** **Quartz schist**		**9. OTHER/ UNIDENTIFIED** **Limestone** *(pure)* and **marble** Other non-reactive *(also single crystals)* Porphyry Quartz-free mylonites
Typical grain size of quartz; < 60 µm **Exception: Sandstone**	**Typical grain size of quartz; < 130 µm**	**Typical grain size of quartz; > 130 µm, or quartz not present**

The critical limits presented in Table 11.3-2 are based on the assumption that the concrete prism test is capable to take into account the effect of different reactivity of various grain sizes. Consequently, the same limit is applied for fine and coarse aggregates (0.040 % after one year of exposure). However, for blends of aggregates a slightly higher critical limit is specified (0.050 % after one year of exposure). The reason for this is that in real life an aggregate classified as “non-reactive” may give a certain contribution to the overall expansion.

Table 11.3-2: Overview of critical limits for the three Norwegian test methods for documentation of alkali-reactivity of single aggregates or blends of aggregates [5].

	Critical limits for the three Norwegian laboratory test methods[1]		
Documentation of	Petrographic analysis, Sv (adjusted results)[2]	Accelerated Mortar bar method[3]	Concrete prism method[4]
Fine aggregate and blend of fine	20.0%	0.14%	0.040%[5]
Coarse aggregate and blend of coarse		0.08%	0.040%
Fine coarse aggregate		0.11%	n/a
Blend of a fine- and coarse aggregate, where the fine or coarse is alkali-reactive	20.0%[6]	0.11%	0.050%

1) A single aggregate or a blend of aggregates shall be classified as innocuous if the values obtained are lower than the specified critical limits.
2) *Sv* shall be compared with the critical limit.
3) The measured expansion after 14 days of exposure shall be compared with the critical limits.
4) The measured expansion after 1 year of exposure shall be compared with the critical limits.
5) A fine aggregate or a blend of fine shall be tested with a coarse non-reactive reference aggregate. A coarse aggregate or blend of coarse shall be tested with a fine non-reactive reference aggregate. The binder used shall have an alkali content of 5.0 kg/m^3 Na_2O eq.
6) A maximum of 15% of the calculated value is allowed to come from the coarse aggregate.

11.4 Performance testing

The alkali-reactivity of various types of aggregates, binders and concrete recipes can be documented by performance testing using the Norwegian concrete prism method. Binders shall be tested in concrete with a specified highly reactive Norwegian aggregate. The acceptance criteria for different types of binders and concrete recipes are presented in Table 11.4-1. A performance test shall be based on one or more batches. If based upon more than one batch, test results shall be plotted in an expansion versus alkali content-diagram as illustrated in Figure 11.4-1. By assuming a linear relationship between concrete prism expansion and alkali content, a limit of maximum accepted alkali content can be obtained. A safety factor of 0.2 kg Na2O eq./m^3 is required to be subtracted to the limit.

Table 11.4-1. Maximum permitted expansion values for the Norwegian concrete prism test [5].

Documentation of	Concrete containing pozzolanas or slag?	Time of exposure	Maximum permitted expansion value after one year of exposure
CEM I binders, CEM II/A-V and CEM II/A-D, in addition to potential added silica fume and concrete recipes with these binders	No	1 year	< 0.050%
	Yes	1 year	< 0.030%
All other types of binders and concrete recipes with these other types of binders	Yes and No	1 year	<0.030%
	Yes and No	2 years	<0.060%

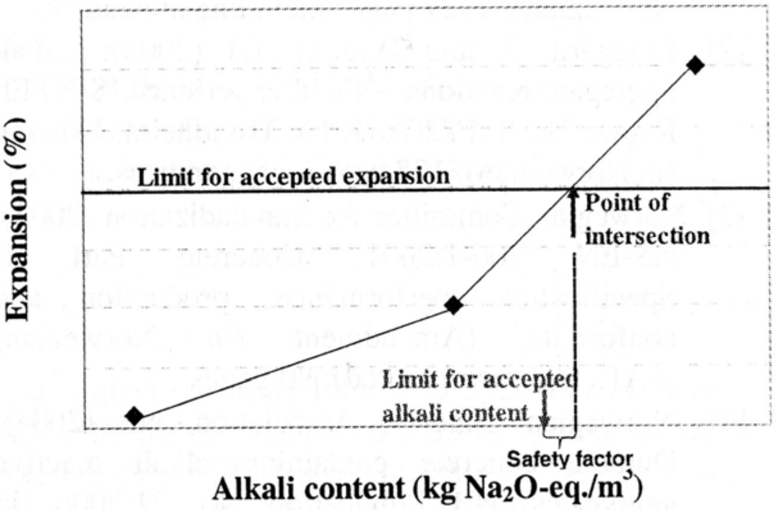

Figure 11.4-1. Principle diagram for determination of acceptance limit for alkali content [5].

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12 Poland

The following section is based on mr. Góralczyk's unpublished paper – cf. reference /1/. This chapter has been reviewed by Stefan Goralczyk, Instytut Mechanizacji Budownictwa I Gornictwa Skalnego (IMBiGS).

In order to prevent destruction of concrete elements under the influence of alkaline reactions, test methods and methods of identification of alkaline reactivity of aggregates were elaborated and quantitative criterions for mineral aggregates used in concrete production were determined.

Standard PN - 86/B – 06712 „Mineral aggregates for concrete" establishes technical requirements and obliges producers to test this qualitative parameter. Different resolutions concern cement attestation, particularly detrimental alkalis content. Standard PN-EN 197-1:2002 „Cement. Part 1: Constitution, requirements and conformity criteria for common cements" does not determine the obligatory of designation of alkalis content. In practice, the producer of concrete mixture doesn't know the alkalis content in the cement that he uses. This situation must not be tolerated any more. Since dozens of years the advanced quantity of alkalis in cement from most of polish cement plants is observed, according to statistics from Instytut Mineralnych Materiałów Budowlanych / Kraków / Institute of Mineral Materials for Constructions/ (Table 11.5-1).

Table 11.5-1: Alkalis content in selected Polish cements of common use.

No.	Type of cement	Alkali content [% Na_2O]
1	CEM I	0.14-1.26 dominant content 0.85
2	CEM II	0.41 – 1.55 dominant content 0.80 – 0.90
3	CEM III / metallurgical /	0.46 – 1.25
4	CEM IV	1.41 – 1.62

In the process of cement production the alkalis are introduced into the oven with raw materials, mainly with clay minerals and in a small amount with carbon. In the temperature above 1100°C they sublimate, pass through a gas phase and they react with other components, particularly with chlorine and sulphur dioxide.

In colder areas of the oven alkalis partially condense in the sulphate and chloride form and they comeback to the parching area. In the wet method of clinker production about 30% of alkalis are off-taking from the oven with exhausting gas. In the new dry method most of alkalis in the sulphate form condense on the charge grains and comeback into oven, and only 10% is off-taking outside. In order to reduce the alkalis content in clinker shunting of a part of exhausting gas (about 5-10 %) from the oven, mixing with cold air, condensation on the dust grains and precipitation in hydrocyclon are used.

