ALKALI AGGREGATE REACTIONS (AAR) IN CONCRETE

WORKSHOP PROCEEDING

FROM A

NORDIC - BALTIC MINISEMINAR

RIGA - LATVIA

21. - 22. NOVEMBER 2013

PREFACE

This publication contains 8 papers presented at a Workshop (Nordic Mini Seminar) concerning Alkali Aggregate Reactions (AAR) in concrete.

The Nordic Concrete Federation aims to establish a closer connection with the Baltic countries. This workshop was therefore arranged in Riga in Latvia, with many participants from the three Baltic countries.

The workshop was organised by FARIN (Forum for AAR in Norway) and chaired by Børge Johannes Wigum from Mannvit/NorStone/NTNU. The Norwegian Public Roads Administration sponsored the event.

In order to stimulate discussions between the participants, the Workshop was arranged as a two-day residential course, November 21th – 22th 2013. Deleterious AAR in concrete structures are relatively recently revealed and documented in Finland and in the Baltic countries. However, in the rest of the Nordic countries, these reactions have been documented and studied for decades. Various contributions at the Workshop presented topics such as; Petrographic analyses of aggregate, Performance based testing of concrete, National- and international test methods and regulations, Diagnosis and appraisal in field structures and Structural consequences and rehabilitation.

35 researchers from Denmark, Finland, Iceland, Norway, Sweden, Latvia, Lithuania, Estonia and the Netherlands participated in the workshop.

The present publication is Number 11 in a special series of Workshop-Proceedings of the Nordic Concrete Research journal.

Nordic Concrete Research (NCR) is a bi-annual publication of The Nordic Concrete Federation, presenting research and practical experience in the field of concrete technology, both from structural and material perspective. Every third year one of the publications is devoted to abstracts from the Nordic Concrete Research Symposia. (Nordic Concrete Research – Research Projects 20XX). The next Concrete Research Symposium will be held in Reykjavik 13. to 15. of August 2014, and we look forward to meet you there.

Papers published in NCR are normally thoroughly reviewed by three reviewers. The papers in the present proceeding have however not been reviewed in this way. Instead the authors revised their papers after the workshop, based on comments and information obtained there.

Reykjavik, March 2014

Børge J. Wigum   Dirch H. Bager
Editor

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Addressing ASR in concrete construction in Finland

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ABSTRACT

Alkali-aggregate reaction (ASR) in concrete is one of the least dealt with concrete degradation mechanism in Finland. It is a common belief that it does not occur due to the exceptional quality of Finland's granitic rock. However, over 50 cases have been identified in the past 15 years in concrete structures, raising uncertainty in this claim and concern about the occurrence and extent of ASR in Finland.
The amount of ASR is expected to increase rapidly as a result of geological constraints, and the inadequate or insufficient national regulatory norms on the topic. Actions now need to be taken otherwise the risks of future cases may be growing significantly. There is also concern over the interaction of damages cause by freeze-thaw and ASR, due to possible synergetic effects. This paper presents the current situation in Finland with regards to ASR and outlines the strategy to proactively address the problem.

Key words: Instructions, layout, headings, figures and tables, references.

1. INTRODUCTION

Alkali-aggregate reaction (ASR) in concrete is one of the least dealt with concrete degradation mechanism in Finland. It was a common belief that Finland does not have ASR. The reasons for this are that the aggregate commonly used originates from very high quality granitic rock; that the very extreme meteorological conditions favour the delay in visual detection due to the effect that the very cold climate has on the chemical reaction rates; and that the visual manifestations of the ASR degradation mechanism are very similar to those of frost attack, and therefore it is possible that the problem has in many cases been erroneously diagnosed. In addition, ASR has traditionally been left out of Finnish academic curriculum so engineers are not knowledgeable concerning it.

An initial study, funded by the Finnish Transport Agency in the year 2011 [1], reviewed petrography investigations of concrete structural samples covering various geographic areas and structural types around Finland. Some susceptible types of Finnish aggregates have been identified based on thin-section studies, though the geological locations have not yet been identified. In two or three cases, ASR was determined to be the primary cause of damage of the structure and in one case has resulted in plans to demolish and replace a bridge. All Finnish ASR cases have been detected in structures over 30 years in age.

Over 50 cases of ASR in Finnish concrete structures have been identified in the past 15 years, disproving this claim and raising concern about the occurrence and extent of ASR in Finland in the future. This figure is expected to increase rapidly as a result of geological constraints, the non-application and in some cases, inadequate or insufficient national regulatory norms on the topic of ASR. Actions now need to be taken otherwise the risks of future cases may be growing significantly. There is also concern over the interaction of damages cause by freeze-thaw and ASR, which should also be a topic of international review in the future. Since this initial Finnish survey-type study [1,5], more ASR cases have come to light. This paper presents the updated results of the initial survey, with cases detected in the last two years.

While historically there have been no well-publicised cases to contradict the general belief that ASR does not occur in Finland, recent findings when investigating reinforced concrete infrastructure for other types of deterioration attack, such as chloride ingress and frost-salt exposure, have also identified the occurrence of ASR. There is the possibility that ASR will become a problem in the near future because of changing materials, such as binders with higher alkalis and use of alternative aggregate sources, but also because better and more sophisticated tools to identify the occurrence of ASR are available. Also much of the infrastructure in Finland has been built during and after the 1970s, and these structures are now coming to the age where ASR has been found. The authors suspect that a mixture of extreme climate and favourable geology has helped keep ASR undetected for so long. However, the ever aging infrastructure is beginning to show more and more signs of deterioration.

1.1 Factors influencing ASR in Finland

It is well documented that for alkali-silica reaction to occur, three conditions must be present: aggregate with reactive forms of silica, high-alkali (pH) pore solution, and sufficient moisture [2]. In the following a description of each of these factors concerning Finnish practise (reality) is given.

Reactive aggregate – Finnish geology
In Finland the bedrock is manly made of Precambrian plutonic and metamorphic rocks. The occurrence of sedimentary rocks is marginal and also these rocks are of Precambrian age. The metamorphic rocks are different metavulcanites, gneisses, schists and quartzites and they are often highly metamorphosed folded. The very old bedrock is covered by soil from the latest ice age, about 10000 years ago, so there is a wide time gap between these two. The soil represents well the composition of the underlying bedrock. Granites and granodiorites are the most common rock types in the bedrock and soil in our country and for this reason Finland is known

as a country having very durable granitic aggregate that is used in a wide range of construction applications.

From a geological point of view, there is very much in common between Finland and Sweden. In both countries the bedrock is made mainly of Precambrian rocks (Figure 1). This is, in itself, quite an important aspect because Sweden has acknowledged the existence of ASR already 20 years ago [3] and has addressed this problem accordingly. On the map it can be seen that the bedrock both in Sweden and Finland represents the same Precambrian association made of metamorphic, cataclastic and mylonitic schists and gneisses, cut by plutonic rocks like granite and gabbro.

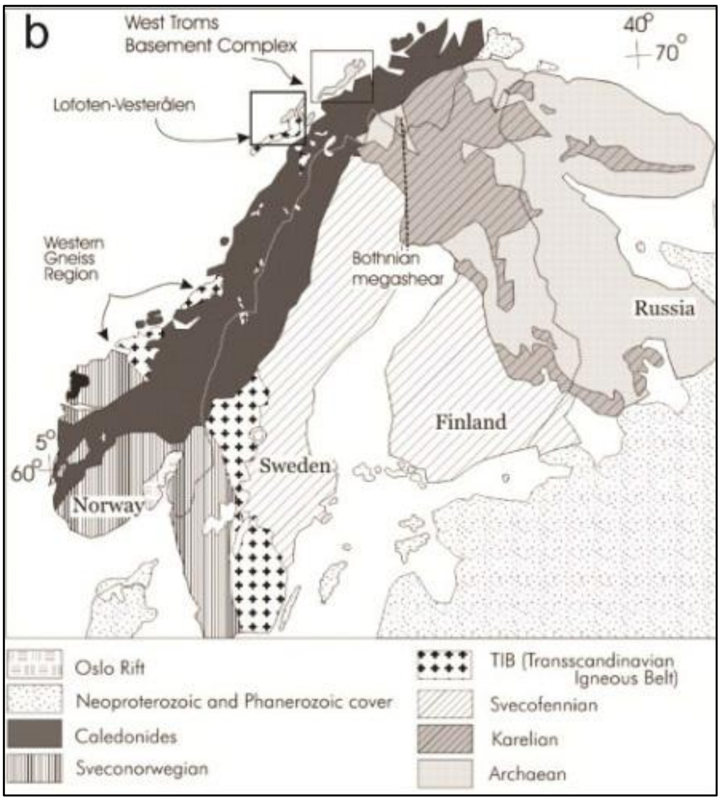

Figure 1 – A simplified geological bedrock map of Fennoscandia [4].

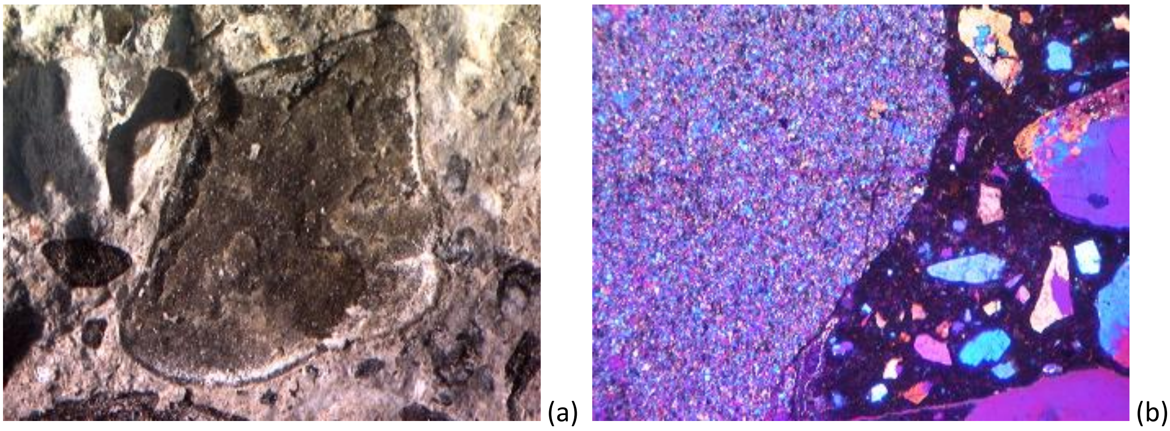

Figure 2 – (a) A reacted black quartz rich aggregate in a bridge concrete; (b) A micrograph of the same aggregate particle. The grain size of the particle is extremely fine. The height of the micrograph is 2.7 mm [5].

In Finland, ASR is more connected to certain rock types than to certain geographical areas. The geology of Finnish bedrock is well-studied and therefore there is a good general view of the composition of aggregates, if crushed from bedrock. For soil (surface aggregate deposits) there is a poor characterization of rock type, which is expected to be addressed in future ASR cooperation with the Geological Survey of Finland. The Finnish rock types associated with ASR are in most cases fine grained stressed and strained mica bearing quartz rich shists and quartzites. The grain size is very often under 0.1 mm (Figure 2). Also some fine grained mylonitic, cataclastic and other rock types have shown ASR. In these cases the ASR is dependent on the structure and size of the quartz.

Source of alkalis

Alkali hydroxides in the pore solution will react readily with reactive forms of silica in aggregate. As the aggregate reactivity increases, the gel reaction products can be formed with lesser concentrations of alkali. For this reason, the use of low-alkali cements alone may not be sufficient to control ASR with highly reactive aggregates. As the alkalinity of the pore solution increases, potential for the ASR increases. At higher concentrations of alkali hydroxides, even the more stable forms of silica are susceptible to attack. If the alkali concentration is great enough, the alkali hydroxides break stronger silicon bonds found in less reactive aggregates to form the gel reaction product. This explains why aggregates thought to be nonreactive sometimes exhibit ASR [2].

Approximately 80% of the cement used in Finnish construction and products is produced by Finnsementti Oy. On average over the past 5 years, the annual amount of cement produced in Finland is approximately: 45% Yleis cement (CEM II/A-M (S-LL) 42,5N), 35% Rapid cement (CEM II/A-LL 42,5 R), 10% Pika cement (CEM I 52.5R), 5% Plus cement (CEM II B-M (S-LL) 42,5 N), 2% Sulphate resistant cement (CEM I 42,5 N) and 2% white cement (CEM I 52,5 R) [Finnsementti 2012].

Ideally, the concept of total alkali loading should include the alkalis from all of the constituents of concrete. However, usually only the alkalis from the cementitious materials are included in the determination because alkali contribution from other constituents is usually small. Recommendations to limit alkali-aggregate reactions often specify the total equivalent alkali content to be less than 0.60 % [6-9]. Alkalis in cement typically vary between 0.2 to 1.5 % Na_2O_{EQ} due to the use heat exchangers in the cement industry leading to increased alkali content in cements today. Depending on the alkali content of cement, the pH of the pore solution varies from 12.5 to 13.5. [10]. Concrete made with low-alkali cement can still exhibit expansive ASR if moisture movement concentrates the alkalis in one location, if the aggregate is extremely reactive, or if alkalis are provided by other constituents of concrete in addition to the binders.

The corresponding chemical composition of the Finnish cement types listed above is given in Table 1, according to clinker types. It should be noted from Table 1 that by international practice, all of the Finnish cements (with the exception of Danish White cement) have equivalent alkali contents over 0.6% Na_2O_{EQ}. The Yleis and Rapid cements have values over 1.0%, which is considered high by international practice.

Producing lower-alkali cements may require higher amounts of energy, due to the bypass systems needed to avoid the alkali concentrations in the clinker. Thus as energy efficiency receives more attention in the global economy, there are also recommendations that the requirements for low-alkali cements should be limited to cases where alkali-reactive aggregates are a problem [12].

Table 1 – Example chemical composition of cements and blast furnace slag, taken Autumn 2007. (Finnsmetti Oy) [11].

Cement type/ Composition	CEM II/B-S 42,5 N Perus cement	CEM I 42,5 N - SR SR-cement	CEM II/A-M(S-LL) 42,5 N Yleis cement	CEM I 52,5 N White cement	CEM II/A-LL 42,5 R Rapid cement	CEM I 525 R Pika cement	Blast furnace slag - KJ400
CaO	64	63	65	69	65	64	40
SiO_2	21	21	21	25	21	21	34
Al_2O_3	4.3	3.3	5.2	2.1	5.2	4.3	9.3
Fe_2O_3	3.0	4.0	3.1	0.3	3.1	3.0	-
MgO	2.9	2.9	2.6	0.7	2.6	2.9	11
SO_3	3.0	3.1	3.1	2.2	3.7	3.5	-
K_2O	*	0.43	1.2	0.04	1.2	0.53	0.47
Na_2O	*	0.52	0.31	0.19	0.56	0.6	0.47
Free Lime	2.0	2.5	1.8	*	1.8	2.0	-
LOI	-	2.2	-	0.44	-	1.7	-
C_3A	6.5	2.0	8.5	5.0	8.5	6.5	-
$Na_2O_{EQ.}$	-	0.80	1.10	0.22	1.35	0.95	0.78

* *unspecified*

Alkalis from external sources may increase expansion caused by ASR, especially when concrete is cracked or is highly permeable. Common sources of external alkalis are deicing salts, seawater, groundwater, and water from industrial processes. Sodium chloride deicing salt solutions and seawater can provide unlimited amounts of alkali.

De-icing roads is a common practice in Finland since the 1950s. The amount of salt spread reached a peak in the 1990s, and today it is about one third of the maximum values. The need for salting depends on weather conditions. If the temperature is low, approximately below –5°C, salt is not used. The conditions when salt is needed most are the temperatures a few degrees below the freezing point. It was found that warmer than normal weather in November and in March and colder than normal weather in December, January and February will decrease salting. Warm midwinter months mean slippery conditions and thus also much salting [13].

As an example of salting practice, along Highway 7 (to Kotka), the de-icing salt used has been mainly sodium chloride (NaCl), which is spread as crystals. Additionally some $CaCl_2$-solution has been used as a preventive measure. Since the autumn of 2010, mainly NaCl-solution has been used instead of $CaCl_2$-solution [14]. Along the highway, de-icing is used normally between October/November and April. The total amount of salt spread on the highway and salting occasions are presented in Table 2. In this example, the highway has four lanes (two in each direction) and all of the lanes were salted with a 3 m working width.

Table 2 – Salt content and application for three consecutive winter season [14].

Winter season	2007/08	2008/09	2009/10
Salt content (kg/m^2)	1.3	1.3	0.7
N. of occasions	165	154	65

Climate and exposure conditions

Moisture allows migration of alkali and hydroxyl ions to reaction sites and the resulting gel absorbs moisture, leading to expansion. For this reason, deleterious ASR does not occur in concretes that are dry in service. Research has shown that expansive ASR can occur in concrete having a relative humidity above 80 % [15]. It is possible for well-cured concrete in arid regions to have a relative humidity at or above 80% just beneath its surface, even after many decades. Any reduction in permeability, by using a low water-cement ratio, supplementary cementing materials or other means, reduces movement of moisture and alkalis into and within the concrete.

Structures in warmer climates are more susceptible to ASR than those in colder climates because the reaction rate usually increases with increasing temperature. For the majority of aggregates, higher temperatures also mean larger ultimate expansions. However, there are studies showing that lower temperatures resulted in significantly larger ultimate expansions with certain aggregates. The effect of high or low temperatures on ultimate expansion is aggregate-dependent, with most aggregates reacting more at higher temperatures [2].

The main factor influencing Finland's climate is the country's geographical position between the 60th and 70th northern parallels in the Eurasian continent's coastal zone, which shows characteristics of both a maritime and a continental climate, depending on the direction of air flow. Finland is near enough to the Atlantic Ocean to be continuously warmed by the Gulf Stream, which explains the unusually warm climate considering the absolute latitudes [16]. With regards to weather, Finland has a humid and cool semi-continental climate, characterized by warm summers and freezing winters, with an average January temperature of -4°C in the Helsinki region and -14°C in Lapland. Winters of southern Finland (average daytime temperature < 0°C) are usually 4 months long. As a consequence of this climate, concrete infrastructures are subjected on average to approximately 70 freeze thaw cycles yearly (data for 2007-2010) [15].

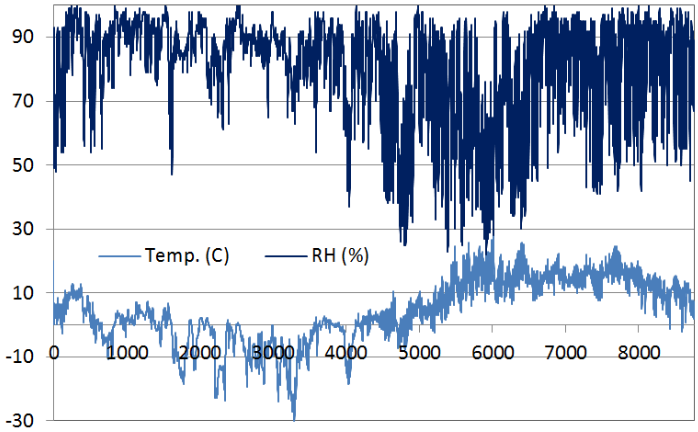

Figure 3 – Average yearly variation (hourly measurements) of temperature and relative humidity in Helsinki [17].

Northern Finland has a harsher climate, though the number of freeze-thaw cycles is about the same. The cycles are concentrated in the spring and fall, while in the capital region of the south they occur through-out the winter period. As an indication of the type of climate around the greater Helsinki region, the average rainfall, temperature and relative humidity values are 650 mm, 4.5 °C, and 79.7 %, respectively. Figure 3 shows yearly variation in both temperature and relative humidity.

Many economically significant Finnish industrial facilities (i.e. pulp and paper factories, pre-cast concrete factories) and public services (pools, spas, etc.) are obliged to create artificial environments to guarantee the continuation of their activity throughout the year, resulting in high risk environments where the concrete structures are often very moist and warm. Exterior concrete structures in Finland are also at a risk of AAR, though the deterioration may take longer to appear. The cold climate slows the chemical reaction rate involved in AAR.

1.2 Typical infrastructure and concrete materials

The structures most at risk for ASR damage in Finland are bridges, highway passes, dam structures, exposed frames (i.e. open multi-storey car parks), foundations, and many types of industries (pulp and paper factories, nuclear energy, power generation, water management, etc.) with concrete infrastructure.

The Finnish concrete industry is estimated to have turnover of about 700 million euros per year, with cement production about 1.5 million tons/year. Even with a cold climate, construction proceeds year-round. This is accommodated by an industry highly dependent on pre-fabrication factories, as 47% of concrete is delivered as precast elements and 31% is ready-made concrete [17].