In order to reduce the alkalis content in clinker into the admissible level of 0,6% it is necessary to shunt 10-15 % of exhausting gas. It requires the construction of additional installation, and causes increase in energy consumption (till 65 kcal/kg of clinker and 8 kWh/Mg of clinker) and considerable rise of production costs. From above it can be stated that in the production of cement there are technical possibilities of reducing alkalis content in the final product. However, with regard to mentioned high costs of their application they are used in practice very sporadically and it concerns only quantitatively small production of special cements.

The aggregate, the second component of concrete, shows the selective susceptibility on the alkaline reactions, depending on mineral composition /mineral group content/ and potentially reactive rocks. Elimination of these components in the process of aggregate modification and refining is limited. It concerns elimination of clay impurities, weathered mineral grains and limitation in the product of aggregates fraction in which reactive compositions are cumulating, for example: commonly occurring white marl limestone. They characterise by lower volume density than other components. It enables separation of these grains in technologies of refining aggregate using hydraulic classification /for example: technology of Allmineral company from Germany. Technology of aggregates modification at an angle of elimination reactive components is a separate problem, which however cannot be correctly solved without identification of reactive components in aggregates in Poland.

12.1 Test methods

Tests on alkalis-aggregate reactions were commenced in Poland in the second half of sixties. At that time the researches were based on American methods, as well as quantitative-qualitative criterions established by ASTM standards. Test researches and standardisation works aiming at elaborating test methods and establishing quantitative criterions for aggregates reactivity effect were started in 1978 on Centre of Investigation and Development of Aggregates for Building Industry /actually Institute of Mechanised Construction & Rock Mining /. The elaboration of methodology for aggregate reactivity testing in PN-79/B-06714/34 "Mineral Aggregates. Testing. Determination of alkaline reactivity" was the result of these works. This standard was then amended taking into consideration the results of researches executed in 1985-1990. This is an adaptation of American standard ASTM C 227 which consist in determination of destruction changes caused by alkaline reactions in concrete prisms made of aggregate and cement with advanced alkali content - 1,2 % in (in Na_2O). Conditions of storing beams during one year as well as method of linear measurement are similar as in ASTM.

Limit value of linear changes of specimens cannot exceed 0,1 %, and it should not occur colour changes on the specimens surface, as well as efflorescence out-flows, scratches, chips etc. In the amended /in 1990/ PN-91-06714/34 standard in order to improve research and to limit accidental errors in designation, many additional information in described methodology was introduced. In amended standard alkaline reactivity test is executed only for aggregates complying the requirements of PN-86/B-06712 "Mineral aggregates for concrete".

It is permissible to use cement with the alkalis content other than 1,2 % Na2O in the researches, what makes possible concrete attestation directly at the construction site. After aggregate reactivity is stated, it is possible to stop the test on every stage of measurement, what strongly limit costs and period of research. High costs and long period of testing by the beam method, searching for the most appropriate aggregate attestation method and further particulars in tests on alkaline reactions in concrete were the ground for these researches in the eighties. Two chemical test methods and amendment for standard concerning petrography constitution were the result of these researches.

Standard PN-92/B-06714/46 "Mineral Aggregates. Researches. Designation of potential alkaline reactivity by rapid method".

This method consist in determination of loss of aggregate mass resulting from reacting by sodium hydroxide (NaOH) on aggregate specimen. Aggregate particles are subjected to crushing or superficial softening. The grade of potential alkaline re-activity is determined on the basis of the following results:

- Designation of loss of fine and coarse aggregate mass under sodium hydroxide (NaOH) action
- Designation of fire-stones content with volume weight of 2.0-2.6 g/cm^3

Aggregates with 1 or 2 potential reactivity degree should be recognise as potentially reactive according to Table 12.1-2.

Table 12.1-2: Reactive aggregate criterions according to method from PN-92/B-06714/46.

Feature	**Alkaline reactivity**		
	0	1	2
Loss of mass for given fine aggregate fraction X1 or coarse aggregate X4 under NaOH action	≤ 0.5	> 0.5 ≤ 2.0	> 2.0
reactive fire-stone content X3 *	≤ 3.0	> 3.0 ≤ 10.0	> 10.0
5 X4 + X3	≤ 4.0	> 4.0 ≤ 15.0	> 15.0
Potential alkaline reactivity degrees: 0 – non-reactive aggregate 1 – potentially reactive aggregate 2 – reactive aggregate			

In case of statement that the tested aggregate represent 1 or 2 potential alkaline reactivity degree in order to confirm the final reactivity estimation it should be execute additional beam method test according to PN-91/B-06714/34.

Test method according to PN-88/B-06714/47 "Mineral aggregates. Determination of silica soluble in sodium hydroxide content":

This is the adaptation of Russian standard GOST 8269. This test method consists in determination by weight method the content in aggregate of silica soluble in sodium hydroxide /NaOH/ at 80 ^{o}C temperature in the period of 24 h. This method does not differ from GOST 8269 standard. This is why the qualitative criterion of max. 50 mmol/l of soluble silica in tested specimen can be applied in Poland.

PN-87/B-06714/11 „Mineral aggregates. Determination of petrography constitution":

In the amended /in 1986/ standard the necessity of separation grains from rocks, which because of their occurrence report the possibility of reacting with alkalis, was introduced. In the macroscopic description the weathered grain participation and dominant grain surface character (with weathered traces etc.) should be determined.

Comparing content of PN standard with analogous methods from BS, ASTM standards and taking into consideration the necessities of counteracting alkaline re-activity of aggregates the accuracy of grain petrography distribution is not sufficient. It is necessary, besides macroscopic description, to introduce the notice about necessity of detach in aggregate natural grains deriving from particular rock types and considered as reactive (researches on determination reactive rocks occurring in Poland were executed by Institute of Mechanised Construction & Rock Mining in 1988-89) for example: chalcedony, opal, marls, flint-stones, volcanic glasses, some dolomites etc.)

Methodology of alkaline reactivity of carbonate aggregates

Up to now there was no unequivocal method of designation alkaline reactivity of aggregates from carbonate rocks. Test methods described above with regard to limited repeatability of results cannot be used in determination of alkaline reactivity of aggregates resistance. The alkaline resistance of carbonate aggregates used in production of construction elements is not fully recognised.

Works on this subject were taken up in Institute of Mechanised Construction & Rock Mining in 1993. The identification of alkaline reactivity of certain polish carbonate aggregates was the final effect of these works. The modified ASTM C 1226 standard and own method of beams from rocks were used in these tests. Both methods are rapid, the results can be obtained after 14 days of researches and the specimens were held in 1 N NaOH in the temperature of 80^{o}C. With regard to different character of reactivity of carbonate aggregates (dedolomitization) this problems will be raised in other article.

Assessment and possibilities of reactive aggregates application

Until very recently mineral aggregates susceptible to alkaline reactions tested according to PN-91/B-06714/34 were disqualified and their use for concrete elements production partially or fully limited. At the present time the possibility of using reactive aggregates for concrete with certain restrictions and reservations is permitted.

The aggregates that characterise:

– 1 degree of alkaline potential reactivity according to method from PN-92/B-06714/46 can be used for concrete that are not jeopardised to constant or momentary influence of moisture

– 2 degree of alkaline reactivity (reactive aggregates), can be used for concrete with low-alkaline cements of alkalis content below 0,6% (in Na_2O), and the cement content should not exceed 500 kg/m3 of concrete. Proper pozzolanas additives preventing alkalis-aggregate reaction should be used in reactive aggregates application, according to Instruction ITB no 306 „Prevention of alkaline corrosion of concrete by application of mineral additives" Warsaw 1991 r.

In case of reactive aggregates the instruction mentioned above recommends introduction, instead of cement, one of two pozzolanas additives into concrete:

– 15% of silica dust

– 10% of silica dust in connection with 20% active volatile ashes.

Portland poor cement and rule not smaller than 35 should be used during application of mentioned additives. Both pozzolanas additives do not worsen qualitative concrete parameters and prevent against:

- Stains, silica efflux, chips and other external symptoms of concrete alkaline corrosion, even by using high-alkaline cements
- Concrete swelling

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13 Portugal

The following chapter has been prepared by António Santos Silva and Arlindo Gonçalves of Materials Department of Laboratório Nacional de Engenharia Civil.

13.1 Introduction

Since the nineties research on the diagnosis and preventive measures against AAR in concrete was carried out at Laboratório Nacional de Engenharia Civil (LNEC). Some concrete structures (mainly dams) have began to show evident signs of ASR occurrence, and in LNEC, quite lot of work was performed aiming at not only the diagnose of the declared cases of damaged structures but also the evaluation of the factors which can conditionate ASR in Portugal [1,2]. Due to a better knowledge of this pathology by the Portuguese engineers and owners, the number of concrete structures affected by ASR in the last years is continuing to increase. This situation has pointed the need to study the applicability of the existing methods in Portugal for evaluation of the alkali reactivity of the aggregates and the research related to the preventive measures for ASR inhibition in new concrete structures.

As a result of the studies performed [3] the conclusion is that the test methods employed in Portugal until 2004 are inappropriate for evaluation of the alkali reactivity of the aggregates. These tests include the ASTM C 227 (Mortar bar test) and ASTM C 289 (Chemical method). Due to this research the first Portuguese Recommendation for ASR inhibition in concrete was prepared in 2004 by LNEC [4], and it's a normative document through the National Annex of the Portuguese Standard NP EN 206-1 [5].

13.2 Specification against ASR

The Portuguese specification identifies the rocks and minerals that occur in Portugal and which are potentially reactive to alkalis; it defines the methodologies to assess the reactivity of aggregates and the possibility of the concrete mixtures to develop ASR. Furthermore, it establishes the measures to be taken to avoid the occurrence of expansive reactions, either by alkali-silica reactions or by delayed ettringite formation. This specification does neither address alkali-carbonate reactions, because their occurrence is not known in Portugal, nor the other internal sulphate reactions because, in principle, the compliance with the applicable product standards prevents their occurrence.

13.3 Assessment of the aggregate reactivity

The assessment of a certain production of concrete aggregates is done in accordance with the type(s) of involved rock(s):

- Alluvial aggregates, by each aggregate size;
- Aggregates from sedimentary rocks: by type of aggregate using whatever size, provided that the continuity of the formation is geologically demonstrated;
- Aggregates from eruptive and metamorphic rocks: by geologic stratum, using whatever size.

The general procedure employed assess the reactivity of aggregates is presented in figure 1. According this methodology the aggregates are classified in three categories:

- Class I: non-reactive aggregates;
- Class II and III: aggregates potentially reactive. The probability of occurrence of expansive reactions with Class III aggregates is higher than the one with Class II aggregates.

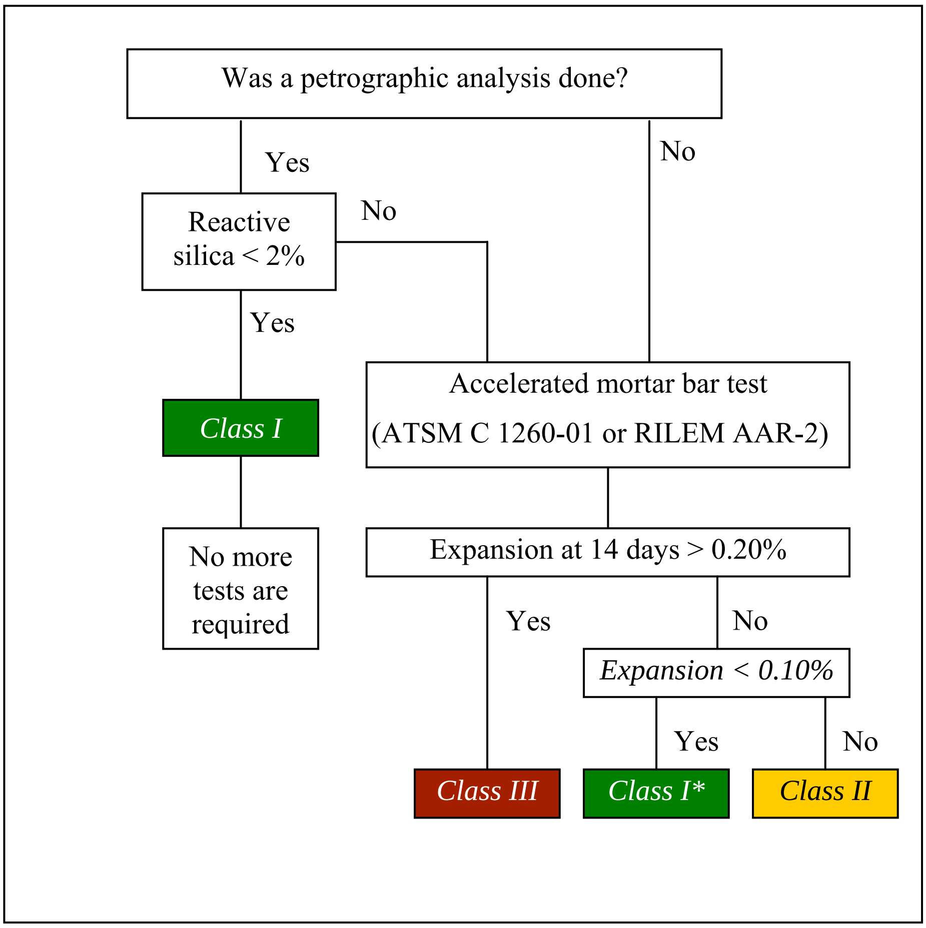

Figure 1 – Methodology to assess aggregate reactivity.

* Some granitoids defined as Class I have locally demonstrated to be potentially reactive to alkalis. For this type of aggregates, the application of the methodology mentioned in figure 2 is recommended.