The common bridge concrete mixes in Finland are characterized by a water/binder ratio of 0.40 to 0.55, typically with 5% air and a design compressive strength class of 35-45MPa. The most common cement type is classified as CEM II/A-M (S-LL) 42,5 N, which contains up to 10% interground limestone or blast furnace slag and a Blaine fineness of 370-440 m^2/kg. Finnish practice allows for relatively high amounts of mineral by-product additions to concrete, with blast furnace slag and fly ash readily available. Sulphate resistant cements are also produced and used in some cases.

1.3 Why has ASR not been seen earlier?

When considering that Finland’s neighbours have already for many years addressed ASR, the question arises, why has ASR not been seen earlier in Finland?

Historically, is has been taught in Finnish building technology education and material science that alkali-aggregate reactions are not a concern. Finnish aggregate and concrete durability guidelines even state that “According to common belief, ASR is not a problem when using Finnish aggregates.” [18].

Another contributing factor is that, not only have petrographers not been trained to identify ASR, but also condition assessment or inspection experts have difficulty in differentiating between damage initiated by frost action and that induced by ASR, especially because it is widely believed that ASR does not occur in Finland. For example, field inspections in Finland have typically focused on the near surface area from which cores are typically taken to look at frost damage. It has often been neglected to take deeper cores to consider if ASR was a possible cause of cracking. The results of visual inspections show cracking on the surface of a structure, either as a map-pattern or aligned due to constraints. The AAR-induced cracking can also be affected by restraint, leading to aligned cracking. On the basis of crack patterns, a skilled person can make a quite good estimate on the cause of cracking. And sometimes it is possible to even see ASR gel that has moved out to the exposed concrete surface. This is the most distinguishing factor between frost damage and AAR: the appearance of gel within interior cracks [19]. However, the gel can also be leached from the cracks. In frost damage, the cracking is normally heaviest near the surface and weakens deeper, whereas in ASR an internal reaction causes expansion from within the structure causing a homogeneous cracking network in the whole structure. In this way ASR can be more harmful than frost action.

As noted above, Finland has a strong tradition of using mineral by-products, such as fly ash and blast furnace slag, which are known to lower the risk of ASR. In addition, the use of air entrainment possibly accommodates expansive gel from the reaction mitigation ASR deterioration. What effect this gel in pores would have on freeze-thaw performance is not yet well researched. Yet closer attention should now be given to the individual material components in Finland, to identify where there may be ASR risks.

2. ADDRESSING ASR IN FINLAND – RESULTS OF 1ST STUDY

An initial study, funded by the Finnish Transport Agency in the year 2011 [1], reviewed petrography investigations of concrete structural samples covering various geographic areas and structural types around Finland. The main goals of the study were to: clarify phenomenologically ASR; identify the locations and extent of occurrence in Finland; show the approach other Nordic countries have adopted to address ASR; and, identify the needs to proactively address ASR in Finland.

A questionnaire survey about the past diagnosis of ASR was distributed to 10 Finnish companies making concrete analysis using petrography/ microscopy thin-sections. From the survey results, a total of 56 cases of obvious ASR damage in Finnish concrete structures were found from investigations over the past 15 years. An obvious ASR case meant that gel could be detected in the cracks and/or aggregates. It must be emphasized that the survey did not include cases where ASR might have been a cause of cracking or deterioration but was inconclusive due to a lack of a gel product in the cores and/or thin-sections.

When core samples were taken for studies of concrete performance, thin-sections were always made. The ASR was not evident in all of the thin-sections for the surveyed structures. Within the 56 structural cases, there were a total of 331 thin-section studied and ASR was evident in 33% of the thin-sections (111 of 331 thin sections). The occurrence of ASR was not uniform. In some cases, 4/4 thin-sections revealed ASR damage, but in other cases only 1/24 or 1/12 thin-sections showed signs. It is known that in at least two cases ASR was the primary damage mechanism of the structure, i.e. resulting in structural maintenance or replacement actions being taken.

The regions in Finland where ASR was found are shown in Figure 4. It must be noted that the survey sampling was not comprehensive, so that many geographical areas of Finland are not represented due to lack of sampling, and not lack of ASR occurrence. The types of structures where the ASR was reported are shown in Figure 5. The Figure shows that the majority of cases were in bridges (39%) and houses (34%).

It is during the recent years that ASR has been identified in about 10 bridges from various locations around Finland (Figure 4). The greatest number of those are however located in southern Finland and near bigger cities. The findings of ASR are distributed around the country as shown in Figure 4. In these cases the level of damage varies from low to high. Often the role of ASR has been smaller than the role of other damaging forces. In 2 – 3 of these cases, ASR has played a major role. In these cases initial thin section studies gave a warning about high levels of ASR and a heavier damage than was recognized on site. The studies were completed with additional thin section studies, where strong ASR was found on the bridge decks. In Finland, bridge decks have been more favoured to ASR than other structures (like columns), because of the use of de-icing salts and occurrence of rain that pools on the decks.

The age of Finnish bridges where ASR has been found has varied from 31 years to over 50 years. In most cases the age of the bridge has been between 40-50 years when studied. ASR has not yet been found in Finnish bridges under the age of 30 years. In general, frost action is still the most prevalent damaging force in Finnish exposed structures, yet it is expected that when it hits together with AAR, severe damage could result (Figure 5).

In addition to bridges, ASR has also been found in other Finnish structures. For instance, ASR has been seen in industrial indoor structures where the conditions are favoured, i.e. the temperature is high (even 30-40 °C) and the relative humidity is high. One example is a paper mill from early 1970s, where the temperature is around 40°C and the concrete structures are wet during the use or manufacturing processes. In this case ASR was found in a cataclastic medium to fine grained schist (Figure 6).

Besides bridges and industrial structures, hundreds of Finnish facades and balconies have been studied in recent years, but only in some cases has ASR been found. In these cases the aggregate used proved to be from abroad. Finnish indoor swimming pools and spas have moist and warm conditions and are potential risk structures of ASR. The number of these structures studied so far is low, and no significant damage caused by ASR has been found yet.

The survey asked to roughly identify the types of aggregates that were seen in petrographic studies. The distribution of the aggregate types seen in the survey is given in Figure 6. The largest proportion of the studies did not specify one aggregate type alone or it was unclear, for instance the aggregate was noted to be granite or quartzite. Besides identifying aggregate, the survey respondents noted that in most ASR documented studies the mixture design of the concrete was unknown (i.e. w/c ratio, cement type, aggregate source location, etc.). The survey did not ask the age of the structures where the ASR was found. Both of these items should be a topic for future Finnish studies when clarifying the occurrences of ASR. The survey results also showed a steady increase in in the number of ASR cases being reported in Finland. This may be attributed to the age of the structures but it could also be due to the recent increase in education, awareness and discussion about ASR in Finland [20, 21].

Finally, it should be noted that some of the respondents were aware of other concrete structures affected by ASR that did not get included in this initial survey due to lack of finding the initial report and/or thin-sections and in some case a lack of time for reporting by the survey deadline.

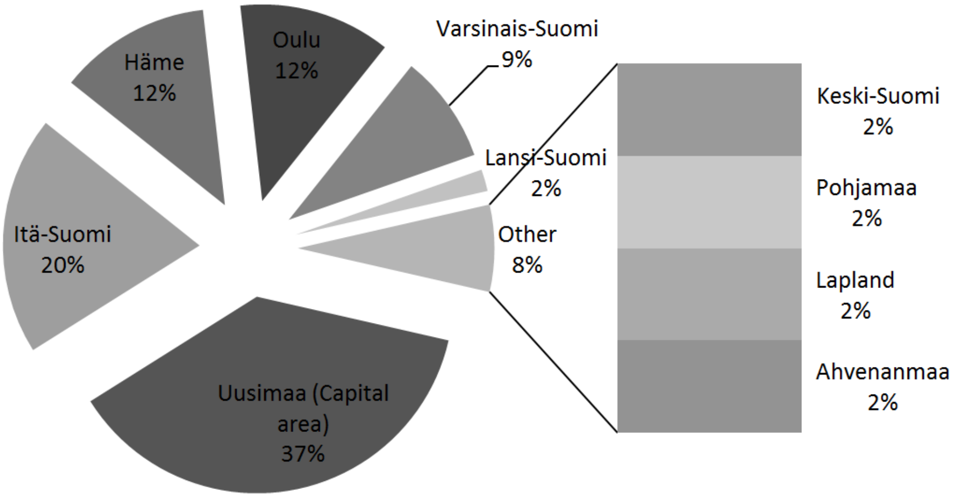

Figure 4. Regional distribution in Finland where ASR in concrete structures has been found based on survey results.

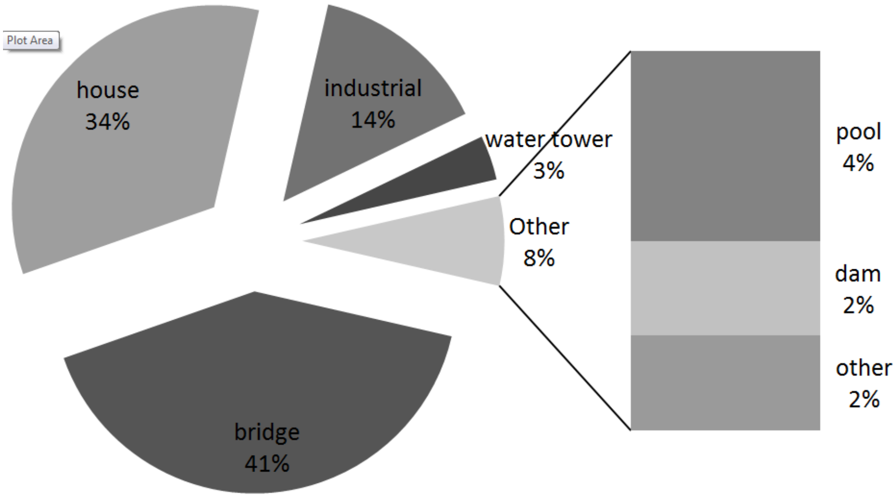

Figure 5. Types of concrete structures where ASR has been found based on survey results.

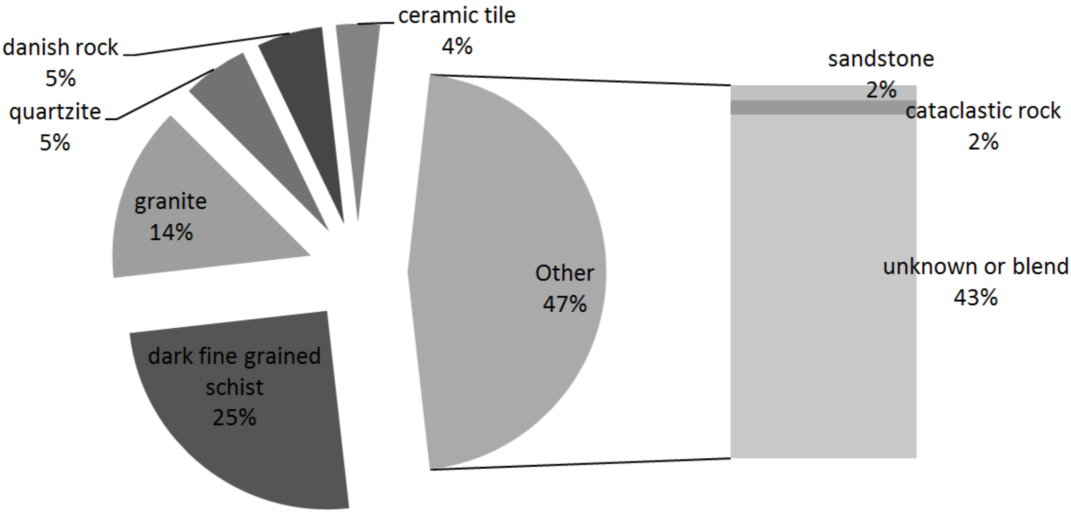

Figure 6. Types of aggregate in Finnish concrete/structures where ASR has been found based on survey results.

3. CASES OF ASR IN CONCRETE IN FINALND

In this chapter, several condition assessment studies are documented photographically. These have been graciously provided by Huura Oy.

(a) (b)

(c)

Figure 7 – Bridge on Paltanmäki. ASR damage was detected in the deck slab (c) during inspection addressing crash damage, in 2004 (b). Relative humidity in the deck slab was 97-99% (a). The deterioration went un repaired.

(a)

(b)

(c)

(d)

(e)

Figure 8 – (a) General view from underneath the Mäntylä Bridge in Turku. ASR was detected mainly in the deck slab and the edge beams. Repair work was performed in 2006. (b),(c) The deck slab repair work consisted mainly in waterjetting the top 5-10 cm and recasting. (d), (e) The edge beam repair work consisted mainly in demolishing and recasting.

(a) (b)

Figure 9 – Kokki Bridge in Tampere. ASR in detected only in the edge beams. These were demolished and recast in 2012.

(a) (b)

Figure 10 – Bridge over the Oulujoki. ASR was detected in deck slab, edge beam and supports (a), (b). Repair work was undertaken in 2004:the top surface of the deck slab was removed (5-10 cm) by waterjetting and recast. The edge beams were demolished and recast

4. RESEARCH NEEDS

As a result of the study mentioned above, the next steps recommended for gaining knowledge and understanding on the risk of ASR in Finland were defined. These are based on the approach taken by other countries, as they have also followed a similar path in clarifying the significance of ASR for concrete structures. This has included the need to: have a better understanding of the expected quantitative magnitude of ASR expansions when specimens are made with Finnish aggregate sources and Finnish cements (most are very high in alkali content, over 1%); learn to better identify ASR or separate it from other deterioration mechanisms; and, develop guidelines about how to mitigate and avoid ASR in both existing and new constructions respectively.

5. FINAL REMARKS

This paper is written with the intent of informing the Nordic/Baltic scientific community of the occurrence of ASR in Finland. While ASR is thought not to be an extensive problem in Finland, no specific inspection program has been undertaken to confirm this. It is not yet possible to define the extent or severity of the reaction in Finland. Several cases that have been identified where ASR is presented, of which, some have been presented and discussed. It is imperative that both the causes and the level of widespread damage of reinforced concrete infrastructure due to ASR in Finland be addressed.

Based on the initial survey, it is clearly showed that alkali aggregate reactions occur in Finland's concrete. Over 50 Finnish concrete cases were confirmed to have shown obvious signs of ASR in studies conducted over the past 15 years. These structures have come from a wide range of geographic areas of Finland and from various types of structures, though ASR has been most evident in bridges and houses. Some susceptible types of Finnish aggregates have been identified based on thin-section studies of structural concretes. Although there are many structures surveyed as being affected by ASR in Finland, this figure is expected to increase rapidly as a result of geological constraints, the non-application and in some cases, inadequate or insufficient national regulatory norms on the topic of AAR.

The time is ripe for a deeper look at the occurrence of ASR in Finland. It is known that there are reactive types of Finnish aggregates and that the source of alkalis is plentiful (cements and de-icing chemicals). The slow reaction rate can have been influenced by the climate, the used of secondary additions and/or the use of air entrainment, however the now aged infrastructure stock is beginning to show it sign. Other reasons that have made difficult the diagnosis of ASR have been the lack of appropriate dissemination of knowledge. New reasons also urge the way towards further research, such as the unknown interaction between freeze-thaw and ASR. Furthermore, it is necessary to guarantee that future investment in new infrastructure should be checked for ASR potential. There still is neither Finnish legislation nor Finnish guidelines for practice.

Can we risk not knowing the extent and possibility of future damage? Can we build new infrastructure without checking for ASR risk when we have the knowledge to avoid it? Only time will tell.

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The advantages of using microscopic analyses as part of the assessment of alkali aggregate reactivity

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ABSTRACT

In Norway, alkali silica reaction (ASR) is a widespread cause of damage in concrete, and this paper focuses on the importance of performing microscopy analyses as part of the assessment of alkali reactivity. A very important part of several research projects since about 1990 was to examine field concrete with microscopic techniques in order to collect information about the behaviour of different rock types in structures. Laboratory examinations of concrete aggregates are important, but the "true answers" are given by the field behaviour.

Key words: Alkali silica reaction, microscopy, durability, concrete aggregates.

1. INTRODUCTION

Concrete is a very important building material, often used in huge structures such as bridges, dams and platforms. In Norway, it is strong focus on the durability of this building material. It is important to avoid future damage to secure long lifetime and low maintenance costs for the owners/society. Usual types of damage on Norwegian concrete structures are rebar corrosion, freeze/thaw damage and alkali-silica reaction (ASR). The damage mechanisms can enhance each other when the concrete cracks, leading to more ingress of chlorides and water. ASR is a widespread cause of damage in Norway, and new cases arise every year. This paper focuses on the importance of performing microscopy analyses as part of the assessment of alkali silica reactivity. Some Norwegian history with microscopic analyses is also included. Chapter 2 describes structural analysis, and chapter 3 deals with petrographic analysis.

A very important part of several research projects since about 1990 [1, 2, 3, 4, 5, 6] was to examine cores drilled from field concrete with microscopic techniques in order to collect information about the behaviour of different rock types in structures. This provided information about the alkali reactivity of the Norwegian rocks. Furthermore, it formed the basis for the Norwegian ASR regulation, latest updated in 2004 [7]. Laboratory examinations of concrete aggregates are important, but the "true answers" are given by the field behaviour (basis to set the critical limits for the various laboratory test methods).

2. STRUCTURAL ANALYSIS

Structural analysis is an important tool (used in Norway since about 1990) to diagnose and describe extent of any ASR in concrete structures. The different steps in an investigation are as follows: Field survey and sampling (core drilling), visual examination of concrete cores in the laboratory, analysis of plane polished sections, analysis of thin sections and in some cases scanning electron microscopy (SEM). All these analyzing techniques are important and supplementary, providing valuable information about the quality of the concrete, the extent of damage and the cause(s) of damage. The occurrence of cracks and the crack patterns are special important parameters in the ASR diagnosis.

2.1 Field survey and sampling

In most cases, the field survey is carried out by the structure owner or a consultant, and not by the executing laboratory. However, it is a great advantage if qualified and experienced personnel can take part in this important start of the examination. An in-situ survey of the structure is important for evaluation of the damage mechanism, and to indicate suitable localities for sampling.

2.2 Visual examination

The visual examination of drilled cores is a very important start of the structural analysis. During this investigation there is special focus on the presence of cracks, their character and extent, in addition to possible precipitations in cracks and air voids. Other features of interest have to be reported, too. Pictures of the drilled cores are valuable and useful. Figure 1 shows a picture of a concrete core with ASR. Figure 2 and 3 show its surface and fracture surface, respectively.

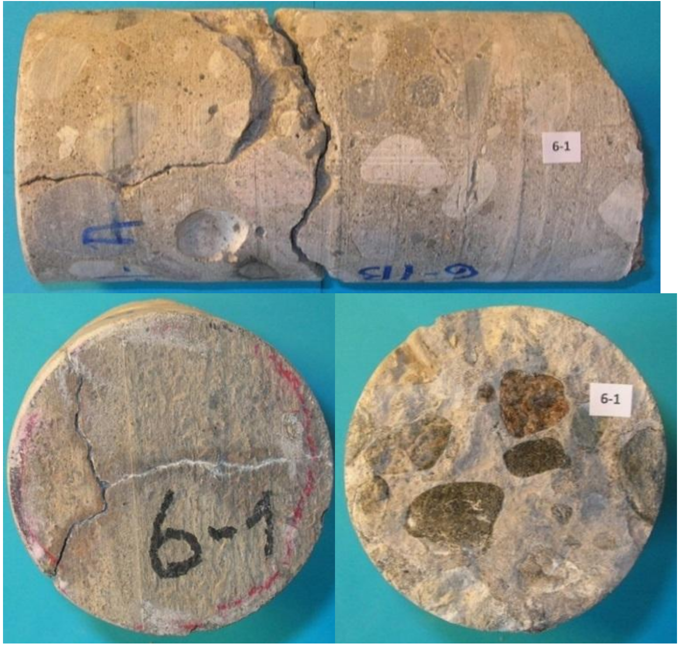


Figure 1-3 – The upper picture shows a concrete core cracked due to ASR, while the lower picture to the left shows the surface of the core. The lower picture to the right shows the fracture surface.

2.3 Analysis of plane polished sections

In the next step, the concrete core is divided into two parts in the length direction. One of the halves is used to produce a plane polished section for macroscopic investigations (i.e. presence and extent of cracks, type of aggregates, air distribution and possible reacted aggregates). The plane polished section is impregnated with fluorescent epoxy, thus the cracks and air voids are easily detected by use of an UV-lamp. This analysis technique gives a good overview of the extent of cracking in the concrete. Figure 4 shows a plane polished section in normal light, while Figure 5 shows the same specimen in UV light.

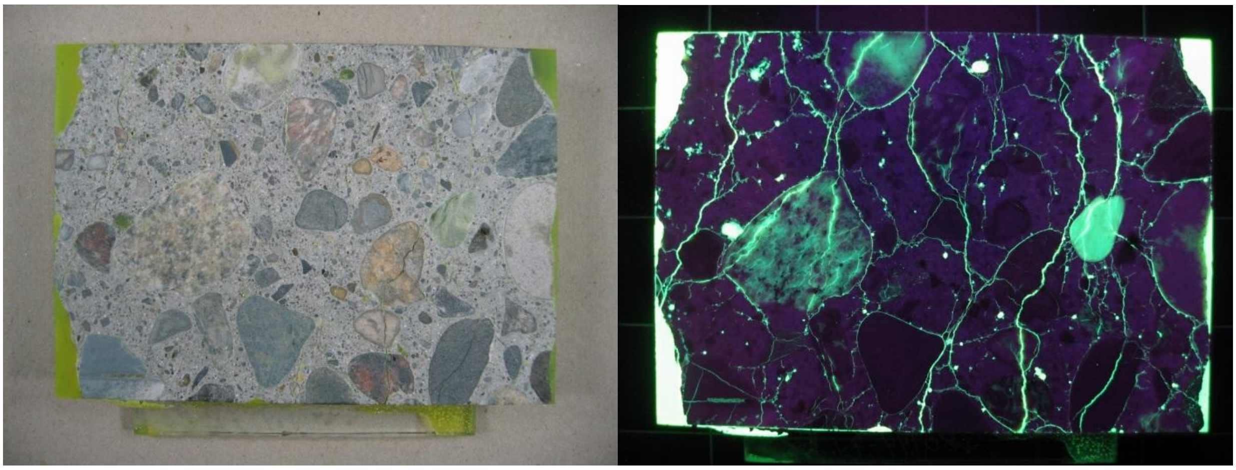


Figure 4-5 - Photos of a plane polished sections in normal light and UV light, respectively. Cracks caused by ASR and air voids are easily visible in UV light.

In Norway, the extent of cracking is in most cases described verbally. However, different methods for quantifying the cracking have been used during the last years. One of these methods [8] has so far given promising (reliable) results for laboratory specimens. Image SXM, a public domain image processing and analysing software, was used to create maps of the cracks in the plane polished sections through thresholding of the grey level histogram. The input images were acquired in fluorescent light (Figure 6), so the cracks were easily differentiated from the aggregates and the cement paste. The careful image acquisition (Figure 7) guaranteed that the brightness and contrast were reproducible and that the corresponding digital pixel values were stable. The output was a number representing the area% occupied by cracks in the image, the so-called "cracking intensity". With this method it is easy to compare the crack intensity between different specimens and internally within each specimen. However, care should be taken when using this method. The image analysis method only measures the intensity of cracking, without reference to the origin of cracking. Consequently, the image analysis technique is essentially a tool to complement petrographic examination by allowing quantification of the degree of damage, but not to diagnose the source of the cracking.

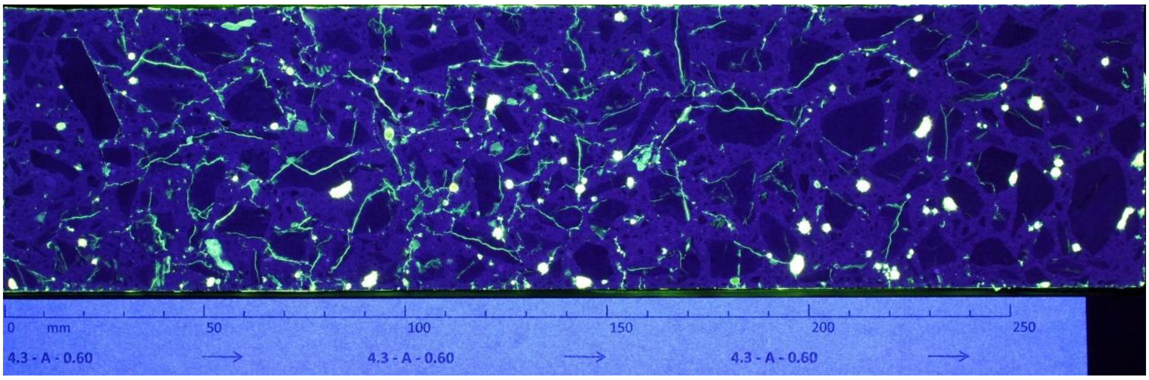


Figure 6 – Photo of a plane polished section in UV light.

Figure 7 – Crack map of the specimen in Figure 6.

2.4 Thin section analysis

The other half of the concrete core is used to prepare fluorescent impregnated thin sections from selected areas in the concrete. The thin sections are examined in a polarization microscope with different lightening techniques. Microscopy of thin sections gives valuable information about the concrete quality, i.e. with respect to porosity, air content, aggregate (rock types), cracks, precipitations, carbonatization and damage mechanisms. Cracks and air voids are easily visible in fluorescent light, one of the possible lightening techniques. Figure 8 shows a photo from a thin section in normal light, with a crack in an aggregate particle and alkali silica gel in air voids. Figure 9 shows a picture from a thin section in UV light. In this sample there are cracks in aggregate particles.

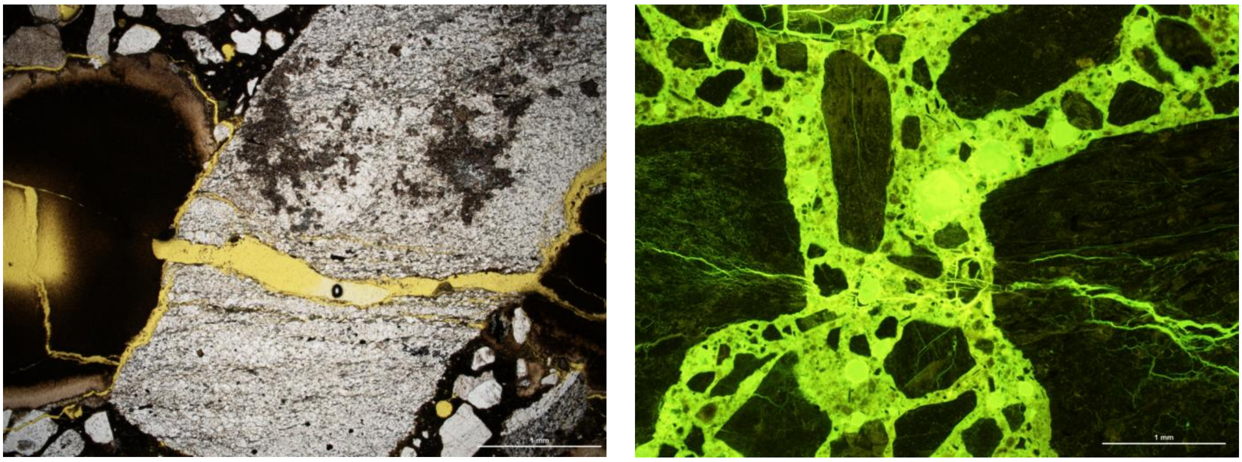
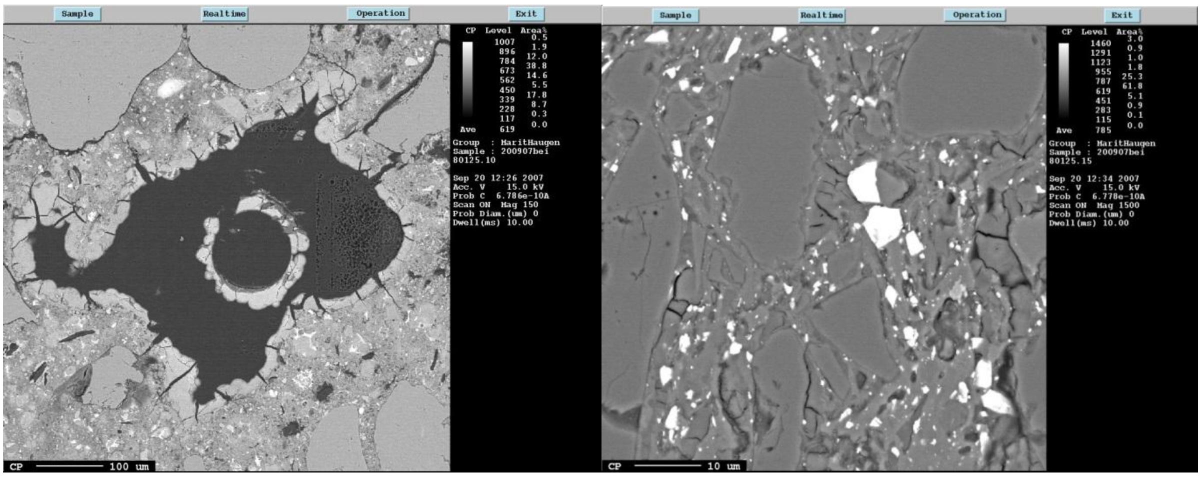


Figure 8-9 – Photos of a fluorescence impregnated thin section, to the left in normal light and to the right in fluorescent light. Cracks are easily visible. In the left picture alkali silica gel is visible in air voids.

2.5 Scanning electron microscopy (SEM)

Scanning electron microscopy (SEM) is used occasionally when details are important, i.e. documentation of reaction products and their composition. Figure 10 and 11 show photos from a concrete with alkali silica gel in an air void and in the cement paste.



```
 CP  Level  Area%
                0.5
        1007    1.9
         896   12.0
         784   38.8
         673   14.6
         562    5.5
         450   17.8
         339    8.7
         228    0.3
         117    0.0
 Ave     619
Group  : MaritHaugen
Sample : 200907bei
80125.10

Sep 20 12:26 2007
Acc. V    15.0 kV
Prob C  6.786e-10A
Scan ON  Mag 150
Prob Diam.(um) 0
Dwell(ms) 10.00
```

```
 CP  Level  Area%
                3.0
        1460    0.9
        1291    1.0
        1123    1.8
         955   25.3
         787   61.8
         619    5.1
         451    0.9
         283    0.1
         115    0.0
 Ave     785
Group  : MaritHaugen
Sample : 200907bei
80125.15

Sep 20 12:34 2007
Acc. V    15.0 kV
Prob C  6.778e-10A
Scan ON  Mag 1500
Prob Diam.(um) 0
Dwell(ms) 10.00
```

Figure 10-11 – SEM photos of a sample with ASR. In the picture to the left, alkali silica gel is visible in and along the boundary of an air void. In the picture to the right, alkali silica gel is visible in the cement paste.

3. PETROGRAPHIC ANALYSIS

The collected field information regarding the alkali reactivity of Norwegian aggregates resulted in a list of alkali reactive rocks and later a corresponding Norwegian "petrographic atlas" [1, 2, 3, 4, 9]. The experience showed that the mineralogy and the microstructure determined the reactivity, and hence thin section microscopy was required. In addition, the Norwegian alkali reactive rocks are typically very fine grained, consequently identification and classification is very uncertain without thin section microscopy. Figure 12-14 illustrate this. Just by looking at the surface of a selection of aggregate particles it is impossible to give a reliable classification of the different rock types. However, by use of thin section microscopy, the different optical and physical properties of the minerals appear in the polarization microscope, and hence an exact classification of the various rock types and assessment of the potential alkali reactivity is possible.

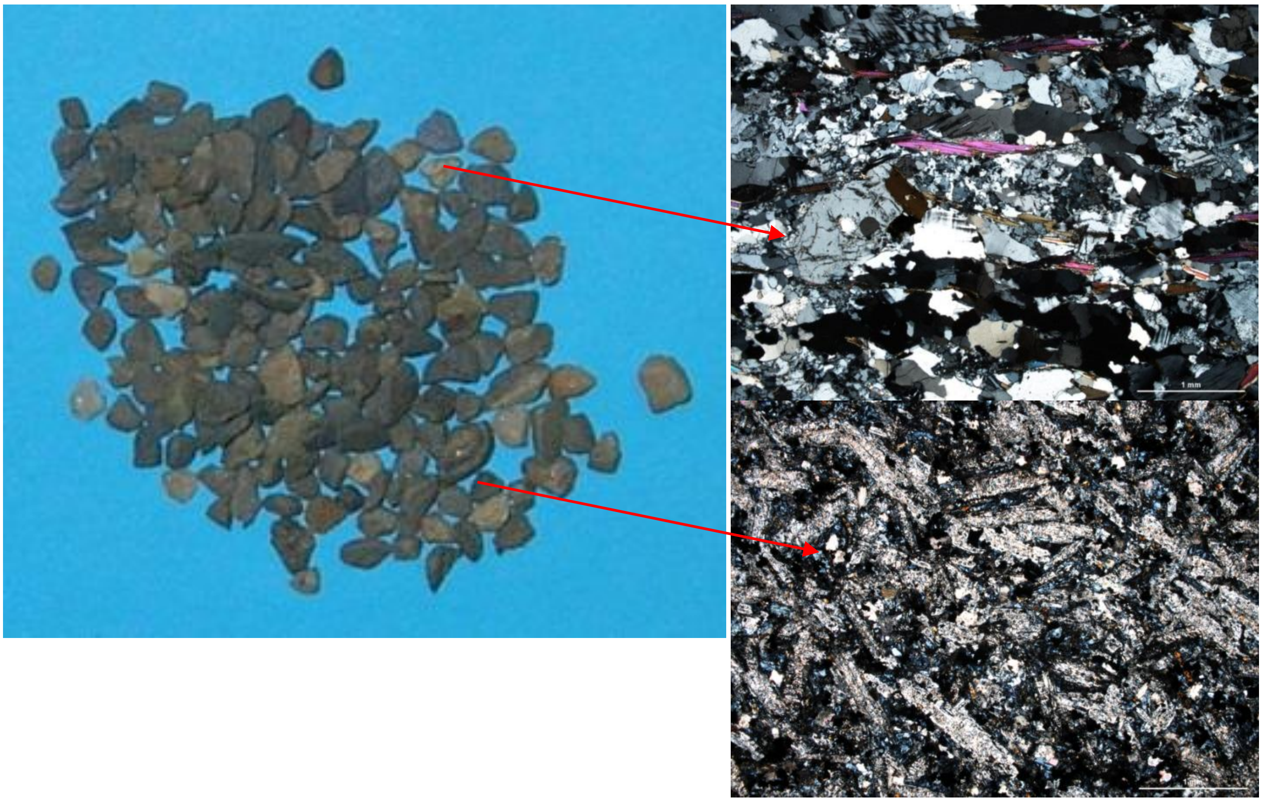


Figure 12-14 – The picture to the left shows a group consisting of 12 different rock types. Thin section microscopy makes an exact classification of the various rock types possible.

Petrographic analysis of aggregates with point counting in thin section as a first step in evaluating the aggregate reactivity has since 1992 been part of "the Norwegian ASR testing system" [7, 10]. The Norwegian petrographic method [7] formed the basis for the RILEM AAR-1 method, published in 2003 [11]. It is of great importance that the assessment of reactive constituents follows national or regional experience, recommendations and specifications. Furthermore, the analysis has to be performed by an experienced geologist/petrographer.

In Norway, there is a "critical limit" of 20 % of alkali reactive rocks for fine aggregates and 10 % for coarse aggregates. If the content exceeds these limits, further "expansion testing" can be performed. However, in most cases the petrographic analysis is used as the only testing method for assessment of the alkali reactivity of aggregates.

To make a reliable quantitative classification of the aggregate composition in a concrete, a new method for separating aggregates from the concrete was developed during one of the research projects [6]. A water saturated core was placed in liquid nitrogen, and then heated in a micro-wave oven. The number of cooling/heating cycles needed depended on the concrete quality. Between each cycle the concrete was hammered, and the stones loosened from the paste. After sieving, sand and stone were washed in a acid solution. Limestone in the stone fraction was identified in advance (may be dissolved in the acid). Separated sand and stone particles were prepared for petrographic analysis and point counted in thin sections according to the same procedure as used for virgin aggregates. The experience after examination of about 50 concrete structures is that the separation technique is a very good tool. In most cases the operator was able to recognize the origin of the aggregates, and comparison of the results from the petrographic analysis of virgin and separated aggregates were for most cases almost identical.

From 2002-2006, SINTEF took part in an EU project named "PARTNER". A part of this project was to perform petrographic analyses of 22 concrete aggregates from 10 different European countries [12]. These analyses demonstrated the importance of experience. The results from the experienced petrographers showed acceptable correlation, while the results from the inexperienced petrographers deviated too much (se Table 1) [13]. The project also established a good network between petrographers and flow of experience across the borders. The work resulted in a petrographic atlas, which is a helpful tool for new petrographers (Figure 15) [14].

Table 1 – Results from precision trials in the EU "PARTNER" project (three aggregate types with various particle sizes). The laboratories written in bold are experienced. These results are more even compared with the results from inexperienced laboratories.

Lab.	**S**	**S**	**S**	**D**	**D**	**D**	**N**
	2-4	**1-2**	**0.063-1**	**2-4**	**1-2**	**0.063-1**	**2-4**
Lab 2	99	94	71	80	66	86	30
Lab 3	12	16	7	38	29	16	13
Lab 4	**44**	**34**	**18**	**52**	**33**	**4**	**25**
Lab 5	**53**	**39**	**29**	**43**	**26**	**4**	**20**
Lab 8	99	99	88	76	74	65	100
Lab 9	**57**	**54**	**26**	**52**	**32**	**3**	**25**
Lab 15	30	26	14	16	11	2	21
Lab 22	**42**	**38**	**27**	**48**	**32**	**9**	**32**
St. dev	30.8	30.7	28.8	20.5	21.2	32.8	27.6
Mean	55	50	35	51	38	24	33
Variance	57	61	82	40	56	139	83

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GEOLOGICAL SURVEY OF BELGIUM
PROFESSIONAL PAPER 2006/1 – N. 302

PETROGRAPHIC ATLAS
OF THE POTENTIALLY ALKALI-REACTIVE ROCKS IN EUROPE

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Figure 15 – Petrographic atlas prepared in the PARTNER project [14].

4. SUMMARY

Structural analysis of drilled cores concrete from real structures is very important for understanding the behaviour of the different rock types in field concrete and provide information about the alkali reactivity of the rocks types in the actual country. Laboratory analyses of concrete aggregates are important, but the "true answers" are given by field behaviour.

SINTEFs main experience after more than 20 years of application of the petrographic method on Norwegian aggregates, as well as on a whole range of foreign aggregates, is that it is a very reliable tool for evaluating the alkali reactivity of aggregates. However, a prevision is that it is performed by an experienced petrographer.

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Petrographic Analysis for Assessment of AAR in Norway: Precondition, Methods, Reliability and Experiences

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ABSTRACT

The background and history of research on Alkali Aggregate Reaction (AAR) in Norway and Norwegian requirements for assessment of concrete aggregates is given in the paper. Since 1993 all Norwegian concrete aggregates have been classified according to alkali reactivity by a petrographic method (NB 32) using point counting in thin sections similar to the Rilem AAR-1 "Petrographic Method" published in 2003. The reliability and errors is influenced by four major procedures: 1) sampling in the aggregate deposit, 2) errors introduced in the laboratory, 3) the counting procedure and 4) operator uncertainty. The total error is given by pooling all the sub standard deviations by an equation. Examples on the reliably of the petrographic analysis from literature and 10 years commercial testing by the Norwegian Concrete and Aggregate laboratory (NBTL) is given and suggest the reliability of the method analysing 300 particles is "good enough". The total error is only marginally improved counting more particles.

Keywords: concrete aggregate, Alkali Aggregate Reaction (AAR), Norwegian specification to control AAR: NB 21 and NB 32, petrographic analysis, reliability, RILEM AAR-1.1

1 BACKGROUND AND HISTORY

The first documented case of Alkali Aggregate Reaction (AAR) in Norway was reported in 1978 [1] but AAR was first accepted as a real concrete problem in the early 1990th. National research projects have given the necessary scientific and technical knowledge to deal with this problem and for recommendations of preventive and remedial measures. The first national research project 1990-1993 "Alkali Aggregate Reaction in Southern Norway" deals with survey investigations of structures in Southern Norway, micro structural examination of concrete (diagnosis), alkali reactive aggregates (negative list), laboratory test methods (establishment of test methods) and rehabilitation [2,3,4]. The project include Viggo Jensen's doctoral project [5] and was basis for the first Norwegian recommendation on AAR [6,7,8]. The second national research project 1993-1996 "Alkali Aggregate Reaction in Northern Norway" deals with survey investigations of structures in Northern Norway, micro structural analysis, assessment of the petrographic method and a new in-situ system for measurements of relative humidity and expansions in structures [9, 10, 11]. By this project the "survey" of AAR in Norway was completed. Two succeeding projects 1998-2000 "Normin 2000" [12] and 2003-2006 "Field examination" [13]have given basis for revision of the Norwegian recommendation on AAR "NB 21 2004" and "NB 32 2005". [14, 15].

2 SPECIFICATIONS TO CONTROL AAR

The Norwegian Standard NS 3420, L5 [16] in use in the 1990th specified "that reactive aggregates in harmful amounts should not to be used in concrete", but no methods or recommendations to minimise the risk for ASR was presented in the standard. Because AAR recently was found to be a concrete problem in Norway recommendations and guidelines how to deal with AAR was urgently needed.