13.4 Preventive measures

For the occurrence of alkali-silica reactions, it is necessary that the following conditions are simultaneously observed:

- Enough alkalinity in concrete pores;
- Eritical content of reactive silica;
- Enough amount of water.

Therefore, to prevent the development of alkali-silica reactions, every action shall be done to avoid the occurrence of at least one of the conditions below. Nevertheless, the decision to operate on one or more of these conditions depends on various factors, and, sometimes, no type of action may be necessary to avoid the occurrence of the reaction.

Control of alkalinity of the solution of concrete pores

The alkalinity of the pore solution can be controlled either by limiting the alkali concrete content or by using appropriate binders.

The alkali content of the concrete is equal to the sum of the alkali content of its components, excluding fly ash and ground granulated blast-furnace slag, if in percentage higher than 30% of the total binder. The maximum admissible amount of soluble alkalis is indicated in Table 1, as a function of the reactivity class of aggregates.

Table 1 – Maximum content of soluble alkalis in the concrete

Reactivity class of aggregates	Maximum alkali content (kg/m^3 of concrete)
I	Without requirement
II	≤ 3.0
III	≤ 2.5

The binders consisting of cements (NP EN 197-1) and of cement mixtures and admixtures (LNEC Specification E 464) provide effective protection against the occurrence of expansive reactions caused by Class II aggregates, as long as the proportion of the components listed below is not less than:

- 10% if only silica fume is used (D);
- 30 % if only siliceous fly ash is used (V);
- 50 % if only ground granulated blast-furnace slag (S) is used;

If more than one of these components is used, the condition $5D + 5/3V + S \geq 50\%$ must be fulfilled.

When class II aggregates are used or, in the cases where other binders not complying with the former conditions are used with class II aggregates, the suitability of these materials to control the ASR must be assessed in the concrete prism test (see figure 2).

Control of the content in reactive silica

The mixture of different types of aggregates with a total reactive SiO_2 content < 2% can only be considered of class I if each aggregate in the mixture has reactive SiO_2 < 2%. The aggregate mixtures are also considered in the class I if the classification has resulted from the accelerated mortar-bar test. In all the other cases, the reactivity of the mixture must be assessed by means of the mortar-bar test (see Fig. 2) using the mixture proportioning indicated in standard ASTM C 1260 or in RILEM AAR-2 method. The three larger fractions must be prepared from the coarse aggregate, and the two smaller fractions from the fine aggregate. If the mixture is considered as reactive, then the methodology indicated in Fig. 2 should be adopted. If the reactivity is confirmed in the tests on concrete, the preventive measures indicated above should be taken.

Humidity control

The control of the humidity access must be considered in the stage of design using appropriate covers. The protection techniques must be designed so as to facilitate their inspection and maintenance.

In view of the possibility of occurrence of expansive reactions in large mass concrete, even when protected from humidity, another prevention method must be simultaneously used.

Modification of expansive properties of the gel formed in the ASR

The expansive character of the gel resulting from ASR can be modified with the inclusion of lithium salts in the batching water. The use of lithium nitrate is recommended in the minimum dosages as follows:

- With class II aggregates: 3.75 kg of a 30% solution of $LiNO_3$ per kg of $(Na_2O)_{eq.}$ in the concrete;

- With class III aggregates: 5.95 kg of a 30% solution of $LiNO_3$ per kg of $Na_2O_{eq.}$ in the concrete.

Assessing the susceptibility of aggregate and concrete mixtures to the ASR

If none of the preventive measures indicated above is used, the aggregates included in classes II or III may also be used in the concrete, either mixed or not with class I aggregates, provided that, by following the methodology indicated in Fig. 2, the aggregate mixture, or a specific concrete mixture, has been defined as non-reactive.

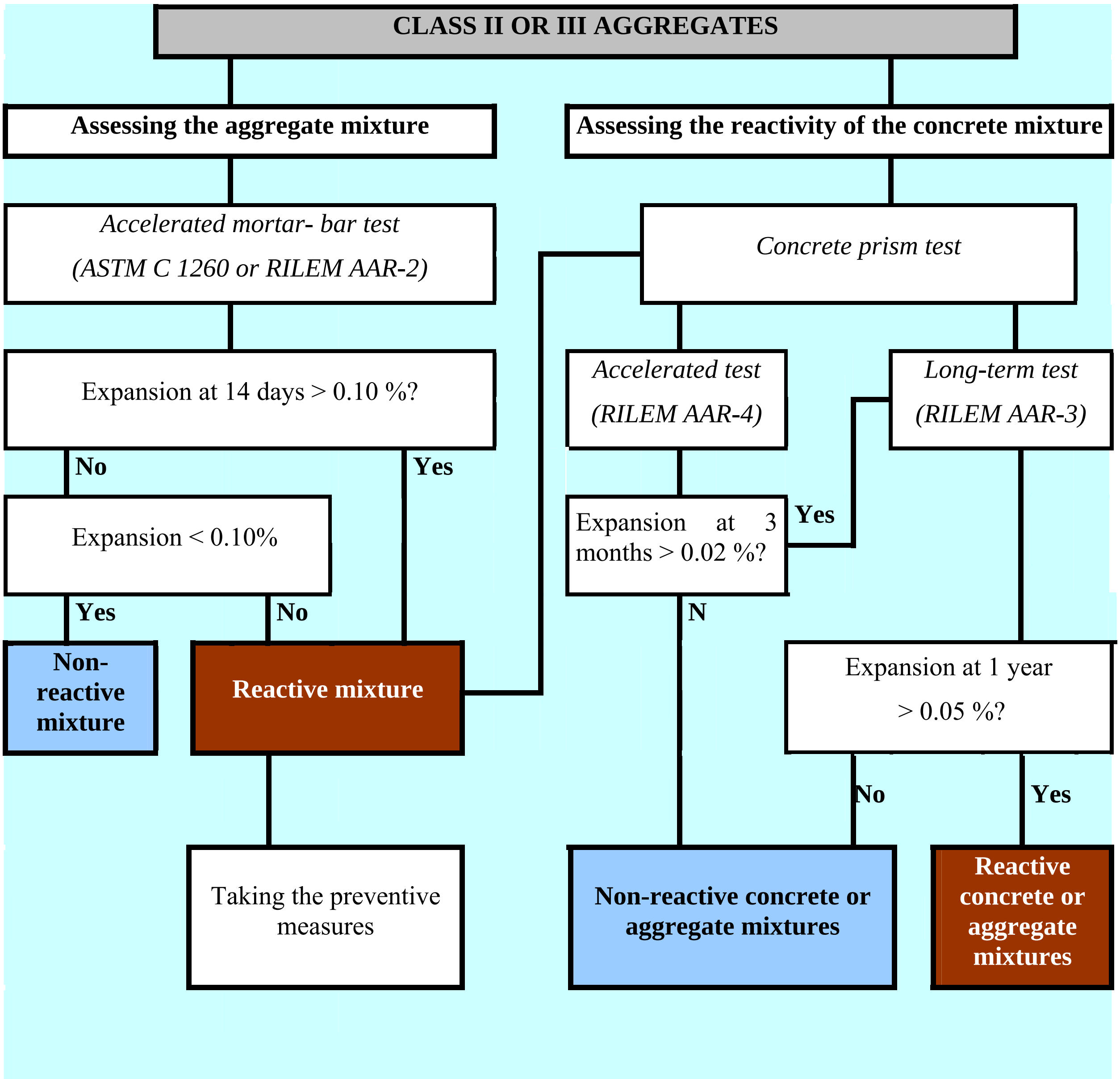

Figure 2 – Methodology for assessing the susceptibility of an aggregate or concrete mixture to develop alkali-silica reactions.

13.5 References

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2. Braga Reis, M.O.; Silva, H.S.; Santos Silva, A. (1996). AAR in Portuguese structures. Some case histories. Proceedings of the 10th International Conference on Alkali-Aggregate Reaction in Concrete, Melbourne, Australia, p. 93-100.
3. Santos Silva, A. (2005). Concrete degradation due to alkali-silica reaction. The use of fly ash and metakaolin for its prevention (in Portuguese), University of Minho, p. 340.
4. LNEC E 461 (2004). Concrete. Methodologies for avoiding internal expansive reactions (in Portuguese), LNEC Specification, p. 7.
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14 Spain

The following Chapter has been reviewed by Angel Lopez-Buendia, AIDICO, Spain.

In the 70's, after having observed that the alkali-aggregate reactivity did not happen with the Spanish aggregates admixtures, the RC-75 report did not mention any tests to evaluate it. During this period, Spain had a boom in the construction sector for the increase in residential areas demanded to cope with the enormous tourist growth. This growth did not only affect the construction companies, but also the construction material suppliers, that had to solve the market demand. As a consequence, new quarries were opened in diverse geographical places, near by the areas where the construction works were carried out, to supply the construction companies.

Few years later, new pathologies in buildings were found in the Western part of Spain. The final conclusions in those studies, established the alkali reactivity of the aggregates as the responsible cause for the damages in several buildings.

There are two main major areas of aggregates family affected by the geological structure of Spain. The granitic domain, in late sense, is formed by the Hesperic massif affecting to western part, covering part of the north and centre of Spain. The other part is formed by carbonate rock domain, affecting to the alpine relieves geographically distributed in the eastern part covering part of the South and eastern north. Regarding this simplified distribution, most of cases of alkali-silica reaction has been detected in the "granitic Spain" and the cases attributed to carbonate has been found in the "carbonate Spain".

14.1 Spanish standards and alkali-carbonate

In 1998, the EHE included again alkali reactivity tests for aggregates. Following the Spanish standard, the silica, silicate and carbonate aggregates have to be chemical and physical tested following the procedures of the UNE 146507EX, UNE 146508EX and UNE 146509EX, respectively.

On the one hand, the results obtained by the chemical tests have been proved of not being decisive. Some of the aggregates that have given positive results through the chemical tests, have not showed any expansive reaction by the physical tests, being classified as non reactive aggregates. So none concluding results can be obtained through chemical tests. Therefore another approach is needed to estimate the sample reactivity.

The proceeding of the structural concrete –EHE- actually demands a strict fulfilling of the standards, which are focus on the detection of aggregates with a potential reactivity between Aggregate-Alkalis. The Spanish standards establish several tests, which are described on the methods UNE 146507 EX (chemical test for a quick response of the own aggregates), UNE 146508 EX [1] (Physical test accelerated in mortar), and UNE 146509 EX [2] (Physical test accelerated in concrete).

The application of the method described on the standard UNE 146507 EX provides wrong reactive aggregate results, which classified inert aggregates as "potentially active aggregates". This point is manifested applying the standards UNE 146508 EX and UNE 146509 EX.

The non-fulfilling of the standard UNE 146507 EX automatically refuses the use of the aggregate for the elaboration of structural concrete, even when the aggregates achieves the standards UNE 146508 EX and UNE 146509 EX. It determinates the interruption of the exploitation or even the abandoning of quarries, and a deficient organization of the exploitation. The development of a predictive methodology would allow a prevention of future pathologies.

There is a strong controversial around the occurrence of the pathology ACR due mainly to the cryptic symptoms, which is usually unclear with respect to the genesis of this pathology.

According to the chemical test described in the standard UNE 146507 EX, the aggregates has to be grinded to a particle size smaller than 0.16 mm; afterwards, it is dissolved into ClH and Ammonium chloride after UNE-EN 196-2. The solution is valued with EDTA, EGTA and DCTA, which allows obtaining the contents of Al_2O_3, CaO and MgO. Finally, the content of each component is calculated according to the points 13.11, 13.12, and 13.13 of the standard UNE 146507, respectively. After the obtaining of the contents, the rate between CaO/MgO is calculated. This rate is represented with the percentage of Al_2O_3. It classifies the aggregates as: Non-reactive aggregates or potentially active aggregates (Figure 14.1-1).

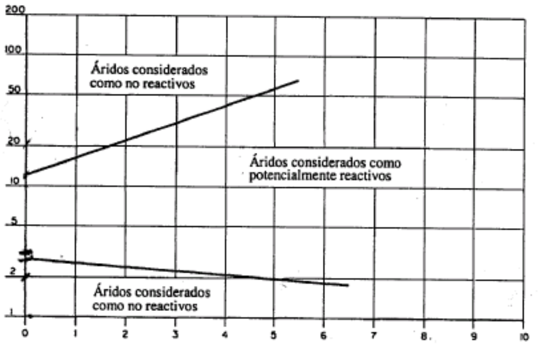

Figure 14.1-1. Chemical test to detect potentially active aggregates. In X axes represents the content of percentage of Al_2O_3 , the Y axe represent the rate CaO/MgO.

This graphic determinates the possibility of a reaction aggregate-alkalis occurs. The aggregates are considered potentially active when the value of the rate CaO/MgO and the value of the percentage of Al_2O_3, are between the values delimited by the two lines of the graphic.

According to the mortar bar test described on the standard UNE 146508, the detection of the pathology aggregate-alkalis is based on the change of length (caused by the expansion) of the mortar made with the studied aggregate. The mortar is introduced into water 80ºC during 1 day and into NaOH 1M 80ºC during the next 14 days. The measures of length provide an idea of the reactivity of the study aggregate:

- When the expansion is less than 0.1%, the aggregate is considered as non-reactive.
- When the expansion is higher than 0.2%, the aggregate is considered as potentially active.
- In case, where the expansion is between the described values, the measures have to continue until 28 curing days. If during these 28 days, the value of the expansion is still between the previously described values, additional information would be required for determining the reactivity of the aggregate.