2.1 Norwegian Concrete Society publication NB 19 and Optional arrangement DGB

In 1991 the Norwegian Concrete Society - aggregate committee, proposed methods for control and acceptance of concrete aggregates in publication NB 19 [6]. Testing of aggregates for AAR was to be carried out in 3 successive steps where the test result from the succeeding tests overruled the former result: 1) petrographic analysis, 2) accelerated mortar bar test, 3) concrete prism test. The petrographic method is based on "grain counting" of reactive constituents similar to ASTM C295 [12].

In 1992 an optional arrangement for acceptance and approval of aggregates for concrete were introduced in Norway named DGB (Deklarasjon- og Godkjenningsordning for betongtilslag) [7]. Aggregates were to be tested according to the procedures given by publication NB 19 [6].

2.2 Norwegian Concrete Society specification NB 21

In 1996 the Norwegian Concrete Society published a specification NB 21 [8] for production of durable non reactive concrete with use of alkali reactive aggregates. The NB 21 specification was advisory and based on resent knowledge and experiences with the ASR problem in Norway. Research had also revealed that potentially alkali reactive aggregates were wide spread and common in Norway

Testing of aggregates shall be carried out in 3 successive steps where the test result from the succeeding tests overruled the former result:

1. Petrographic analysis, point counting of thin sections similar to RILEM AAR-1 (2003) [18]
2. Norwegian mortar bar test, modified from NBRI accelerated mortar bar test [19]
3. Norwegian concrete prism test modified from Canadian standard CSA A23.2-14A [20]

All aggregates shall first be assessed by the petrographic analysis and the alkali reactivity can be assessed here. It is optional to use the Norwegian mortar bar test and Norwegian concrete prism test. The results from the succeeding tests overrule the result from previous tests. In practice all concrete aggregates are assessed by the petrographic method in Norway.

In September 2004 a revised NB 21 specification was published as a compulsory specification to be followed by all concrete and aggregate producers in Norway [14]. This because the Norwegian national application document (NA) for the European product standard NS-EN 206 “Concrete” [21] and NS-EN 12620 “Concrete aggregates” [22] require production of non alkali reactive concrete and because the Norwegian National Application document (NA) refer to the revised NB 21 publication. It has to be mentioned that the European standard NS-EN 12620 (concrete aggregates) recommends that AAR should be dealt on a National level with National requirements and test methods.

2.3 Norwegian Concrete Society publication NB 32 from 2005

NB 32 publication is part of the NB21 “system” and describes test methods to be used in Norway [15].

3 PETROGRAPHIC METHOD AND REQUIREMENTS TO PREVENT AAR

3.1 Grain counting versus point counting

A “grain counting” method, where aggregates were grouped into different rock groups (inclusive potentially alkali reactive rocks) was in use in Norway from 1990 to 1993. The method identified rock and minerals visual by use of the naked eye, lupe and stereo microscope. When necessary, selected particles were mounted into thin sections for further identification. The major uncertainty with the method was that Norwegian alkali reactive rock types in most cases are dense or fine grained and can therefore not be classified correctly without use of thin section microscopy. Moreover, micro textures as cataclase (mylonite) and micro-crystalline crystal sizes important for the susceptibility for AAR is very difficult to identify without use of thin sections.

Figure 1 shows test results of percent alkali reactive rocks obtained by grain counting respective point counting in thin sections from 29 different Norwegian quarries [23]. Samples for grain counting and point counting were sampled in the same quarry with about one year intervals. Note that an “acceptable” correlation exists but that grain counting obtains significant higher content of potentially alkali reactive aggregates compared to the point counting method.

Experiences and testing revealed that the grain counting method could not be considered reliable as documentation for alkali reactivity of Norwegian concrete aggregates.

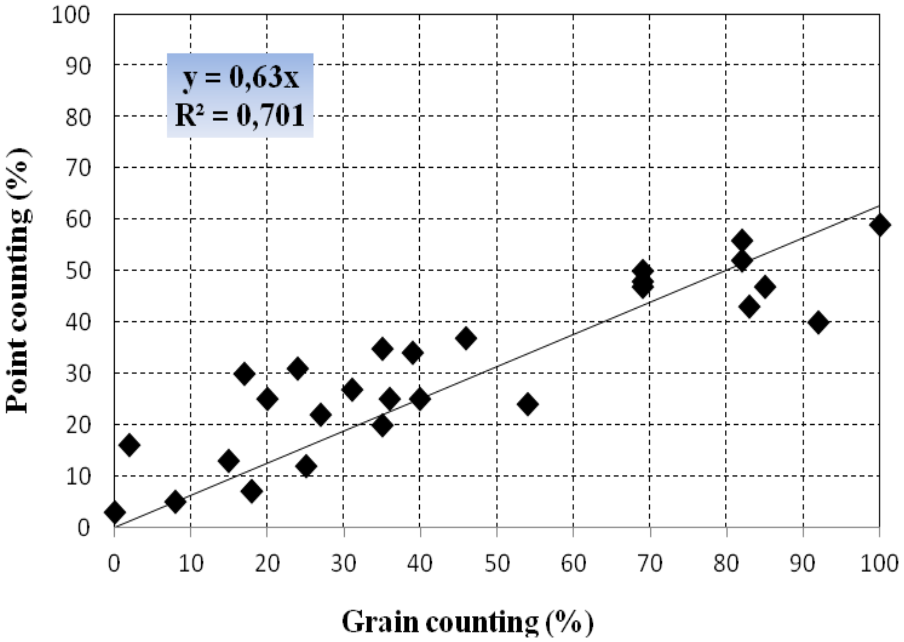

Figure 1 – Percent reactive aggregates obtained by grain counting versus point counting from the same quarry

In 1993 the grain counting method for commercial testing of concrete aggregates was replaced by point counting in thin sections. This method was implemented in the optional acceptance and approval agreement DGB [12] and in the concrete specification NB 21 in 1996.

3.2 Effect of crushing

Assessment of coarse aggregates by the point counting methods requires that the coarse aggregate particle size is initially reduced to 2-4 mm by crushing before mounting in thin sections. Hereby will a sufficient number of particles be examined and the reliability of the test will be improved.

Figure 2 shows the correlation of percent alkali reactive rocks obtained by point counting sand and coarse aggregates from 37 different Norwegian quarries [23]. Samples of sand and coarse aggregates were sampled in the same quarry. The coarse aggregates have been crushed and sieved before being mounted into thin sections. Fine aggregates (sand) have only been fractionated by sieving before mounting in thin sections.

Note that a good correlation exists between the coarse and fine fraction from the same quarries, and crushing of coarse aggregates gives similar content (relationship 1: 1) of alkali reactive aggregates compared to fine aggregates (sand), which only have been sieved. It can therefore be concluded that crushing of Norwegian aggregates not influence the content of alkali reactive constituents between coarse and fine aggregates.

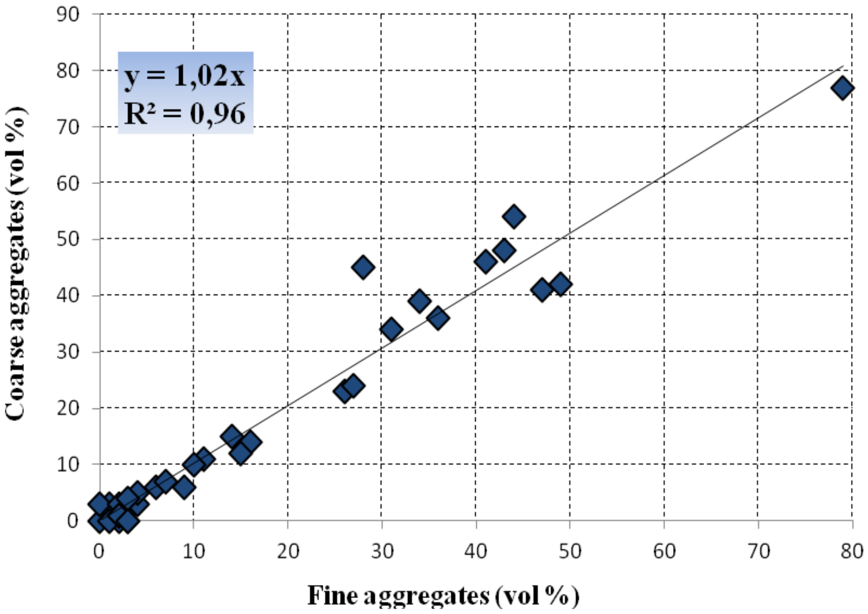

Figure 2 – Percent alkali reactive aggregates in coarse and fine aggregates from same quarry

3.3 Norwegian point counting method

The Norwegian point counting method [15], list of potentially alkali reactive aggregates [15], comparison value and limit values are briefly described in the following [14].

Number of thin sections
For natural sand and manufactured sand representative sub samples of the 1-2 mm size fraction and 2-4 mm fraction are moulded into normal size thin sections, one thin section of the 1-2 mm fraction (600-800 particles) and 2 thin sections of the 2-4 mm fraction (about 300 particles), total about 1000 particles.

For coarse gravel and crushed rock the fragments or particles are stepwise crushed down. Sub samples of the 2-4 mm fraction is separated and moulded into 2 thin sections for point counting (about 300 particles).

Point counting is carried out on the thin sections. Minimum 1000 points are counted on each size fraction (1000 points for coarse fractions and 2000 points for sand fractions). All major rock types and minerals are recorded and grouped. Results are given as average percent of rock – and minerals of total counted points.

Potentially alkali reactive aggregates (risk aggregates)
Identification of reactive and potentially reactive rock types is done from an aggregate list in NB 32 [12] which has changed very little since 1993, see table 1. The sum of all identified reactive + potentially reactive rock types is called “risk aggregates” and used for assessment of alkali reactivity.

Table 1- alkali reactive, potentially reactive and innocuous aggregates in Norway

Risk aggregates		Non reactive aggregates
Alkali reactive aggregate	*Potentially reactive aggregate**	*Innocuous aggregate*
Sedimentary rock:	*Quartzite*	Mafic rocks
Sandstone	*Quartzite shiest*	*Basalt*
Arkose	*Rocks with > 20 % quartz*	*Greenstone*
Quartz sandstone	*Impure carbonate rock*	*Gabbro*
Siltstone	*Hornfels*	*Amphibolite*
Clay stone/shale	*Mylonite with 1%-5% quartz*	
Marl		Quartz containing rocks
Greywacke		*Granite/gneiss*
Cataclastic rocks:		*Quartzite*
Mylonite (> 5% quartz)		*Quartz shiest*
Cataclasite(with quartz)		*Mica shiest*
Mylonite gneis		Felspatic rocks
Acid volcanic rocks:		Other/unidentified
Rhyolite		*Pure carbonate rock and marble*
Quartz keratophyre		
Other rock types:		*Other non alkali reactive rocks*
Quartzite		
Phyllite		*Porphyric rock*
Quartz schist		*Mylonite without quartz*
Grain/crystal size < 60 µm**	Grain/crystal size 60-130 µm	Grain/crystal size >130 µm

* called questionable aggregates ** Not for sedimentary rocks

Comparison value and limit values

After September 2004 the assessment of alkali reactivity shall not only be based on a single analysis but from a calculated comparison value including up to 6 previous results from the deposit since 1993 and added a safety value [14]. For the petrographic analysis the critical limit value for assessment of alkali reactivity is 20,0 % Sv (comparison value). Therefore the limit value for the actual content of risk aggregates analysed in the sample will vary depending on how many previous analyses there have been used for the calculation of Sv. For aggregates the following limit values for risk aggregates is required depending on how many analyses there have been carried out [14], [24], se Table 2.

Table 2 - Limit values of risk aggregates for 1 analysis and more than 6 analyses

Fraction	Risk aggregate 1 analysis	Risk aggregate ≥ 6 analyses
Limit value for fine aggregate (sand)	15 %	20 %
Limit value fine stone (d >1mm, D=4-8mm)	*10 %*	*15 %*
Limit value for coarse aggregate	7.5 %	10 %

Approving of operators

All operators carrying out petrographic analysis for assessment of alkali reactivity on a commercial basis shall be approved by the certifying council (Kontrollraadet). New operators have to follow a comprehensive study and shall be examined before approval [14]. The certifying council (Kontrollraadet) have since year 2000 initiated round robin test each year for all approved operators [23].

4 STATISTICS

The reliability of the petrographic analysis is influenced by the succeeding procedures necessary to make a petrographic analysis; from the sampling in the aggregate deposit to the examination in the laboratory. Repeated sampling over several years gives in addition the variation in the deposit. Errors are influenced by four major procedures, which can be subdivided into sub procedures (not included here), Jensen [25], [26]:

1. Sampling in the aggregate deposit (how representative is the sample?).
2. Errors introduced by laboratory (splitting, crushing, sub-sampling and preparation of thin sections).
3. The counting procedure (number of particles, thin sections, points and statistical errors)
4. Operator uncertainty (the ability of the operator - the largest error)

The standard deviation is a good measure for the reliability and should be measured and calculated for all the four major procedures. To find the total error of the petrographic analysis the standard deviation (variance) from each major procedure should be "pooled" and the resulting standard deviation calculated according to equation 1.

$$S_{resulting} = \sqrt{S^2_{deposit} + S^2_{lab} + S^2_{counting} + S^2_{operator}} \qquad S^2 = variance \qquad (1)$$

Howarth [27] published "asymmetrical" two sided error charts which are reliable especially for very low and very high target values. The charts are intended used for estimating the certainty of proportions by the counting method based on a two sided 95% confidence interval. It has to be mentioned that a two sided 95 % confidence interval more or less is similar to 4 standard deviations. Therefore the one sided error % is more or less equal to 2 standard deviations. The charts can be used for target values or number of points up to 5000. For most petrographic methods a given number of standard size thin section are specified which include e.g. 300 or 1000 particles [15], [18].

Figure 3 show the distribution of absolute errors % for 1000 and 300 counted points/particles measured from the graphs by Howarth [27] by Jensen [25] [26]. Note that the graphs are asymmetrical. For 300 points/particles and a target value on 10% the absolute error % are is 10 % -3.2 %/+4 % equal to the 95% confidence interval 6.8 % –14 %. For 1000 particles/points and a target value on 20 % the absolute error % is 20 % -2.4 %/+2.6 % equal to the 95% confidence interval 17.6 %–22.6 %.

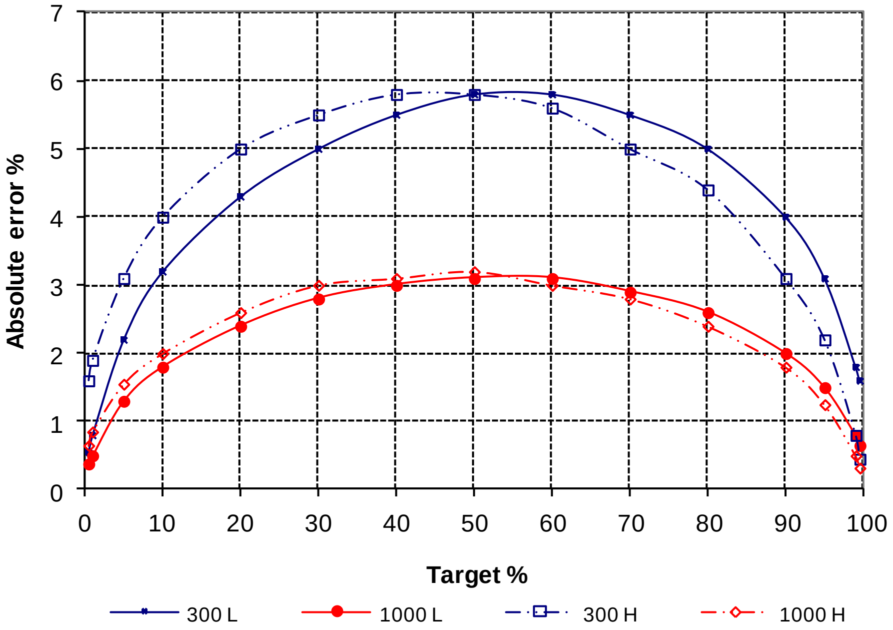

Figure 3 - Absolute error % relative to number of points/particles counted (target %). Broken lines are upper half-width of the 95 % confidence bound and full lines the lower half-width of the 95 % confidence bound. L is lower half-width and H is upper half-width.

Table 3 gives the standard deviation (absolute error % = 2 standard deviations) for counting 300 particles and 1000 particles with target values 10 % and 20 % (limit values according to the Norwegian requirement for more than 6 analyses NB 21) [14].

Table 3 - Measured standard deviations from figure 1

Target %	Particles	2 Standard deviation		Differences 300-1000 particles	
		Lower half-width $2\ S_{counting}$	Upper half-width $2S_{counting}$	Lower half-width $2\ S_{counting}$	Uppper half-width $2S_{counting}$
10	300	3.2	4.0		
10	1000	1.8	2.0	1.4	2.0
20	300	4.2	5.0		
20	1000	2.4	2.6	1.8	2.4

Note: absolute error % is equals to 2 standard deviations

Most published reliability tests or round robin test of petrographic methods are based on counting same thin sections or sub samples and results are given as averages and standard deviations [25], [28]. Point counting on same thin sections or counting of constituents from same sub samples with same procedures gives the operator uncertainty and not the true or total uncertainty. Round robin tests for commercial petrographic testing are required for approved operators in Norway. The average standard deviation on same thin sections from round robin tests since 2000 has been calculated to be 3.8 % exclusive results from one inexperienced operator [25].

Table 4 gives calculated results according to equation (1) for the resulting standard deviation $S_{resulting}$ using $S_{counting}$ from table 3 and $S_{operator}$ = 3.8. The standard deviations (error) is unknown for $S_{deposit}$ and S_{lab} but is in the example given in table 2 assessed to be 2 standard deviations

Table 4 - Example on calculations of resulting standard deviation

Target %	Particles	2 Standard deviation		Differences 300-1000 particles	
		Lower half-width $S_{resulting}$	Upper half-width $S_{resulting}$	Lower half-width $S_{resulting}$	Uppe half-width $S_{resulting}$
10	300	5,0	5.1		
10	1000	4.8	4.8	0,2	0,3
20	300	5.2	5.4		
20	1000	4.9	4.9	0,3	0,4

Note: using higher values for deposit and lab the resulting standard deviation only increases not the differences

Note that counting 1000 particles instead of 300 particles reduce the 2 standard deviations (errors) with 1.4 to 2.4 (standard deviation 0.7 to 1.2) depending on target % and number of particles. However, the differences on the resulting standard deviations using 1000 particles respectively 300 particles gives a total reduction only on 0.2 to 0.4. Therefore improving the counting statistic by counting more particles e.g. using more thin sections only gives marginal improvements of the resulting error.

5 10 YEARS EXPERIENCES WITH THE PETROGRAPHIC METHOD AT NBTL

The Norwegian Concrete and Aggregate laboratory (NBTL) has carried out petrographic analyses on a commercial basis in more than 10 years according to the Norwegian method NB 32 [15] and RILEM AAR-1 (2003) [18]. Test results from more than 10 years commercial testing for fine aggregates (sand) containing about 1000 particles and coarse aggregates containing about 300 particles has statistically been analysed. Results do not confirm a better reliability for sand compared with coarse aggregates as described in the paper. This because errors by the petrographic analyses not only is caused by the counting statistic but in addition influenced by errors caused by the sampling, laboratory errors and the operator’s ability.

The following figures shows analyse results from same aggregate deposits which have been examined 2-6 times during a period up to 10 years by NBTL and by same operator (V. Jensen). Aggregates have been delivered the laboratory as natural sand mostly as 0-8 mm fraction and coarse gravel and crushed rock mostly as 8-16 mm fraction. The figures show the average content of alkali reactive aggregates (risk aggregates) according to Norwegian requirement and calculated standard deviation.

Figure 4 show results from 36 natural sand deposits and 26 results from coarse gravel and coarse crushed stone deposits. Note that coarse gravel and coarse crushed stone visually not obtain higher standard deviations even only about 300 particles have been counted for the coarse fractions compared with about 1000 particles for fine aggregates (sand). Average standard deviation was calculated to be 1.8 for sand and 1.7 for coarse gravel and coarse crushed stone.