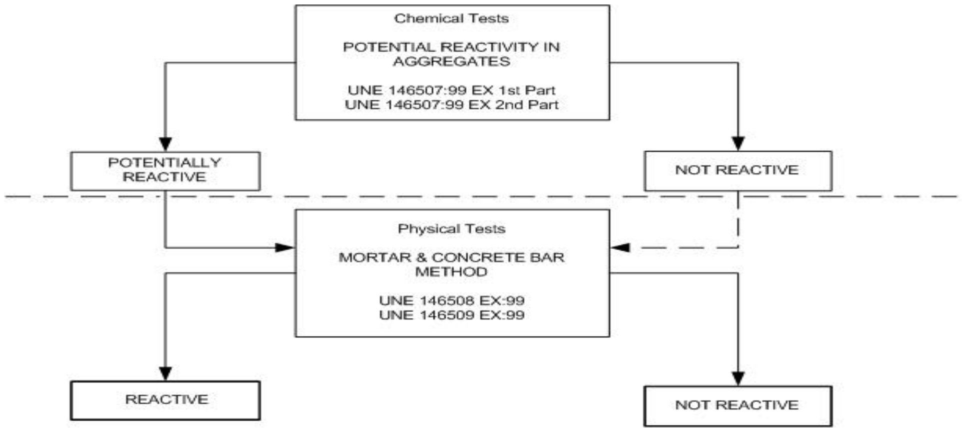

Figure 14.1-2. Proceeding for determining the reactivity Aggregate-Alkalis according to the Spanish Standard.

The chemical test results, on contrary, no always reproduce the reality of the tested aggregate. Some of the tested aggregates according to this method provide positive results, whereas the results were negative according to the physical test. The problem is that for the demonstration of that, test of mortar expansion and concrete expansion during long periods of time (at least one year) are required. During that time, the exploitation of the quarry is stopped. This implies the immediate prohibition of the use of the aggregate for structural concrete, with the evident environmental and economic implications.

The first tests according to the valid standards were based on physical tests [3]. The time required for these tests to get reliable results was too long, causing the introduction in the standard of a new chemical test, which provides quick answers. The tests were divided into two parts: Reactivity Aggregate-Silica and Alkalis-Silicate [4], and, in the other hand, reactivity Alkalis-Carbonate [5].

Thus, an increasing demand on alkali-carbonate reactivity methodologies, to solve the actual problems occurring in constructions, is pushing scientist to find good techniques to afterwards implement them as standardised tests. In this way, López-Buendía et al (2006) [6] studied several Spanish carbonate aggregate and determined that there are petrographic parameters to infer the alkali stability or reactivity risk in silicified, ferric, clayed carbonates with different proportion of dolomite and dedolomite. This additional evaluation test based in some premises of Katayama (2003) [7] and Rilem testing method (Sommer et al, 2000) [8] is nowadays in use as preliminary study to facilitate alkali-carbonate diagnosis.

In conclusion, the actual Spanish standards referred to the alkali aggregate reaction are not accurate enough to classify the reactivity of an aggregate sample. Therefore, further approaches like petrographic techniques should be encouraged in this yield to have a better understanding of alkali aggregate reactions and improve the methodologies to recover buildings affected.

14.2 More significant cases of alkali-silica in Spain

In Spain cases has been attributed to different aggregates, mainly granite, quartzite, granodiorite, diabase, gneiss, schist and even slate. The age of appearing is very variable reporting damages since 5 up to 45 years with strong variations depending on aggregate nature and waterproofing surface treatments. Although some cases in tunnels are knowledge most of the cases are associated to damps.

Most of the cases can be associated to expansive gel formation although the clay fine content appear as important contribution (Soriano, 1981).

Table 14.2-1. Summary of cases associated to alkali-silica reaction.

Damp case	Age when damage appeared	Nature of aggregate with damages		
SE	-	Granite, Diabase, Gneiss, schists with trace of pyrite	Gel production an craking	Repaired
S	5	Granite (undulant quartz content)	Vitreus and microcrystalline gels	Paralyzed due to fly ash content (8%)
Bs	20		Local expansion	Still causing expansion
T	30	Granite, Ortoquarzite with undulant extinction quartz	Non-expansive deterioration	
P	5	Slate	Expansion	Quick reaction
A	23	Granite and granodiorite feldspars plagioclases and undulant quartz with alteration	Local expansion	
B-C	-	Granodioririte,granite with feldspars, quartz and mafic minerals.	Local expansion	
V	20 for quartzite and 45 for granite	Granite, quartzite with undulant quartz	Expansion	

14.3 References

[1] ASTM C295-98. Standard Guide for Petrographic Examination of Aggregates for Concrete.

[2] *UNE 146 508. Ensayos de áridos. Determinación de la reactividad potencial álcali-sílice y álcali-silicato de los áridos. Método acelerado en probetas de mortero.*

[3] UNE 146507-1EX. *Ensayos de áridos. Determinación de la reactividad potencial de los áridos.* Parte 1: Determinación de la reactividad álcali-sílice y álcali-silicato

[4] UNE 146507-2EX. *Ensayos de áridos. Determinación de la reactividad potencial de los áridos.* Parte 2: Determinación de la reactividad álcali-carbonato

[5] *UNE 146 509. Determinación de la reactividad potencial de los áridos con los alcalinos. Método de los prismas de hormigón.*

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[7] Katayama, T (2004), How to identify carbonate rock reaction in concrete, Materials Characterization, 53, 85-104.

[8] Sommer, H., Grattan-Bellew, P, Katayama, T, Tang, MS (2004). Development and inter-laboratory trial of the rilem AAR-5 rapid preliminary screening test for carbonate aggregates, 12th International Conferences of Alkali-Aggregate Reaction in Concrete.

15 Sweden

The following Chapter has been reviewed by Björn Schouenborg, Swedish National Testing and Research Institute (SP).

15.1 Introduction

There are many different rock types in Sweden. Some of these are very reactive, e.g. those in the south-western part of Sweden, Scania. Here many parts of the bedrock are of the same type as in Denmark and in northern Germany, limestone with flint (chert) and in cases porous flint. Consequently, in this part of Sweden the problems with ASR have been the same as in the countries mentioned above. To the north of this area siliceous gneisses dominate the bedrock. In most cases these rocks do not cause any harm. However, locally microcrystalline siliceous rocks (porphyries, metarhyolites) and strongly deformed rocks (cataclasite and mylonites) are reactive if they are combined with a high alkali cement or alkali is added externally. The third type of rock formation, which has been known to cause ASR are the erogenic metamorphic rocks from the Swedish-Norwegian mountain chain, the Caledonides. The known reactive rocks from Caledonides are greywackes, mylonites and some altered sparagmite sandstone. A compilation of the most commonly used concrete aggregate types in the Nordic countries is given in a Nordtest report from 1990 [1].

The most harmful type of rocks are those from Scania. How to assess these types of rocks is best described in the Danish specifications. Therefore, in this part of Sweden most of the aggregates to be used in aggressive environments are taken from parts to the north of the area with flint bearing limestone. The reactivity of the aggregates from the south-western Scania are best assessed by petrographic micro-analysis on this sections, combined with testing according to NT BUILD 295 [2] but for more cycles than prescribed in this Nordic method.