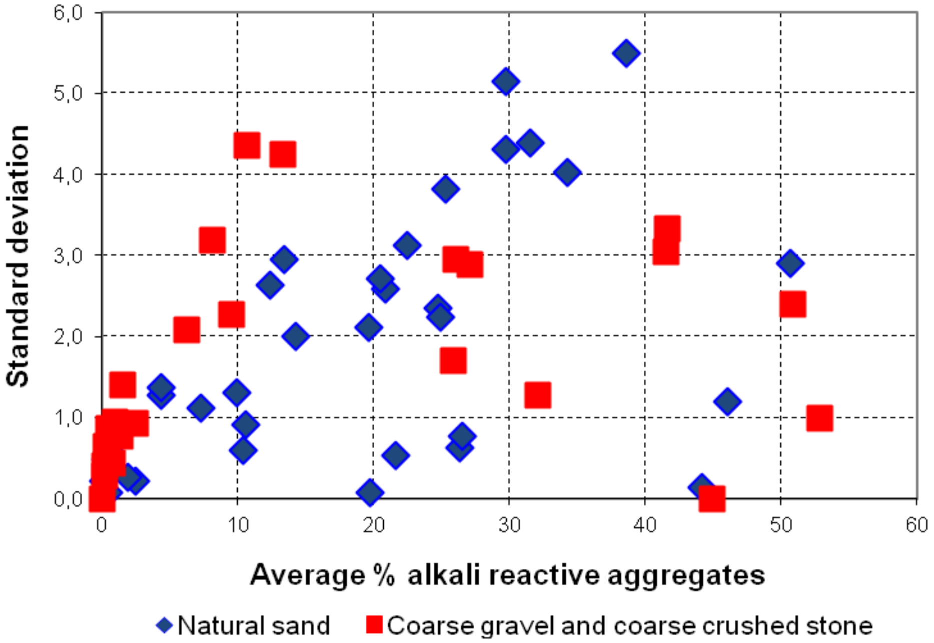

Figure 4 - Standard deviation and average % alkali reactive aggregates from 36 natural sand deposits and 26 results from coarse gravel and coarse crushed stone deposits (NBTL).

Figure 5 show results from same 14 natural aggregate deposits where both sand and coarse gravel have been analysed.

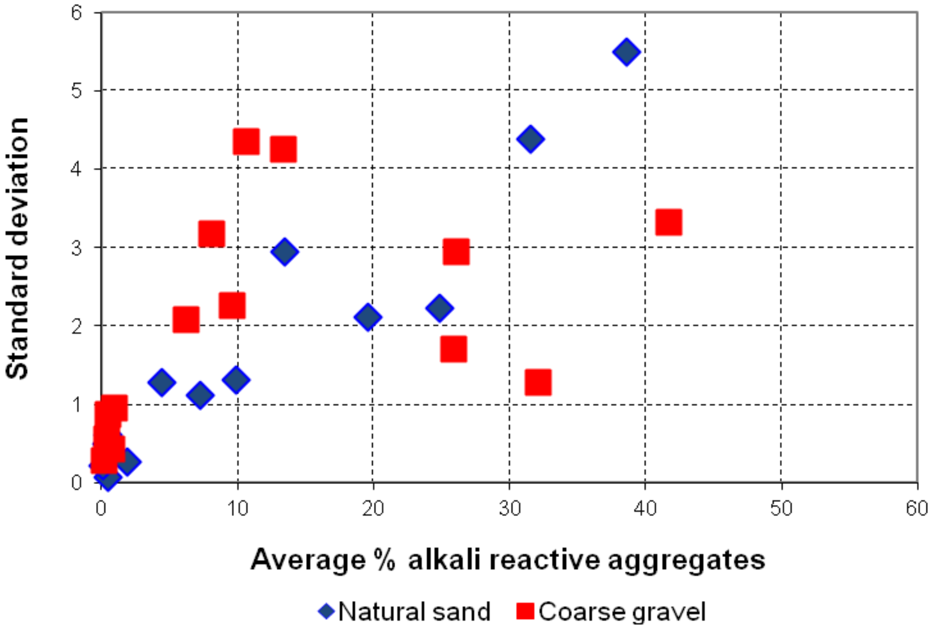

Figure 5 - Standard deviation and average % alkali reactive aggregates from same 14 natural sand and coarse gravel deposits (NBTL).

Note that coarse gravel with about 300 particles visually not obtains higher standard deviations compared to fine aggregates (sand) with about 1000 particles. Average standard deviation for sand has been calculated to 1.9 and for coarse gravel 2.0.

Figure 6 show the correlation between average % alkali reactive aggregates from fine aggregates (sand) and coarse gravel from the same aggregate deposits. Note that coarse gravel which has been crushed and sieved to 2-4 mm fraction has an excellent correlation $R^2 = 0.96$ with sand which only has been sieved.

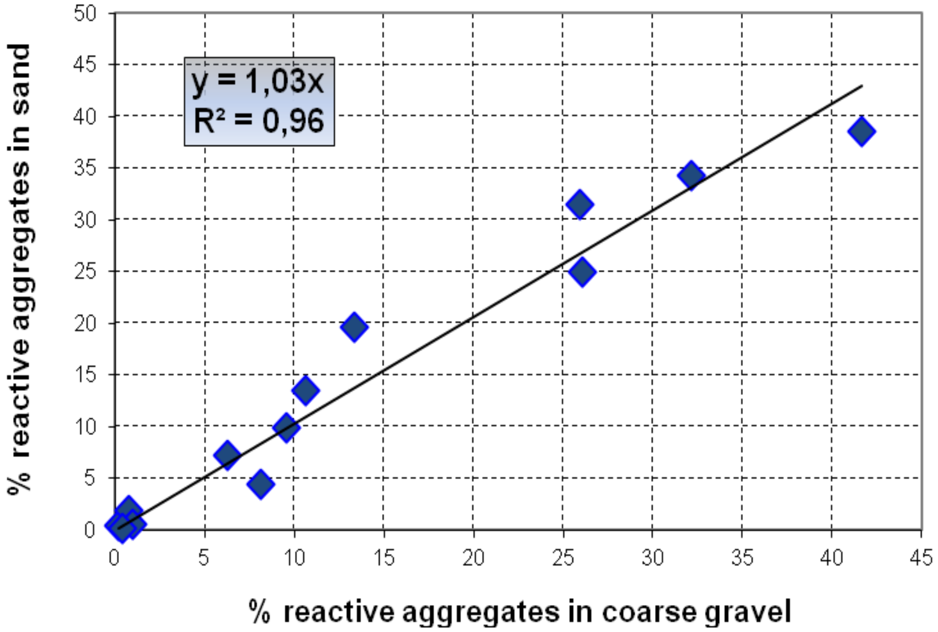

Figure 6 - Correlation of 14 natural sand and coarse gravel from same deposits (NBTL).

6 CONCLUSION

In Norway all concrete aggregates shall be assessed for alkali reactivity by a petrographic method NB 32 [15] similar to RILEM Method AAR-1: 2003[18]. It is required that aggregates first are assessed by the petrographic analysis and it is optional to use the Norwegian mortar bar test and/or Norwegian concrete prism test which can overrule the results from the petrographic method. In practice all Norwegian concrete aggregates are assessed only by the petrographic method. The statistical error counting 300 particles for coarse aggregates is assessed to be “good enough” because counting more particles only gives marginally improvement on the total error as shown in the examples given in the paper. It can be questioned if it is necessary to analyse the 1-2 mm fraction in sand as required today because the total error is not improved counting more particles. Some improvements in the present requirement NB 21 and method NB 32 can be suggested e.g. how to deal with innocuous aggregates as granite, quartzites and carbonates which occasionally have been reported to cause deleterious AAR in concrete structures in Norway.

AFTER NOTE

RILEM AAR-1: 2003 “Petrographic method” is under revision (at present time unpublished (January 2014) and now called RILEM AAR-1.1) [29]. The principles and procedures in the method for coarse aggregates will be changed significantly. The new RILEM AAR-1.1 method recommend for coarse aggregates a similar method as the “grain counting method” formerly used and not recommended to be used in Norway. This because the grain counting method not is able to identify important micro structures (cataclase) and micro crystalline crystal sizes important for the susceptibility for alkali reactivity in all the examined aggregates. Testing in Norway has also shown that the grain counting method obtains higher content of alkali reactive constituents compared to the point counting method. A “vague” formulation in the text open op for use of crushing coarse aggregates: “Crushing of coarse aggregate is not usually required in preparation for petrographic examination. However, crushing can sometimes be useful for quantitative assessment of bulk coarse aggregate materials > 4mm using thin-section petrography and point counting”. RILEM AAR-1.1 recommends examination of the fraction 0.063 mm – 1 mm alternatively 0.063 mm – 2 mm together with the coarser fractions (1-2 mm and/or 2-4 mm). NB 32 and RILEM AAR-1 (2003) and for coarse aggregates only recommend testing of thin sections with the 2-4 mm fraction because target is rock aggregates and not individual minerals which will be increased in the finer fractions. It can therefore be discussed the Norwegian petrographic method NB 32 in the future will be “similar” and “compatible” to the new RILEM Petrographic Method. Moreover, RILEM AAR-1.1 only mentions the counting statistic to be the error of the method, not the total error as discussed in the paper.

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Alkalisilica-reactivity of Swedish aggregates used for concrete

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ABSTRACT

23 rock samples were analyzed using RILEM AAR 1, a petrographic examination based on microscopic point-counting; and RILEM AAR 2, an ultra-accelerated mortar-bar test. Petrographic examination by RILEM AAR 1 detects potentially reactive aggregates, whereas the expansion of the mortar-bars gives a dimension of the alkalisilica-reactivity of the aggregates. The two methods correlate and alkalisilica-reactive aggregates, with expansion ≥0.25%, include metavolcanic rocks, deformed granitoids, mylonites, and sandstones. Non-reactive rocks, with expansion well below 0.25%, are primarily 'undeformed' or weakly foliated granitoids. Potentially reactive aggregates, with an expansion between 0.19-0.24%, are generally granitoids with moderate degrees of deformed or microcrystalline quartz.

Key words: aggregate, alkalisilica-reaction (ASR), concrete, petrography, RILEM AAR 1, RILEM AAR 2

1. INTRODUCTION

The Swedish authorities are demanding a decrease in the usage of natural aggregates to safeguard natural water deposits against exploitation. Crushed rock aggregates are often suitable as replacement material in concrete, although sometimes crushed aggregates, as well as natural aggregates, can cause alkalisilica-reactions (ASR) in the concrete. The Swedish Standard SS-EN 12620 - Aggregates for concrete [1] stresses the relevance of alkalisilica-reactivity in aggregates in conjunction with cement. The Swedish standard SS137003 Concrete - Application of EN 206-1 in Sweden [2] gives guidance on acceptable contents of potentially alkali-reactive particles within the aggregates. Using the RILEM AAR 1 petrographic method, the content of potentially slow or medium alkali-reactive constituents should be less than 15vol% for an aggregate to be defined as low-reactive in Sweden, whereas for the RILEM AAR 2 ultra-accelerated mortar-bar test, the recommendation is a maximum expansion of 0.25% for mortar bars with dimensions 40x40x160mm under testing for 28 days.

During 1995-2011 the Geological Survey of Sweden carried out extensive aggregate quality mapping in Sweden. As a part of this survey, the Swedish Cement and Concrete Research Institute (CBI) evaluated over 1500 thin-sections from bedrock samples, classifying them according to modified Rilem AAR 1, which is a petrographic evaluation method [described in 3]. The method classifies the aggregates into three classes: Class 1 – aggregates unlikely to be alkali-reactive, Class 2 – potentially alkali-reactive aggregates or alkali-reactivity uncertain, and Class 3 – aggregates very likely to be alkali-reactive. Alkalisilica-reactive aggregates in Sweden include deformed rocks such as cataclasite, mylonite and gneissic granitoids; fine-grained metavolcanic rocks; as well as sedimentary rocks such as metagreywacke, quartzite, sandstone etc. In this study, to establish whether varieties of these different rock types are alkali-reactive, a number of Class 1, Class 2 and Class 3 rocks were analyzed with the RILEM AAR 1 and RILEM AAR 2 methods.

Some results from this study were previously presented in a correlation study between different test-methods for alkalisilica-reactivity of Swedish aggregates used for concrete [3]. The different test methods include RILEM AAR 1 by point-counting within thin-sections; the modified RILEM AAR 1 petrographic method; and RILEM AAR 2. This paper presents results for all samples from the RILEM AAR 1 and 2 methods, but also gives a more comprehensive description of the different aggregate types and deliberates the differences between their reactivity. Results from the petrographic examination of mortar bars from the RILEM AAR 2 ultra-accelerated mortar bar test will be presented in a separate paper [4]. All results are reported in [5].

2. SAMPLES

23 rock samples from Southern Sweden are studied, including two very fine-grained metavolcanic rocks, three fine-grained metavolcanic rocks, one ambiguous fine-grained felsic rock with megacrysts, two sandstones and 15 granitoids of varying structure. Granitoids vary from massive, foliated, gneissic to mylonitic in order to describe the great variety of rocks represented in Sweden. The samples are described in Table 1. The samples have previously been classified into three groups based on an evaluation estimating the amount of potentially reactive components. Class 1 aggregates are unlikely to be alkali-reactive, with potentially reactive constituents estimated to be well below 15vol%. Class 2 aggregates were estimated to have

potentially reactive constituents in the range of 10-20vol% and are regarded as potentially alkali-reactive aggregates or aggregates with uncertain reactivity. Class 3 aggregates are those with potentially reactive constituents estimated to be well above 15vol% and are hence considered very likely to be alkali-reactive in concrete.

3. METHODS

Approximately 25-30 kg of each sample was crushed, sieved and reduced by splitting with a rotating divider according to SS-EN 932-2 [6], RILEM AAR 1 [7] and RILEM AAR 2 [8].

3.1 RILEM AAR 1

A representative test portion of the 2-4 mm fraction of each sample was casted with epoxy resin. From these specimens thin-sections with the dimensions $30x48mm^2$ were prepared. The RILEM AAR 1 petrographic method by point-counting in thin-sections [7] was used and the amount of potentially alkalisilica-reactive material refers to the quantity of crypto-, microcrystalline or highly strained quartz. Cross-points falling onto these mineral-grains are recorded as potentially reactive constituents. In this study 500 points in one thin-section were counted. The statistical error is in this study based on the number of grains present in each thin-section, as opposed to number of points counted [9].

3.2 RILEM AAR 2

Mortar bars with the dimensions 40x40x160mm were made with a 1.0 Na_2O-eq. cement, according to the RILEM AAR 2 accelerated mortar bar test [7]. The mortar bars were stored in a 1.0M NaOH-solution at 80^0C and their expansion was measured over a 28 days period.

After storage in NaOH, the mortar bars were further examined by cutting them in half and thin-sections were made for petrographic examination. Thin-sections were impregnated with fluorescent epoxy, which penetrates into cracks and pores and is enlightened in fluorescence light in order to enhance these structures. The samples were examined with regards to cracking of aggregates, cracking of cement paste and ASR products. Petrographic examinations include investigation in transmitted-, polarized- and fluorescence-light.

4. RESULTS

4.1 RILEM AAR 1

With the RILEM AAR 1 petrographic method by point-counting, ten aggregate samples were classified as non-reactive or very unlikely to be alkali-reactive (potentially reactive components <15vol%). These include ‘undeformed’ to gneissic granitoids and one sandstone with quartz-grains >120µm. Two samples are classified as potentially alkali-reactive if the uncertainty (±5%) of the measurement is considered (one gneissic to mylonitic granitoid and one sandstone with major quartz-grains >120µm). The remaining 13 samples have potentially reactive constituent contents above 15vol% and are considered potentially or very likely to be alkali-reactive. These include metavolcanic rocks; one heterogeneous, partly deformed granitoid; gneissic to mylonitic granitoids; and a heterogeneous sandstone with major quartz-grains <120µm.

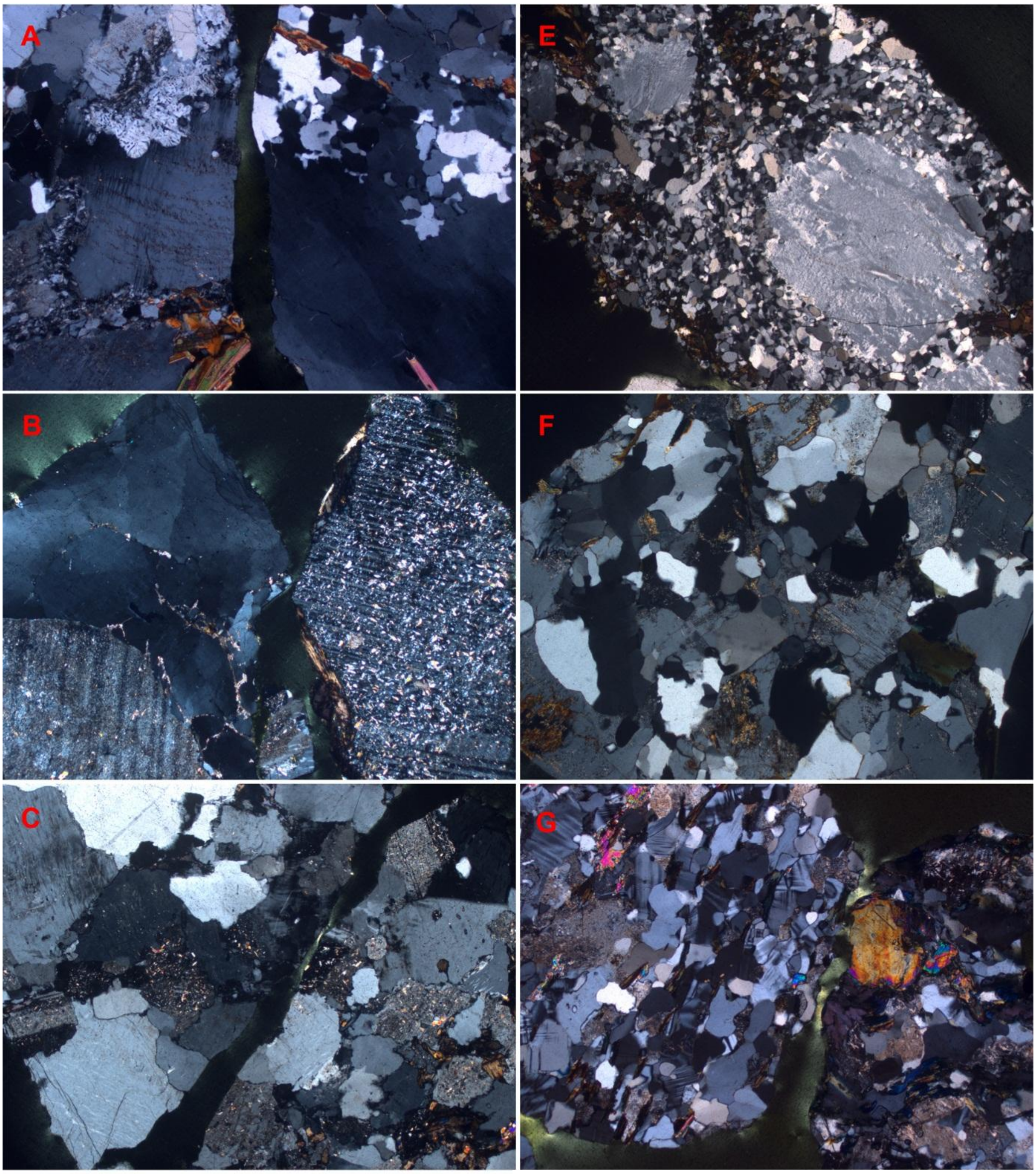
Table 1 – Sample description and results

Sample	Sample description	Class[1]	ASR, vol%[2]	Expansion, %[3]
10	Granitoid	1	1	0.06
11	Granitoid	1	2	0.10
17	Granitoid, unequigranular	1	4	0.15
20	Sandstone (quartz mainly >120µm)	1	12	0.25
22	Banded, foliated granitoid/gabbroid	2	4	0.10
23	Banded, foliated granitoid/gabbroid	2	6	0.11
8	Heterogeneous granitoid, partly deformed (gneissic)	2	8	0.12
5	Heterogeneous granitoid, partly deformed (gneissic)	2	10	0.20
6	Heterogeneous granitoid, partly deformed (gneissic)	2	10	0.24
14	Heterogeneous gneissic (to mylonitic) granitoid	2	13	0.23
2	Fine-grained metavolcanic rock, with megacrysts	2	17	0.19
15	Heterogeneous (gneissic to) mylonitic granitoid	2	19	0.28
18	Fine-grained felsic rock, with megacrysts	2	22	0.14
9	Heterogeneous granitoid, partly deformed (gneissic)	2	23	0.25
7	Heterogeneous mylonitic granitoid	2	29	0.30
21	Heterogeneous, fine-grained, mica-rich metavolcanic rock	2	39	0.32
19	Heterogeneous sandstone (quartz mainly <120µm)	2	41	0.43
12	Gneissic (to mylonitic) granitoid	3	15	0.23
13	Fine-grained mylonite	3	43	0.35
16	Fine-grained mylonite	3	50	0.34
4	Very fine-grained metavolcanic rock (porphyritic)	3	61	0.55
1	Fine-grained metavolcanic rock, with megacrysts	3	65	0.27
3	Very fine-grained metavolcanic rock (porphyritic)	3	69	0.44

[1] *Results from modified RILEM AAR 1, petrographical method. 1: unlikely to be alkali-reactive, 2: potentially reactive/uncertain, 3: very likely to be alkali-reactive*

[2] *Results from RILEM AAR 1, ASR: potentially alkalisilica-reactive components*

[3] *Results from RILEM AAR 2, prisms 40x40x160mm, measurement after 28 days in 1.0M NaOH*



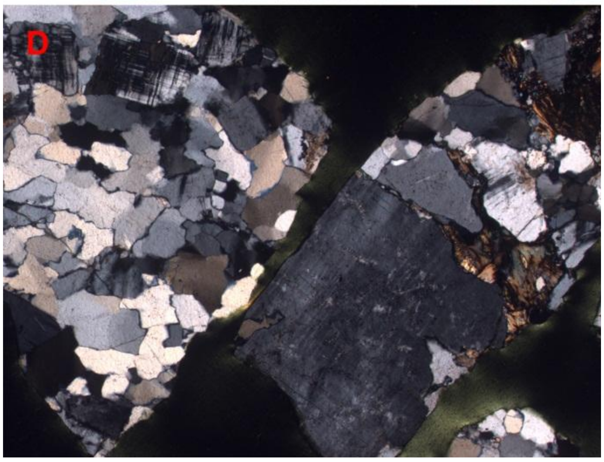


Figure 1 - Photomicrographs in cross-polarized light of crushed Swedish rock types considered non-reactive, if both Rilem AAR 1 and 2 are considered. Field of view is 2.8x2.1mm^2. ***A.*** *Sample 8: Heterogeneous granitoid, partly deformed (gneissic).* ***B.*** *Sample 10: Coarse-grained, massive granite.* ***C.*** *Sample 11: Granitoid.* ***D.*** *Sample 17: Unequigranular granitoid.* ***E.*** *Sample 18: Fine-grained felsic rock with megacrysts.* ***F.*** *Sample 22: Banded, foliated grantoid/gabbroid.* ***G.*** *Sample 23: Banded, foliated grantoid/gabbroid.*

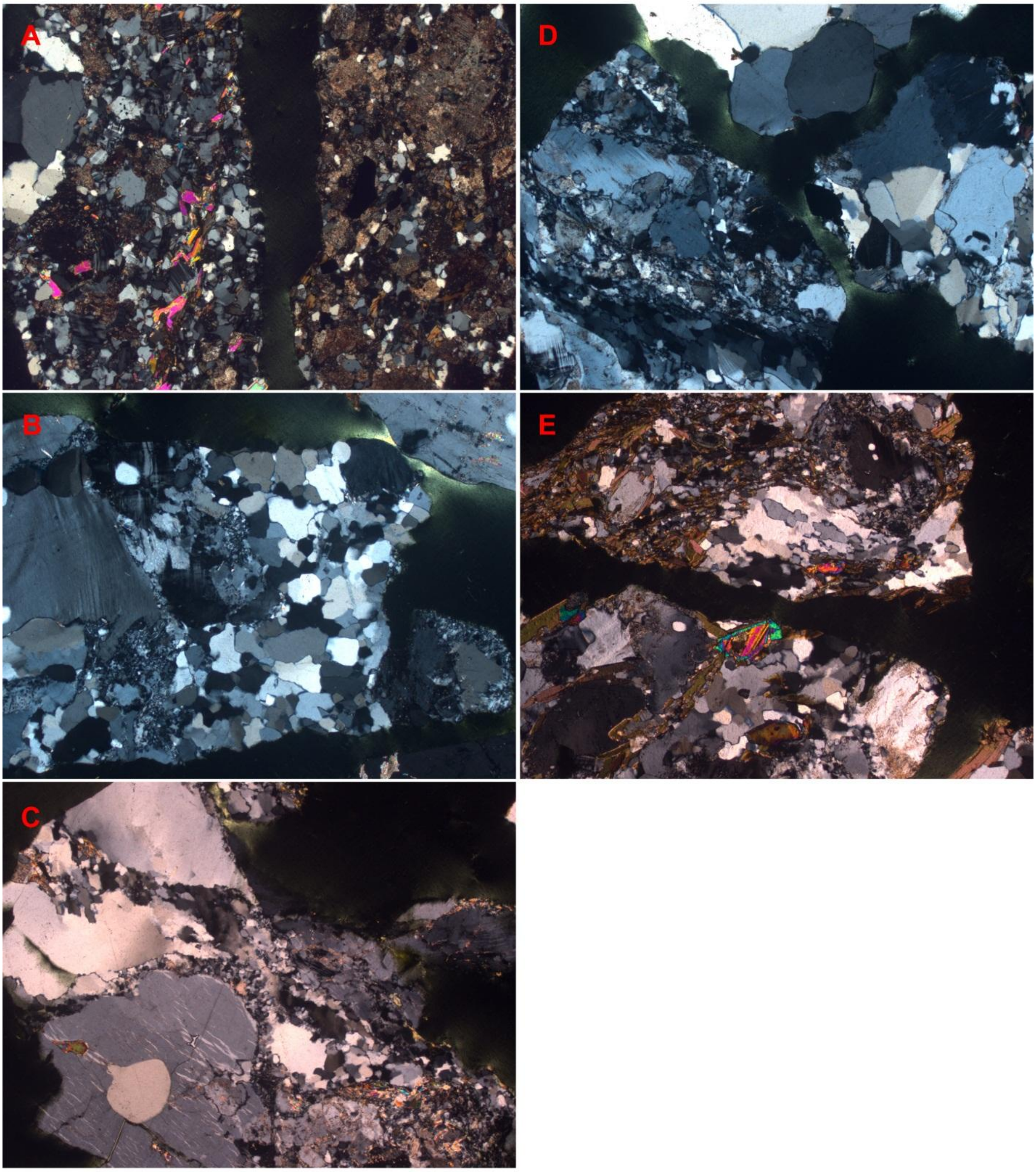
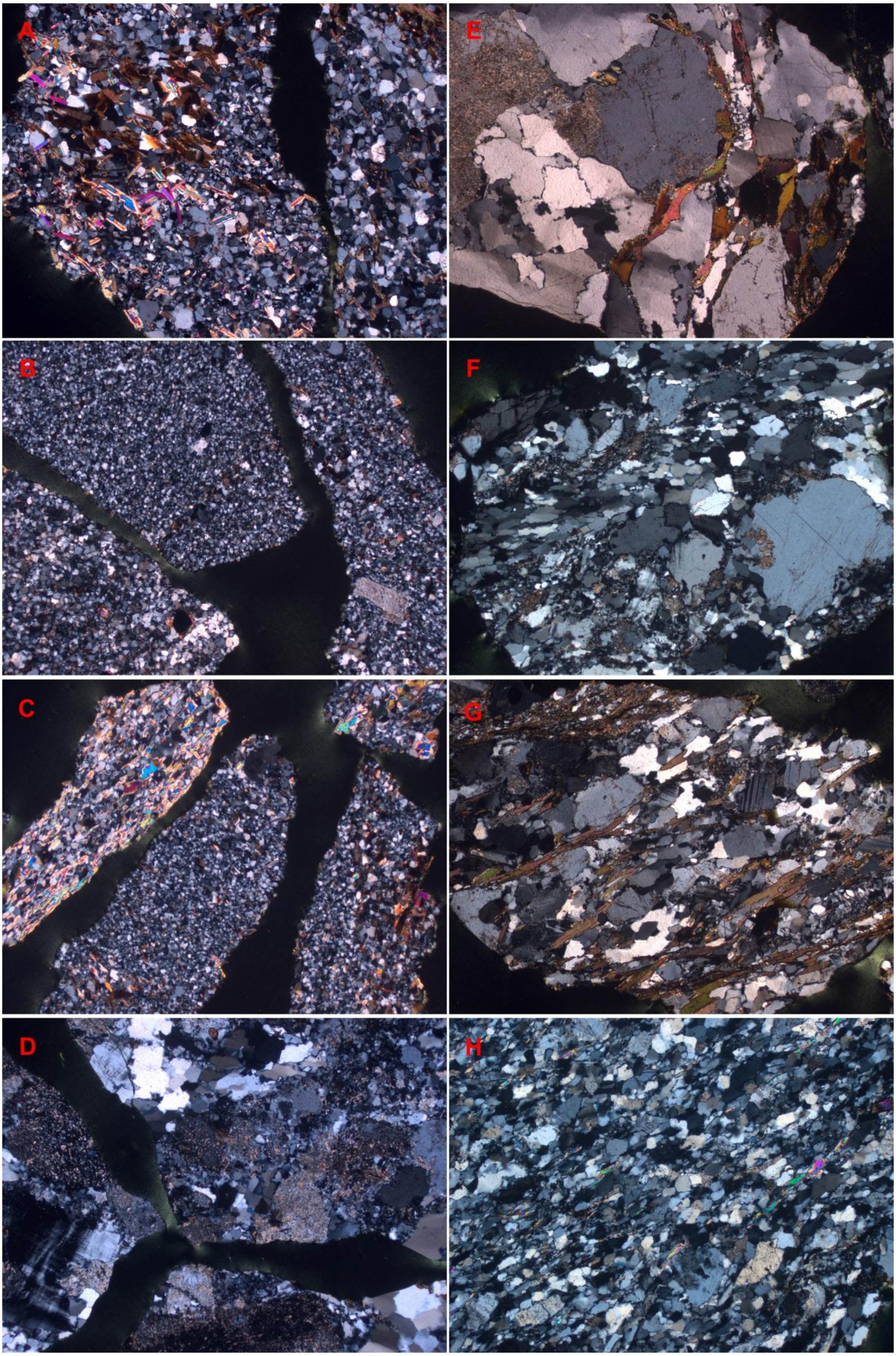


Figure 2 - Photomicrographs in cross-polarized light of crushed Swedish rock types considered potentially alkali-reactive, if both Rilem AAR 1 and 2 are considered. Field of view is 2.8x2.1mm^2. ***A.*** *Sample 2: Fine-grained metavolcanic rock with megacrysts.* ***B.*** *Sample 5: Heterogeneous, massive, partly cataclastic granitoid (strongly deformed).* ***C.*** *Sample 6: Heterogeneous granitoid, partly deformed (gneissic).* ***D.*** *Sample 12: Gneissic (to mylonitic) granitoid.* ***E.*** *Sample 14: Gneissic (to mylonitic) granitoid.*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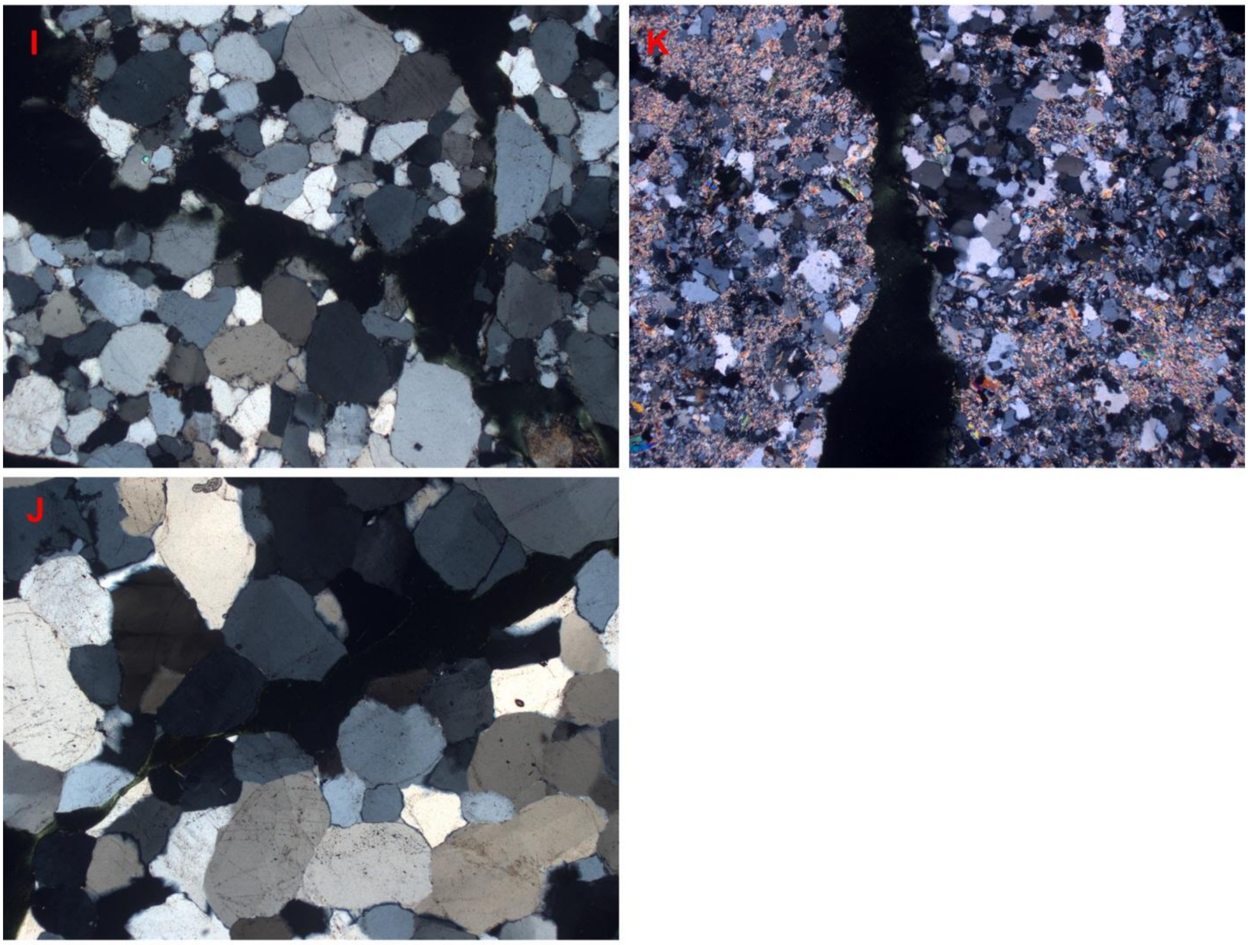

*Figure 3 - Photomicrographs in cross-polarized light of crushed Swedish rock types considered very likely to be alkali-reactive, if both Rilem AAR 1 and 2 are considered. Field of view is 2.8x2.1*mm^2*.* ***A.*** *Sample 1: Fine-grained metavolcanic rock (porphyritic) with grain-size of matrix ranging between 0.05-0.1mm.* ***B.*** *Sample 3: Very fine-grained metavolcanic rock.* ***C.*** *Sample 4: Very fine-grained metavolcanic rock (porphyritic) with micro- to cryptocrystalline <0.063mm matrix.* ***D.*** *Sample 7: Heterogeneous mylonitic granitoid.* ***E.*** *Sample 9: Heterogeneous granitoid, partly deformed (gneissic).* ***F.*** *Sample 13: Heterogeneous fine-grained mylonite.* ***G.*** *Sample 15: Heterogeneous (gneissic to) mylonitic granitoid.* ***H.*** *Sample 16: Fine-grained mylonite.* ***I.*** *Sample 19: Heterogeneous sandstone with quartz-grains <120µm.* ***J.*** *Sample 20: Sandstone with quartz-grains >120µm.* ***K.*** *Sample 21: Heterogeneous, fine-grained mica-rich metavolcanic rock.*

4.2 RILEM AAR 2

Non-reactive aggregates in the RILEM AAR 2 mortar bar test, with expansion ≤0.15%, are mainly ‘undeformed’ or weakly foliated granitoids. One fine-grained meta-/subvolcanic rock with megacrysts also fall into this category. Potentially reactive aggregates with an expansion between 0.19-0.24% include one metavolcanic rock; and deformed granitoids with moderate degrees of deformed or microcrystalline quartz. Aggregates which are very likely to be alkali-reactive in concrete, with expansion ≥0.25%, include four very fine- to fine-grained metavolcanic rocks, gneissic to mylonitic granitoids and sandstones.

5. DISCUSSION

5.1 Correlation between RILEM AAR 1 and RILEM AAR 2

The correlation between RILEM AAR 1 and 2 has previously been accentuated [10] and is well established in this study. Only two samples deviate from this trend. One fine-grained metavolcanic rock (sample 1) has lower expansion than the most expansive metavolcanic aggregates with very fine-grained cryptocrystalline matrix. This is probably because the more fine-grained silica-phases react faster in the alkalisilica-reaction [11-14]. Sample 18 is somewhat ambiguous and also deviates from the general trend. In the field the Geological Survey of Sweden classified it as a granitoid, but microscopically it was, by CBI, first classified as a fine-grained meta/subvolcanic rock with megacrysts. This merely states the relevance of the combined field observations with microscopical studies; and as suggested from the Rilem AAR 2 results it is more likely to be a fine-grained, possibly recrystallized (non-reactive) granitoid.

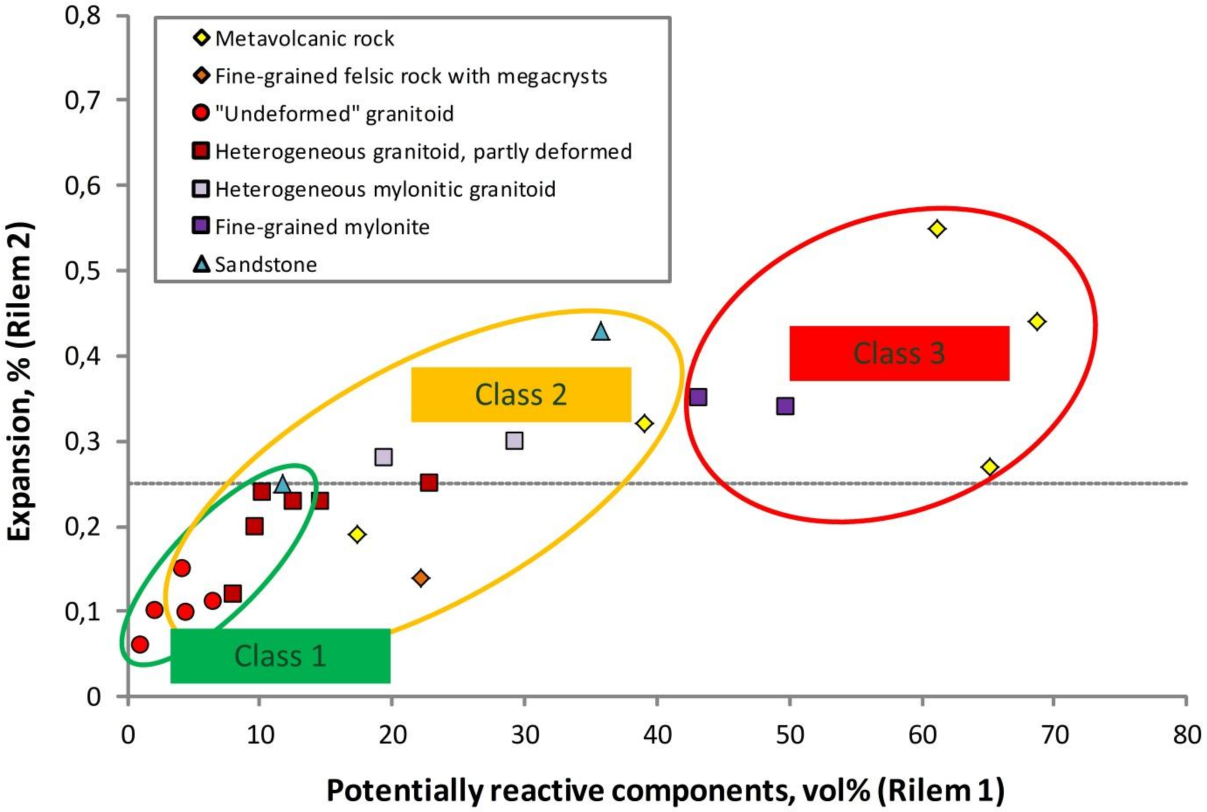

Figure 4 - The amount of potentially alkalisilica-reactive components in Swedish aggregates plotted against their mortar-bar expansion (in percent), tested with RILEM AAR 2. First classification regarding its potential alkalisilica-reactivity in concrete, based on an estimation of the content of potentially reactive components. Class 1: unlikely to be alkali-reactive, Class 2: potentially alkali-reactice/alkali-reactivity uncertain, Class 3: very likely to be alkali-reactive.

5.2 Reactivity of aggregates

General

Most samples regarded as non-reactive or very likely to be reactive according to the first classification, follow the expected trends in the RILEM AAR 1 and 2 methods. However, one sandstone, with quartz-grains mainly larger than 120µm, which was considered unlikely to be alkali-reactive in the first classification (Class 1), but potentially reactive in the RILEM AAR 1 point-counting method (13±5vol%), shows an expansion of 0.25% in the RILEM AAR 2 method.

Samples regarded uncertain/potentially alkali-reactive in the first classification show varying results with the RILEM AAR 1 and 2 methods. Some granitoids were classified as Class 2 rocks due to its heterogeneity, with gneissic features. Also foliated granitoids were classified in this category. However, some of these can easily be redefined as non-reactive after point-counting. Class 2 rock types with ASR-contents between 10-20vol% are more ambiguous as the expansion in RILEM AAR 2 rarely reaches the critical limit of 0.25%. Aggregates with an expansion between 0.19-0.24% are therefore regarded as potentially reactive aggregates, mainly because of their heterogeneous characters. Hence, the importance of the point-counting technique on crushed aggregates is stressed by this study.