The other type of potentially reactive Swedish aggregates are slowly reacting (fine "grained" siliceous volcanic rocks and mylonites). With a high alkali content of the concrete, it may take up to 15 years for the expansion to reach such a level that the cracking becomes evident. These aggregates very rarely cause severe damage when the concrete aggregates are made up of glaciofluvial gravel, that generally is a mixture of different types of rocks. These types of rocks have a pessimum of 100%, [3]. However, if these types of rocks are used as crushed rocks in combination with a high alkali cement, then the mix may become deleterious. The final expansion is in the same order as opaline flint, [3]. N.B! These figures have been achieved by use of a modified version of NT BUILD 295. The difference from the original NT BUILD 295 is that instead of a mortar prism a concrete prism is used.

The experience of the rocks from the Caledonides is limited in Sweden. In general, the aggregates are similar to those in Norway, considering that many of the rocks are derived from 'Norway' during the Caledonian orogenesis.

It is only fairly recently that ASR has been recognized as a deterioration mechanism in Sweden. This is due to the fact that most concrete until about 30 years ago was made with low alkali cement. At this time a high alkali cement was introduced. Today ASR is recognized as being fairly common, although not severe, in some areas and in structures from the 1960s and 1970s. It has also been recognized in some old bridges in which the reaction is probably the result of de-icing salts in combination with frost action and ASR.

15.2 Specifications and recommendations

There are very few specifications concerning the allowed amount of reactive aggregate or allowed expansion in the Swedish standards.

One “specification” concerning ASR is included in the Swedish Concrete Regulations [4]. It is only stated that the aggregate may not contain deleterious components in such amounts that the quality and performance of the concrete or reinforcement is decreased. There is no definition of acceptable amounts.

In the Swedish Bridge Code of 2004 [5], it is only mentioned that aggregates in bridges in contact with sea water should not contain ASR aggregates. The Swedish Road Code of 2005 [6], gives reference to EN-12620 [7] and EN-137003 [8]. EN 12620 only gives general guidance of what to do in connection with a risk of AAR. And the Swedish application document gives about the same information as the concrete regulations mentioned above.

In the regulations for FPC for aggregates from the Swedish Concrete and Aggregate Certification [9] it is stated that the producer shall present a petrographic analysis once every third year. The petrographic analysis shall be carried out by a petrographer with documented competence in the area. In addition, the Report of the analysis shall comment on the suitability of the aggregate for the intended use. The same references as above are given: EN-12620 [7] and EN-137003 [8].

In the regulations for certification of pre-mixed, dry mortar it is stated that the mortar shall be initially type tested for alkali reactivity. The specified method is NT BUILD 295 in a slightly modified version.

In 1998 the Swedish Concrete Association issued the second edition of a report called 'Durable Concrete Structures'. [10]. In this report it is stated that concrete exposed to moisture shall be made with a cement containing less than 0,6 wt % alkali oxides. In addition, 'Aggregates suspected of containing alkali-reactive particles must be investigated as regards its reactivity'. Recommended test methods are given below.

15.3 Test methods

The report 'Durable Concrete Structures' gives recommendations to three different test methods depending on the type of aggregate. For the typical Swedish concrete sand, 0/8 mm of glaciofluvial origin, NT BUILD 295 is recommended. The test method NT BUILD 295 prescribes a saturated NaCl solution (50°C) as the accelerating media.

The recommendation is that the expansion shall be maximum 0.7 ‰ in the highest environmental classes (Class B4/B5) after 20 weeks of exposure in the salt solution. In the third class (B3) a maximum of 1.0 ‰ is recommended.

For crushed rocks, both the Canadian CSA A.23.2- 14A and ASTM C 1260 are recommended. The proposed limits are 0,4% and 0,1 % for CSA and ASTM respectively and class B4/B5. For class B3 the recommendation is to use low alkali cement if the expansion is larger than 0,4 ‰ and 0,1 ‰ respectively. For rock types like sandstone and phyllite, the ASTM C 1260 is the recommended test method.

Presently, SP carries out the majority of all petrographic analysis focussing on AAR, in Sweden. The primary method used is RILEM AAR 1 (with the addition that fluorescent thin sections shall be prepared in order to detect any porous flint). N.B! The used technique is point-counting and not particle counting! The recommendation is that if the amount of slowly reactive material is below 15 % you are on the safe side. If the amount is above 20 % and the aggregate still is to be used, it’s reactivity shall be tested with either the RILEM AAR 2 (Ultra accelerated mortar bar method, continued for at least 28 days) or the RILEM 3 (one year concrete prism method, 38 C° moist storage). The same limits as recommended in ASTM C 1260 and CSA A.23.2- 14A have been used respectively. However, if the result is that the prisms still expand after 28 days and is slightly below the critical limit, the aggregate is considered reactive. If the amount of potentially slowly reactive components, identified by microscopy, is between 15 and 20 % the stability (small variation) of the petrographic composition of the quarry and the product is critical and also that the same petrographer has carried out the analysis through the years. Both these two aspects greatly contribute to the uncertainty of the test results and the representativity of the one sample tested for the product quality over the years. If the content of highly expansive materials, like porous and opaline flint, is above 1 vol %, an accelerated test is recommended, NT BUILD 295.

Finally, for many major construction projects, special requirements are prepared by consultants.

15.4 References

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5. BRO 2004. 'Bridge Code' (in Swedish).
6. ATB VÄG 2005. 'Road Code' (in Swedish).
7. SS 137003:2004 ed.3); "Concrete – Application of EN 206-1 in Sweden"
8. EN-12620 Aggregates for concrete
9. Swedish Concrete and Aggregate Certification Ltd. Certification rules for aggregates. Doc no. CB7-CE from 2005
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16 United Kingdom

The following Chapter has been reviewed by Philip Nixon, Building Research Establishment (BRE).

16.1 Introduction

Damage attributed to alkali aggregate reactions was first identified on the mainland of the UK in 1976 in South West England although cracking in a dam on Jersey had previously been diagnosed as caused by AAR in 1971. Since then damage attributable to AAR has been found in 100 to 200 structures, predominantly in the South West and Midlands of England with isolated cases elsewhere. However, no new cases have been identified since publication of the UK guidance on ASR, outlined below.

16.2 Specifications against ASR

Detailed guidance is given by the Concrete Society in its Technical Note 30 (2) and BRE in its Digest 330 (3). These documents are referred to in the British Standards Institution Specification for Concrete, BS 5328: 1990 (4) and in the complementary British Standard to EN206-1, BS 8500 (5), which replaces BS 5328 at the end of 2003. They are also the basis of the highway Agency (6). A further revision of Digest 330 is planned for 2003. In this new document guidance on the use of metakaolin, micro silica, coarser fly ashes and lithium compounds will be given.

The basic principle underlying these specifications is that for a damaging alkali silica reaction to take place the following three conditions must be met simultaneously:

- there must be a sufficiently alkaline solution in the pore structure of the concrete
- the aggregate combination must be susceptible to attack by this alkaline solution
- there must be a sufficient supply of water.

In practice in the UK there is a concentration on limiting alkali levels in cements and concretes. This is because a high proportion of aggregates contain potentially reactive siliceous rocks and it is not practical to avoid their use.