Examination of thin-sections from mortar bars

Regardless of the degree of alkalisilica-reactivity in the RILEM AAR 1 and 2 methods, all mortar-bars contain ASR-products [4-5]. Petrographic examination of the mortar bars from the RILEM AAR 2 tests, of aggregates with expansion ≥ 0.25%, show a high degree of microcracks within the aggregates but only low to moderate degrees of cracks in the cement paste. Grain-boundary cracks around mineral grains in the aggregates are abundant and ASR-gel in air voids and microcracks are frequent. Within mortar bars composed of potentially alkali-reactive aggregates, which show an expansion between 0.19-0.24%, the abundance of microcracks and ASR-gel is greatest in the more fine-grained or deformed, gneissic or mylonitic aggregates. These parts show the most severe cracking, whereas in general they show moderate cracking in aggregates and low degrees of cracking in the cement paste. ASR-gel in air voids exist throughout the samples. Also “non-expanding” mortar-bars (with expansion ≤15%), consisting of aggregates considered unlikely to be alkali-reactive, contain air voids filled with ASR gel, although sparsely distributed, which was somewhat unexpected. Even if the aggregates contain low contents of potentially alkali-reactive constituents, the content was not believed to be high enough to actually develop ASR-gel. Hence, the extreme conditions probably cause a reaction from any soluble silica present within the aggregates. This merely concludes that the RILEM AAR 2 method initiates all available reactive phases within the aggregates [4].

Non-reactive aggregates

If both RILEM AAR 1 and 2 are considered, medium- to coarse-grained massive to foliated granitoids are considered non-reactive (sample 8, 10, 11, 17, 22, 23) along with the ambiguous felsic rock (sample 18) that was finally classified as a fine-grained granitoid (see section 5.1).

Potentially alkali-reactive aggregates

Potentially reactive aggegates, with an expansion of the mortar bars between 0.19-0.24%, include a fine-grained metavolcanic rock (sample 2); and heterogeneous deformed granitoids with moderate degrees of deformed or microcrystalline quartz (sample 5, 6, 12, 14). In these aggregates, the more fine-grained and deformed, gneissic or mylonitic parts show the most extensive cracking in the mortar bars and it is recommended that heterogeneous aggregates should be treated carefully in engineering structures as deformation zones locally may cause higher levels of alkali-reactive constituents, causing severe damages to the structures [4].

Alkalisilica-reactive aggregates
The alkali-reactivity of aggregates is affected by silica grain size, crystal structure, internal porosity and permeability [15-18]. Further, deformed quartz has an increased alkali-solubility in comparison to perfect quartz-crystals, because a distorted crystal with a deviating structure more easily accommodates foreign ions than a crystal with undistorted structure [17]. Alkalisilica-reactive aggregates in this study include metavolcanic rocks (sample 1, 3, 4, 21); high-grade gneisses (7, 9, 15); mylonites (sample 13, 16); and sandstones (sample 19-20). Sample 7 and 15 are very heterogeneous with both gneissic and mylonitic features. The more fine-grained and deformed, gneissic or mylonitic parts show the most extensive cracking in the mortar bars [4-5]. For the mylonites it is probably a combination of deformed quartz and subgrain formation that causes its alkali-reactivity.

The five metavolcanic rocks are classified as alkalisilica-reactive or potentially alkali-reactive aggregates (sample 1, 2, 3, 4 and 21). As discussed in section 5.1, the most fine-grained varieties (sample 3 and 4) show the greatest expansion. Sample 21 was first classified as a Class 2 type aggregate because of its high mica-content, but the results from the Rilem AAR 2 test suggest that the mica-rich matrix is accompanied by significant amounts of cryptocrystalline quartz. The somewhat lower expansion in sample 2 (compared to sample 1, 3, 4 and 21) is probably related to higher feldspar-contents in this aggregate, which e.g. could be examined further by XRD-analysis. Sample 2 and sample 18 are microscopically similar, although sample 18 yielded a somewhat lower expansion in the Rilem AAR 2 test. Sample 18, which is somewhat ambiguous was microscopically first classified as a meta-/subvolcanic rock, but is instead classified as a fine-grained granitoid (see discussion above). Thus, felsic metavolcanic rocks in Sweden should always be handled carefully for use as aggregates in outdoor concrete structures.

Final remarks
It is important to remember that RILEM AAR 1 is a screening test, whereas RILEM AAR 2 is an accelerated test method in which mortar-bars are exposed to excessive alkalis under elevated temperatures. To assess the true nature of the alkali-reactivity of aggregates, their potential alkali-reactivity within concrete should be investigated further with e.g. the RILEM AAR 3 or 4 concrete prism methods [19-21] and by microscopic investigations including crack-pattern studies of the concrete prisms to determine whether a supposed expansion is actually caused by alkali-reactions.

6. CONCLUSIONS

It is concluded that the RILEM AAR 1 and RILEM AAR 2 methods correlate well with each other. With constituents considered potentially alkalisilica-reactive ranging between 10-20 vol% however, the RILEM AAR 1 and 2 methods need to be evaluated more carefully.

From the investigated rock types only ‘undeformed’ massive to weakly foliated granitoids can be considered as non-reactive aggregates in concrete. Potentially reactive aggregates include heterogeneous deformed granitoids with moderate degrees of deformed or microcrystalline quartz whereas metavolcanic rocks, high-grade gneisses, mylonites and sandstones (regardless of grain size) are considered very likely to form alkalisilica-reactive aggregates in concrete.

Further, an evaluation of thin-sections is not sufficient; the more detailed microscopic point-counting technique is required to get an adequate comprehension of the potential alkali-reactivity of an aggregate.

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Analysis of damages area in concrete caused by AAR

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ABSTRACT

Examination of the defects caused by AAR in the concrete floors surface at the construction site and taking of specimens, SEM and optical microscopy analysis and assessment of the defect area, EDS element analysis and assessment of aggregates and reactive products, determination of density of the concrete and compressive strength and conformity assessment of the concrete strength have been analysed in the paper. Aggregates for concrete are produced from gravel and contain reactive rock, the hazard of which depends on size, quantity and reactivity of the particles.

Key words: Alkali-aggregate reaction, concrete aggregate, reactive rocks, carbonates, flint, opoka.

1. INTRODUCTION

Lithuania is a country in Europe situated at the south-eastern coast of the Baltic Sea. Climate in Lithuania is moderately warm, sea – continental.

The mean temperature in the middle of July is around 17 °C and around -5 °C in winter; the temperature interval is around 20 °C. There have been very hot summers in Lithuania when the temperature reached +35 °C and very cold winters, when the temperature dropped to - 25° C in daytime and up to -30 °C at night. In summer mean temperature changes are bigger than in winter. In winter average temperature is 2 – 3°C and in summer - 7–10°C. There are favourable conditions for occurrence of alkali silica reactions in Lithuania - high humidity and temperature above + 20° C.

Sodium and potassium chlorides are used like de-icing salts for roads in Lithuania.

Alkali-Aggregate Reactions (AAR) cause severe damage in concrete structures worldwide [1-8]. Alkali-aggregate reaction (AAR) is an internal chemical reaction between the alkaline components in the cement and certain active mineral constituents in some aggregates[1]. The more fine-grained silica-phases react faster in the alkali silica-reaction [2]. The most widespread type of AAR is Alkali-Silica Reactions (ASR) in which silica sensu lato in the aggregate react with available alkali from the cement paste forming a hygroscopic gel. [3]

Sand and gravel and crushed gravel are usually used for production of concrete in Lithuania like aggregates. In all gravel deposits there are massive pieces of opoka with inclusions of silicate rocks and massive pieces of porous flint among different types of carbonate and silicate rocks [6, 9]. Thus, products of all Lithuanian gravel pits - coarse and fine aggregates are all fouled with opoka and flint, which intensely react with sodium and potassium alkalis, contained in the cement. Opoka is two times more reactive than flint.

The process of AAR requires a sufficient quantity of alkalis and humidity environment. Exposure classes are defined in the standard LST 1974:2012 XR0, XR1, XR2 [10] (table 1). Aggressive environment, in which the relative humidity is φ ≥ 80 %, and the concrete contains free moisture.

Table 1.Exposure classes by AAR (LST 1974:2012)

XR0	Non-aggressive environment where the indoor relative humidity is φ ≤ 60 %	Moisture-proof interior structures of the building
XR1	Aggressive environment in which the relative humidity is φ ≥ 80 % when the concrete contents free moisture	Alkaline aggregate corrosion takes place
XR2	Very aggressive environment, where the concrete contents free of moisture and additionally is affected by Na and K compounds, for example, with salts because of icing	Alkaline aggregate corrosion is intense

XRO aggressive alkaline corrosive environment in terms of use of concrete mix design is not restricted. Exposure class XR1 in which the relative humidity is $\varphi \geq 80$ %, and the concrete contains free moisture. In the exposure class XR1 from gravel quarries of Lithuania, equivalent Na and K oxides, $(Na_2O)_{eqv.} = Na_2O + 0.658K_2O$ content in CEM I cement should be ≤ 0.8% (by mass) and the equivalent Na and K oxides cement in the concrete mix should not exceed 4.0 kg/m^3. Exposure class XR2 in which the relative humidity is $\varphi \geq 80$ %, and the concrete contains free moisture. In the exposure class XR2 from gravel quarries of Lithuania, equivalent Na and K oxides, $((Na_2O)_{eqv.} = Na_2O + 0.658K_2O$ content in CEM I cement should be ≤ 0.8% (by mass) and the equivalent Na and K oxides cement in the concrete mix should not exceed 3.0 kg/m^3.

The composition of the gravel or crushed gravel from gravel quarries in Lithuania is polymineral, reactive rocks are in separate participles. When this aggregate is used, degradation of concrete volume does not occur, but local defects of the structure surface occur in places where a reactive aggregate is close to the surface of the structure. AAR of concrete with coarse aggregates from gravel quarries of Lithuania does not cause decrease of the concrete compressive strength but only result in formation of pop-outs on the concrete surface.

In Lithuania the AAR in concrete is different from than in other countries, where aggregates are compound from magmatic rocks. Gravel in Lithuania are formed from metamorphic rocks of split origin, split metamorphic rocks and chemical and biogenic sedimentary rocks. Aggregates for concrete are produced from gravel and contain reactive rock, the hazard of which depends on size, quantity and reactivity of the particles.

2. METHODS FOR INVESTIGATIONS

The tests have been conducted in order to determine causes of defects in the concrete floors (figure 1).

Figure 1. The view of the defects on concrete floor surface

The concrete specimens have been cut from the surface of concrete floor by a diamond saw from the top surface. Top and bottom surfaces of the cut specimens have been polished. Having examined the cut surface of the concrete specimens’ structure of the concrete can be observed. The structure of concrete in all specimens is a coarse aggregate. Maximum size of the coarse aggregate is 16 mm. The coarse aggregate is crushed gravel or gravel. The coarse aggregate is evenly distributed in the bulk of concrete. There are no large pores and cavities in concrete, i.e., the concrete has been properly compressed during the concrete placing.

Specimens of concrete with defects (figure 2) in the concrete surface have been used for microscopy tests conducted by optical microscopy and specimens for tests have been taken by a raster electron microscope (SEM) and for evaluation of chemical composition of separate places of specimens in the method of energy dispersive X-ray spectrometry (EDS). Microscopy tests of defects in the concrete surface have been conducted by MOTIC optical microscope (enhances images from 6 to 50 times). The enhanced top layer of the surface of specimens was viewed.

Raster electron microscopy (SEM tests) has been conducted using the microscope JEOL 7600F, resolution of which is 1 nm (enhances images from 25 to 1 million times). Specimens of concrete have been taken from three different places - the aggregate, available in the bottom part of the defect, area of reactive products around the reactive coarse aggregate and further area, i.e., matrix of the hardened cement paste. Before SEM tests specimens have been covered with a layer of gold by using special equipment.

Energy dispersive X-ray spectrometry (EDS tests) has been conducted using the equipment available at the abovementioned raster electron microscope. The same specimens have been used as for the raster electron microscopy tests. Four points have been tested. Chemical elements have been determined, which were found in the tested points and their mean volume in the material has been calculated.

Specimens with the surface containing no defects have been used for determination of the density of the concrete floors and compressive strength. Compressive strength of the concrete specimens has been determined in accordance with LST EN 12390-3: 2009[11], density – in accordance with LST EN 12390-7: 2009 [12].

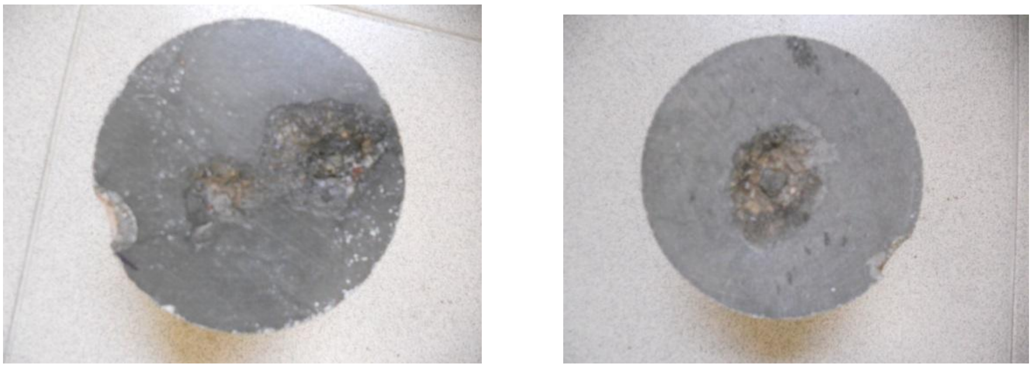

Figure 2. The specimens of concrete with surface defects

Reactivity of the aggregates from Smilgiai (sand fr. 0/4, gravel fr. 8/16 and crushed gravel fr. 8/16) and Beržutė (crushed gravel fr. 8/16) pits were tested.

Reactivity of the aggregate has been determined in accordance with regulations of standard LST 1974-2012 *LST EN 206-1 Application rules and additional national requirements, Annex M.*

First, carbonaceous rocks are removed. The specimens are filled with 10 % hydrochloric acid (HCL) solution, 5 times higher than its bulk.

Carbonaceous rocks - opoka and flint are melted. Specimens are filled 10% NaOH solution 90^{o}C temperature, the proportion of bulk and specimen should be 1:2. In case of reactive rocks (opoka) it is melted for 12 hours. In case of the flint test specimens are additionally kept for 60 hours. Specimens are put through sieves of a proper size.

3. RESULTS AND DISCUSION

During examination it has been determined that local surface defects appear in floor – scales (holes) in the floor surface, the depth of which is several millimetres and reaches the coarse aggregate.

The composition of the gravel or crushed gravel from gravel quarries in Lithuania is polymineral reactive rocks are in separate participles, when this aggregate is used, degradation of concrete in volume does not occur, but local defects of the structure surface occur in places where a reactive aggregate is close to the surface of the structure. AAR of concrete with coarse aggregates from gravel quarries of Lithuania does not cause decrease of the concrete compressive strength but only result in formation of pop-outs on the concrete surface.

Results of the concrete compressive strength and the conformity assessment suggest that compressive strength of the concrete floors complies with the requirements set for class C25/30 of the concrete strength (table 2). Alkaline corrosion of concrete with coarse aggregates from gravel pits of Lithuania does not cause decrease of the concrete compressive strength but results in formation of defects in the concrete surface.

2 table. Density and compressive strength results of concrete samples

Nr.	Area mm^2	Weight, kg	Destructive force, kN	Density, kg/m^3	Compressive strength, MPa
1	8704.2	1.691	339.9	2291	30.5
2	8720.7	1.612	305.1	2216	27.2
3	8704.2	1.814	260.6	2182	24.3
4	8687.6	1.817	266.6	2319	24.5
5	8704.2	1.905	283.2	2229	26.6
6	8720.7	1.981	368.7	2223	34.9
Average				2243	28.0

SEM and EDS tests results demonstrate that pop-outs in concrete surface is caused by the reactive coarse aggregate. After reaction of this all aggregate with sodium and potassium alkalis reactive in the cement sodium and potassium silicates form, which have a glassy form in high porosity that arouse deformations around the reactive aggregate, which causes pop-outs of the concrete.

Microscopy pictures of defects in the concrete floors suggest that scales of the concrete surface appear because of the coarse aggregate, which reacts with the hardened cement paste or materials contained in it. The scaled surface of the coarse aggregate can be observed in the picture (figure 4). The area of reactive products forms around the coarse aggregate (figure 5). High porosity and big amount of glassy phase (figure 6) is typical to this area. Matrix of a hardened cement paste is behind the reactive product area, where reactive products are also seen. Microscopy tests conducted with an optical microscope allowed to determine that the cause of occurrence of defects in the concrete floor is chemical reaction of the coarse aggregate with the compounds found in the hardened cement paste, during which big amount of glassy phase forms, creating the area of reactive products around the reactive aggregate. The area of reactive products is characterized by high porosity, which has formed in the course of deformations as a result of chemical reactions and big number of the reactive products. Increase of the number of reactive products around the reactive aggregate results in high internal pressure, which tears off the layer of hardened cement paste up to the surface of the concrete.

Findings of the microscopy tests suggest that when the reactive aggregate undergoes chemical reaction with alkalis found in the cement, three areas forms around the reactive aggregate: 1) reactive aggregate; 2) area of reactive products; 3) matrix of the hardened cement paste.

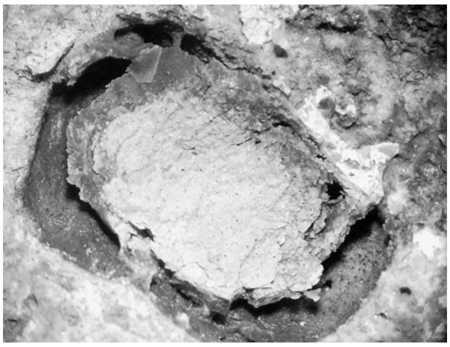

Figure 4. Microscopic pictures of damages areas

Figure 5. Microscopic picture of area of reactive products

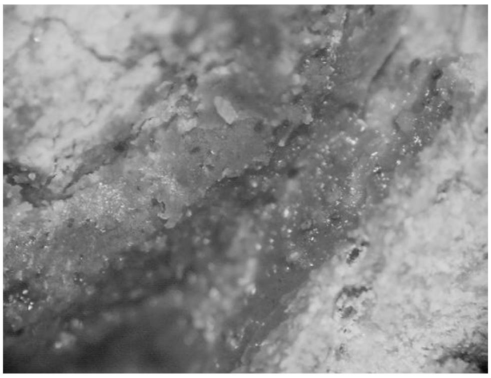

Figure 6.Microscopic picture of matrix of hardened cement paste

SEM test of the concrete floor defect aggregate, using EDS, demonstrate that the reactive aggregate consists of silica (SiO_2).

Analysis of elements of the concrete reactive aggregate suggests that this aggregate consists of silica (SiO_2) (figure 7, table 3). Reactive aggregate of specimen contained also ions of Ca, which form composition of the aggregate. The reactive aggregate also contains small amounts of Na, K, Mg and ions of Al. The mean volume of the amount of chemical elements in the reactive aggregate indicates that it consists of silica with admixtures of calcium, sodium and potassium compounds.