Alkalis in concrete

The recommendations to minimize the risk of damaging ASR in concrete are set out in Table 1. The basis of this table is that aggregates are classified into high normal or low reactivity, depending then on the reactivity of the aggregate and the alkali level of the cement used, different limits are set on the alkali content of the concrete. Where a calculation of the alkali content of the mix is necessary this is primarily based on the alkali in the Portland cement. The alkali level in gabs or PFA used in a cement or combined at the mixer is not included in the calculation provided minimum proportions of these materials are used (see below).

The background to this new guidance is recent research which has shown that different aggregate types require markedly different alkalis in the concrete to produce damaging expansion and that the effectiveness of cement additions such as pulverized fuel ash (PFA) and granulated blast furnace slag (gabs) depends on the type of aggregate used. By adopting this classification the guidance has been able to allow a more rational use of materials, provide environmental and financial benefits and still ensure safe construction.

Aggregate classification

Aggregates are classified into high, normal or low reactivity on the basis of either their petrographic type or by use of the BS Concrete Prism test (6).

Aggregates of low reactivity

The low reactivity class of aggregates, or combination of aggregates, includes many of the rock types that have, to date, not been implicated in damaging ASR in field concretes in the UK.

Aggregates of normal reactivity

Most aggregates extracted in the UK are classified as normally reactive.

Aggregates of high reactivity

These are aggregate combinations known to react at lower alkali levels than more common UK concreting aggregates. The main high reactivity aggregate type is crushed greywacke including greywacke type sandstones and siltstone/mudstones or combinations containing more than 10% of these. Additionally, pending more experience, recycled demolition waste and crushed concrete waste is classified as highly reactive.

Assessment of aggregate reactivity

The principal method of aggregate classification is by petrographic examination to identify the rock types present. This is done using the BS method (7) Detailed guidance on how different rock types should be classified is given in the BRE Digest(3) and Concrete Society Technical Note(2)

Alternatively, a British Standard method using concrete specimens which is now a Wt for development can be used (7). Acceleration is achieved by use of a high alkali level (7 kg Na2Oequiv) in the concrete and storage of 38°C and high humidity. The length change of specimens is monitored up to at least a year, although, for some greywacke aggregates longer monitoring is necessary. On the basis of their 12 month expansion in this test aggregates are classified as:

low reactivity	$\leq 0.10\%$
normal reactivity	$\geq 0.10 \leq 0.20\%$
high reactivity	$\geq 0.25\%$

Cementitious materials:

Portland cements

Investigations of UK structures affected by ASR have shown that damage is nearly always associated with the use of an abnormally high alkali Portland cement ($Na_2O_{equiv} > 1\%$). Conversely, long term expedience in both field and laboratory concretes has confirmed the well established use of a low alkali ($Na2O_{equiv} < 0.60\%$) cement which is available as an option under BS4027(8), to minimise damage from ASR. For this reason the underlying principle of the advice is that greater precaution is required to minimise damaging ASR as the alkali content of Portland cement increases.

For the purposes of this guidance Portland cements have been divided into three classes of alkali content:

low	$\leq 0.60\%\ Na_2O_{equiv}$
medium	$\geq 0.60 \leq 0.75\%\ Na_2O_{equiv}$
high	$\geq 0.75\%\ Na_2O_{equiv}$

Use of ground granulated blast furnace slag (ggbs) and pulverized fuel ash (pfa)

Research over the last 10 years has shown that the effectiveness of gabs and PFA in preventing damage from ASR depends on the reactivity of the aggregate in the concrete and the proportion of gabs or PFA used. In this guidance different minimum proportions are recommended in Table 1 depending on the reactivity class of the aggregate. When used at or above the minimum proportions no account need be taken of the alkali in the gabs or PFA For low and normal reactivity aggregates the minimum proportions are 40% gabs or 25% PFA . However, for high reactivity aggregates the minimums are 50% gabs or 35% PFA. Where it is necessary to use less than these recommended minima a fraction of the alkali contributions from the ggbs or PFA should be taken into account in the mix alkali calculation.

Alkalis contributed to the fresh concrete from other sources.

Alkali from other sources (e.g. salt in sea dredged aggregates or admixtures) included at the time of mixing have been shown to contribute to damaging ASR in a small number of structures. In mixes where no mix calculation is needed alkalis from sources should not exceed 0,6 kg Na_2O_{equiv}/m^3 or specialist advice should be sought. Where alkali mix calculations have to be made either the total allowable cement content or the alkali limit must be reduced.

Diagnosis of ASR

It is recognised in the UK that establishing whether or not observed cracking in concrete is wholly or partly caused by ASR is not simple. Detailed guidance on the procedures to be followed in the diagnosis of ASR and the interpretation of results has been published (9).

References

(1) The alkali silica reactivity of British concreting, sands gravels and volcanic rocks. Transport and Road Research Laboratory Contractor Report 218 1990 (published by TRRL, Crowthorne, Berks, UK).

(2) The Concrete Society, Alkali Silica Reaction - Minimizing the risk to concrete. Technical Report 30. Third edition 1999.

(3) BRE Digest 330, 1999. Alkali Aggregate Reactions in Concrete.

(4) BS 5328: Part 2, Concrete – Parts: Methods for specifying concrete mixes, British Standards Institution, 1997.

(5) BS 8500, Concrete – Complementary Standards to BS EN 206-1, British Standards Institution, 2002.

(6) Highways Agency, Specification for Highway Works, Series 1700 Structural Concrete, Clause 1704/6, Control of Alkali Silica Reaction, Amendment – May 2002.

(7) BS 812-123, Testing aggregates – Part 123. Method for determination of alkali-silica reactivity – Concrete prism method, British Standards Institution, 1999.

(8) BS 812-104, Testing aggregates – Part 104. Method for qualitative and quantitative petrographic examination of aggregates, British Standards Institution, 1994.

(9) BS 4027 Sulfate-resisting Portland Cement, British Standards Institution, 1996.

(10) The diagnosis of alkali-silica reaction; Report of a Working Party, British Cement Association, 1992.

Table 16.2-1: Recommended limits for alkali contents of concrete.

Aggregate type or combination.	Alkali content of Portland cement or the BS 12 component of a combination of ggbs or pfa.		
	Low alkali PC (Guaranteed ≤ 0.60% Na_2O_{eq} on spot samples)	Moderate Alkali PC (Declared mean ≤ 0.75% Na_2O_{eq})	High Alkali PC (Declared mean > 0.75% Na_2O_{eq})
Low reactivity aggregates	SELF-LIMITING: No mix calculation needed.	SELF-LIMITING: No mix calculation needed.	LIMIT: ≤ 5.0 Na_2O_{eq}/m^3
Normal reactivity aggregates	SELF-LIMITING: No mix calculation needed.	LIMIT: ≤ 3.5 Na_2O_{eq}/m^3	LIMIT: ≤ 3.0Na_2O_{eq}/m^3
High reactivity aggregates	LIMIT: ≤ 2.5 Na_2O_{eq}/m^3		