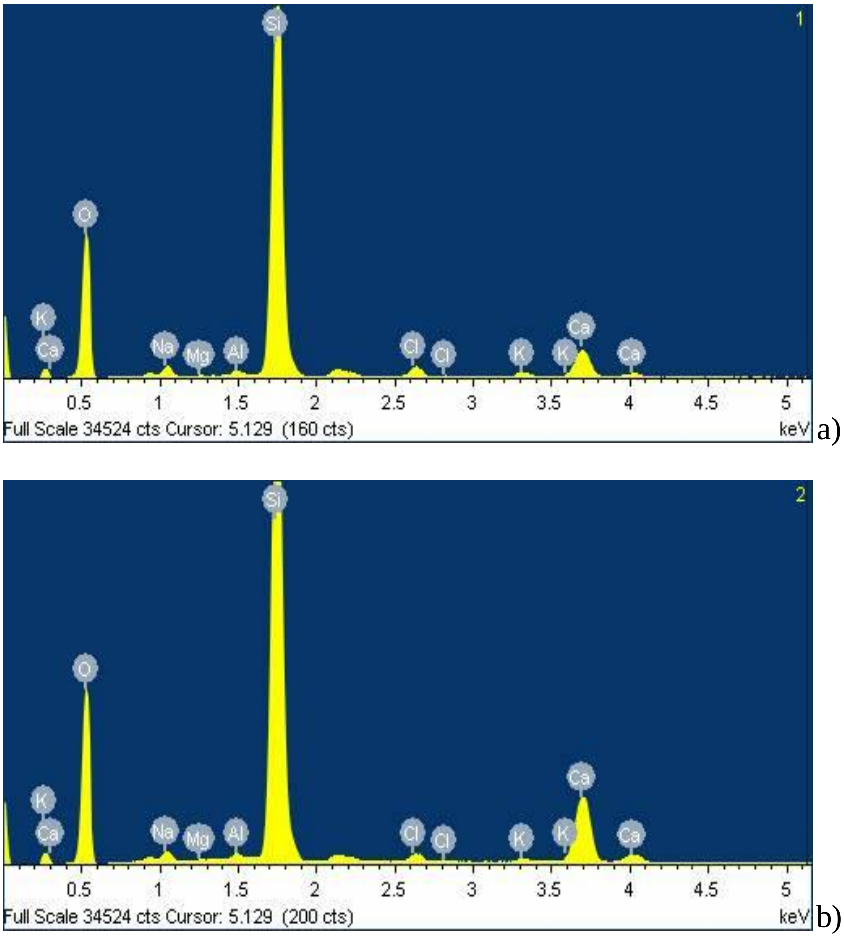

Figure 7. Concrete reactive aggregates zones EDS spectrums

Table 3. Element composition of concrete reactive aggregate area

Nr.	Na, %	Mg,%	Al,%	Si, %	Cl, %	K, %	Ca, %	O, %	Total
1.	1.70	0.06	0.34	40.32	1.84	0.81	5.63	49.29	100.0
2.	1.05	0.03	0.26	39.26	0.98	0.36	9.06	49.02	100.0

SEM tests of the matrix of hardened cement paste of defects in the cement floors and analysis of elements (using EDS) demonstrate that the matrix of the hardened cement paste consists of hydrated silicate of calcium (C-S-H). The mean volume of the amount of chemical elements in the reactive aggregate suggests that it consists of hydrated silicate of calcium and hydroaluminates with compounds of sodium, magnesium and potassium (figure 8, table 4).

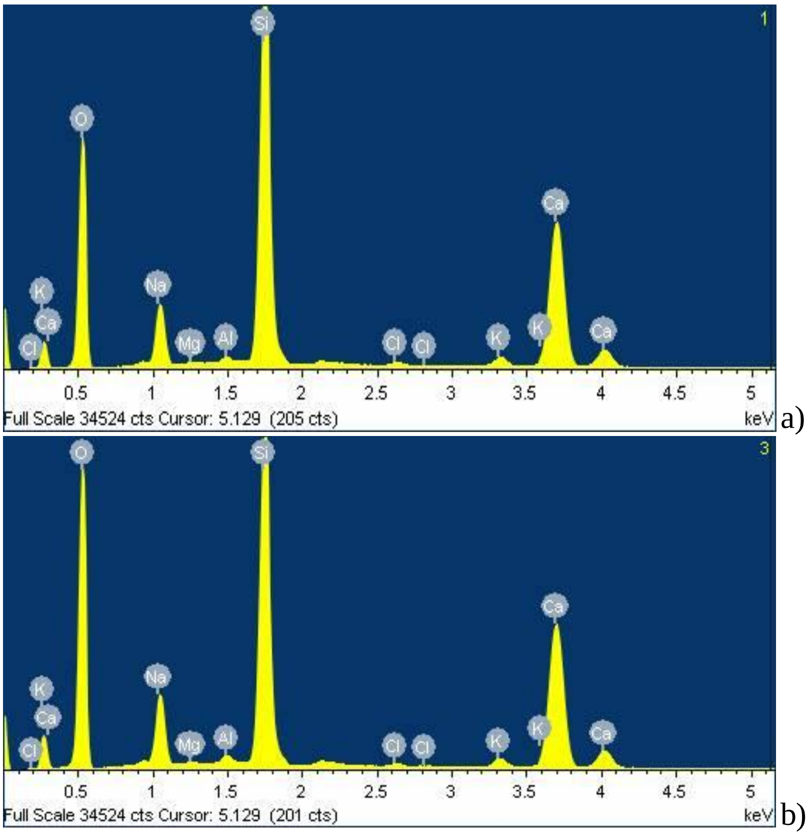

Figure 8. Concrete reactions products zones EDS spectrums

Table 4. Element composition of concrete reactions products zone

Nr.	Na, %	Mg,%	Al,%	Si, %	Cl, %	K, %	Ca, %	O, %	Total
1.	7.14	0.18	0.31	28.93	0.39	1.00	18.59	43.46	100.0
2.	8.76	0.20	0.55	27.09	0.37	1.02	19.48	42.52	100.0

Analysis of elements of the concrete reactive products indicated that they consist of sodium and calcium silicates (Na_2SiO_3 and K_2SiO_3). In the reactive product area of the specimen Ca ions have also been found, the same as in the reactive aggregate.

Findings of SEM and EDS tests demonstrate that scaling of the concrete surface is determined by the coarse aggregate, made of silica. After reaction of this aggregate with sodium and potassium alkalis found in cement, sodium and potassium silicates develop, which form a glasslike bulk of high porosity, resulting in deformation around the reactive aggregate and this results in scaling of the concrete surface.

Gravel pits of Lithuania are fouled with reactive particles, containing amorphous silica, which reacts with sodium and potassium alkalis found in cement and causing alkali corrosion of the concrete. Using aggregates from gravel pits of Lithuania for concrete, which is operated in the conditions favourable for alkali corrosion (high humidity and temperature above +20 ºC), some measures to prevent alkali corrosion should be taken.

Reactivity of aggregates has been determined under methods developed in Lithuania. Aggregates from Smilgiai (sand fr. 0/4, gravel fr. 4/16 and gravel crushed fr. 4/16) (table 5) and Beržutė (gravel crushed fr. 4/16) (table 6) quarries were tested.

Table 5. Reactivity of aggregates from Smilgiai

No	Fraction	Reactive rock (opoka) quantities in coarce and sand		Reactivity rock (flint) quantities in coarce and sand		Reactive rock persentages (opoka) for coarse and sand		Reactivity rock (flint) for coarse and sand	
		1	2	1	2	1	2	1	2
Crushed gravel									
1.	4/8	3.13	4.12	5.86	5.24				
2.	8/16	5.14	4	5.81	6.25				
3.	Two fractions					0.59	0.13	0.83	0.86
Gravel									
1.	4/8	1.8	1.9	3.25	2.85				
2.	8/16	2.05	3.11	4.25	5.09				
3.	Two fractions					0.27	0.36	0.54	0.57
Sand									
1.	2/4	2.97	3.07	3.75	3.37				
2.	1/ 2	2.97	3.03	2.32	2.65				
3.	0.5/1	2.23	2.3	0.45	0.75				
4.	0.25/0.5	1.71	1.8	2.05	1.95				
	All fractions					2.47	2.55	2.14	2.18

Table 6. Reactivity of aggregates from Beržutė

No	Fraction	Reactive rock (opoka) quantities in coarce and sand		Reactivity rock (flint) quantities in coarce and sand		Reactive rock persentages (opoka) for coarse and sand		Reactivity rock (flint) for coarse and sand	
		1	2	1	2	1	2	1	2
Crushed gravel									
1.	4/8	7.88	6.93	4.3	3.9				
2.	8/16	6.25	7.35	7.69	8.35				
3.	Two fractions					1.01	1.02	0.86	0.90

Total volume of reactive rocks (alkaline reactivity) in tested aggregates is as follows: in the sand - 4.66 % and 0.88–1.9 % in the gravel and crushed gravel. The determined volume of reactive rocks in gravel and crushed gravel is similar to the one in other gravel quarries in Lithuania (around 2 %) and the volume of reactive rocks in sand is rather high, i.e. around 2 times higher. Average amount of reactive rocks in the coarse aggregate from Lithuanian quarries (gravel and crushed gravel) about is 2 % and the amount of reactive rocks in fine aggregate (sand) is about two times two times higher. Reactivity of the aggregate has been determined in accordance with regulations of standard LST 1974-2012 *LST EN 206-1 Application rules and additional national requirements, Annex M.* Gravel and sand found in Lithuania is polymineral. Thus, ASTM- 289 methodology used for the test is not fairly suitable.

The composition of the gravel extracted from gravel pits in Lithuania or produced crushed gravel is polymeric in accordance with separate participles, thus when this aggregate is used, degradation of concrete in volume does not occur when the concrete structure start scaling, however local defects in the structure surface occur in places where a reactive aggregate is close to the surface of the structure.

In designing concrete and reinforced concrete structures conditions that can cause alkali corrosion of concrete should be taken into account and restrictions to components of concrete in case of conditions favourable to alkali corrosion should be introduced - to limit volume of alkalis in cement or concrete mix design and evaluate and limit reactivity of the aggregate (both coarse and fine).

4. CONCLUSIONS

Gravel pits of Lithuania are fouled with reactive particles, containing amorphous silica, which reacts with sodium and potassium alkalis found in cement and causing alkali corrosion of the concrete. Using aggregates from gravel pits of Lithuania for concrete, which is operated in the conditions favourable for alkali corrosion (high humidity and temperature above +20 ºC), some measures to prevent alkali corrosion should be taken.

The determined quantity of reactive rocks in gravel and crushed gravel is similar to the one in other pits of Lithuania (around 2 %) and quantity of reactive rocks in sand is rather big, i.e., around 2 times bigger.

Findings of the microscopy tests suggest that when the reactive aggregate undergoes chemical reaction with alkalis found in the cement, three areas forms around the reactive aggregate: 1) reactive aggregate; 2) area of reactive products; 3) matrix of the hardened cement paste.

The composition of the gravel extracted from gravel pits in Lithuania or produced crushed gravel is polymeric in accordance with separate participles, thus when this aggregate is used, degradation of concrete in volume does not occur when the concrete structure start scaling, however local defects in the structure surface occur in places where a reactive aggregate is close to the surface of the structure.

Alkaline corrosion of concrete with coarse aggregates from gravel pits of Lithuania does not cause decrease of the concrete compressive strength but results in formation of defects in the concrete surface.

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Content and Reactivity of Alkali Silica on Concrete Bridge Structures

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ABSTRACT

This report provides an overview of alkali silica reaction (ASR), its influencing factors, and measures to prevent reaction development. The influencing factors of the reaction were taken into consideration for local Latvian climatic and circumstances of material properties, as it is important to source local raw materials for production of durable concrete structures.

Key words: Alkali Aggregate Reaction, AAR, Alkali Silica Reaction, ASR, concrete, bridges.

1 INTRODUCTION

As the number of reinforced concrete bridges located on Latvian State roads is rather high, the question of concrete durability is the urgent one.

In order to monitor technical condition of these bridges, yearly inspections have to be carried out. The inspection results show that the quality of concrete bridge structures, built over the last decade, is rather low. Many indications of ASR have been found on newly-built bridges. Thus, the bridge over the Baltezera Channel on the route A1 Riga – Estonian border 3.70 km, which was built not longer than 20 years ago, has plain ASR mark *(Fig 1)*. The bridge over the Lielupe River in Jelgava city was constructed in the 1970s. Nevertheless, signs of ASR can already be seen on the bridge’s concrete surface *(Fig 2)*.

Figure 1 – Crack network on deck surface on the Bridge over the Baltezera Channel.

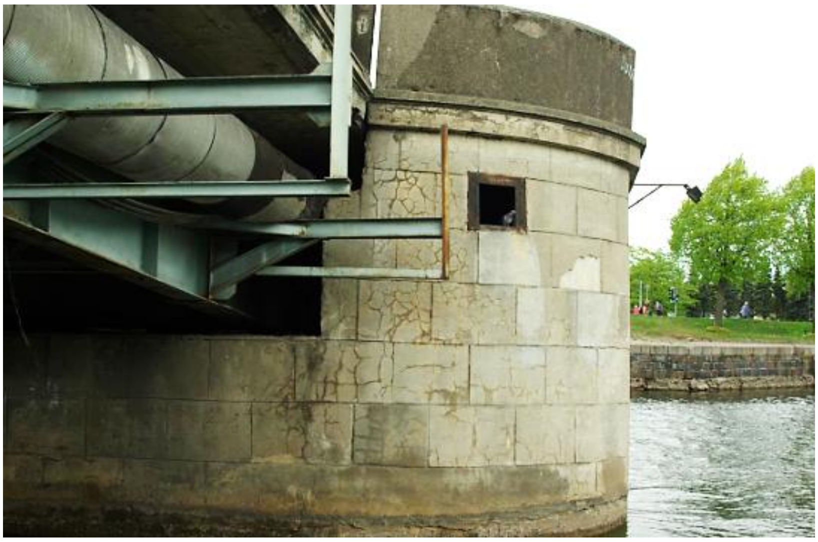


Figure 2 – Crack network on abutment surface on the Bridge over the Lielupe River.

However, conversely, there are a number of longstanding bridges without any ASR marks. The bridge over the Gauja River on the route P25 Smiltene – Strenchi 28.40 km and the bridge over the Salaca River on the route V144 Salacgriva – Vecsalaca 3.10 km are over 100 years old (Figs 3 and 4).

Figure 3 – Bridge over the Gauja River on the route P25 28.40 km.

Figure 4 – Bridge over the Salaca River on the route V144 3.10 km.

To clarify the reason of such an early ASR appearance on recently – built bridges, in 2011, SJSC Latvian State Roads commissioned and Riga Technical University completed the research entitled “Content and Reactivity of Alkali Silica on Concrete Bridge Structures”.

2 RESEARCH OBJECTIVE

The research provides general information of ASR development factors, analyses local climatic conditions and looks at the influence of spot material properties on concrete performance from ASR progress viewpoint.

The main objectives of the research were:

1. to analyse the effects of alkali silica on concrete durability;
2. to collect neighbouring countries’ experience;
3. to observe produced concrete compliance with existing guidelines;
4. to recommend the optimal concrete content for Latvian environmental conditions.

For ASR to occur, certain conditions are necessary - sufficient alkali content in the cement, reactivity of aggregates, humidity. These conditions must be met at the same time and the research observes every influencing factor’s impact on ASR.

3 RESEARCH PROGRESS

3.1 Humidity

Regarding the effects of humidity on ASR, it requires rather a high level of humidity to develop. Accomplished research shows that the more significant expansion, the higher rate of humidity it requires [1] (Fig 5).

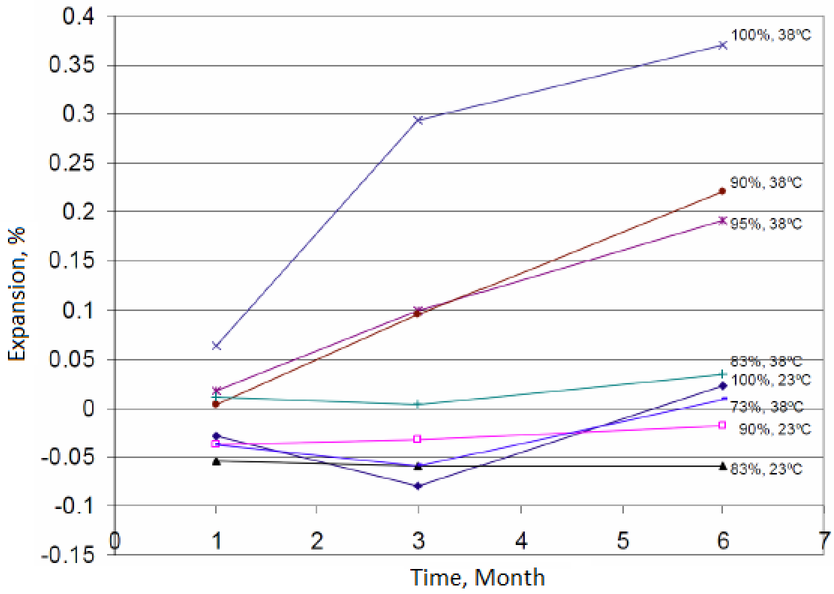


Figure 5 – Mortar bar expansion in diversified relative humidity and temperature.

As humidity is one of the most important factors which influence ASR, it is essential to determine a safe level of humidity in concrete. The research shows that ASR is most active in those structures which are exposed to 80 to 100% relative humidity [2].

With relation to Latvian climatic conditions, there is 80% air humidity on average (Fig 6). It creates perfect conditions for ASR to develop.

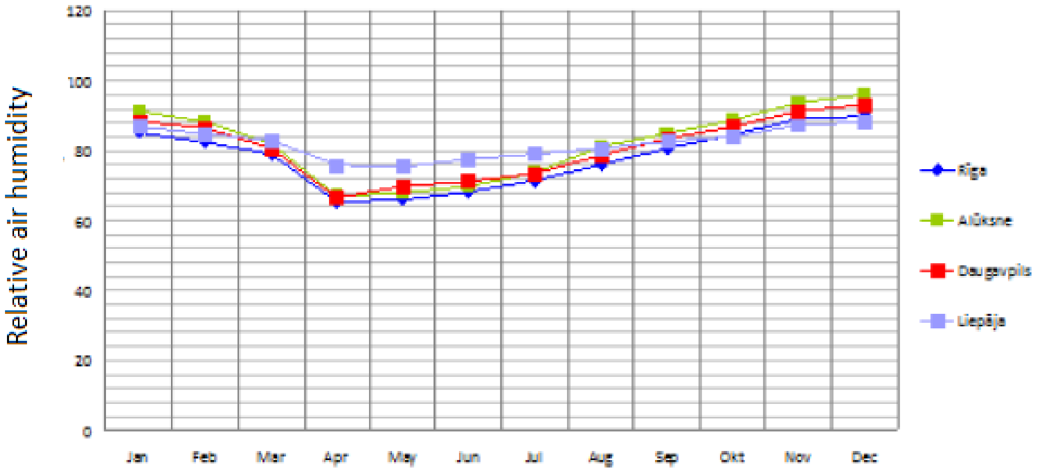


Figure 6 – Monthly relative air humidity in different region on Latvia.

3.2 Alkali content in concrete

Each concrete sample contains certain alkali content. There are 4 sources which may contain alkali. These are cement itself, aggregate, admixtures and external sources, such as de-icing salt. The most significant amount of total alkali content is stored in cement. In order to estimate ASR magnitude expansion test is used as the main measurement method. The research has been done [3] which showed that concrete expansion increases significantly if the equivalent alkali content in concrete exceeds 3 kg/m^3 (Fig 7). Thus, it is important to control the alkali content in cement.

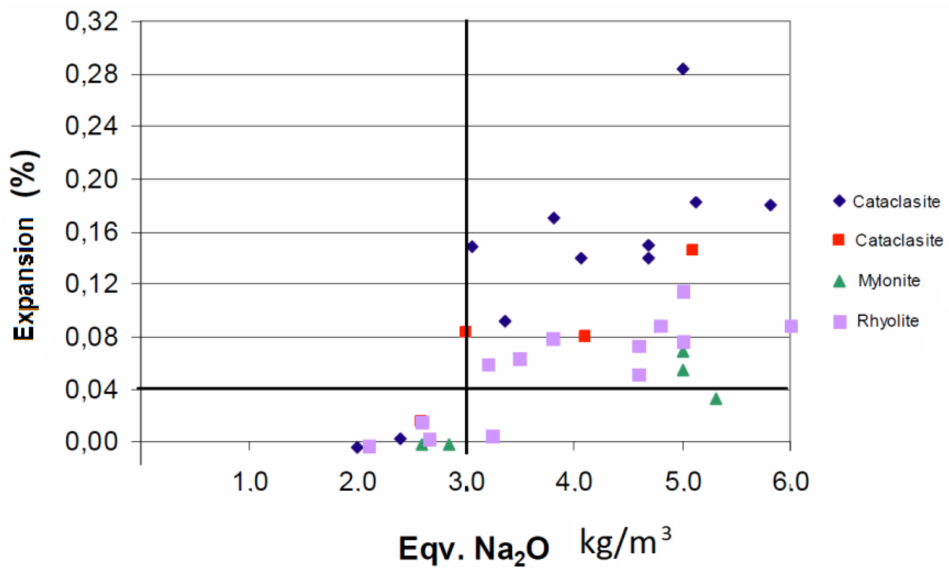


Figure 7 – Expansion magnitude depending on equivalent alkali content [3].

Therefore, there is a National Annex of Latvian Standards to the European Standards for concrete structures, which states that the equivalent alkali (Na_2O_e) content in cement must be less than 0.6%.

“Bridge Specifications 2005”, which offers guidelines for the construction of bridge structures, also defines allowable alkali content in concrete and cement (Table 1).

Table 1 – Requirement for concrete mixture.

Attribute	Environmental impact classes			
	P	M	A	E
Max. water-cement ratio	-	0,55	0,45	0,4
Min. class of concrete strength	C12/15	C25/30	C35/45	C40/50
Min. class of cement strength	32,5	42,5	42,5	42,5
Min. cement content in concrete, kg/m^3	260	300	320	340
Max. ash and cement ratio	-	0.33	0.33	0.33
Max. micro-silicon and cement ratio	-	0.11	0.11	0.11
Equivalent alkali Na2O$_{eq}$ content in cement, (%)	-	$\leq$ 0.60	$\leq$ 0.60	$\leq$ 0.60
Max. equivalent alkali Na2O$_{eq}$ content in concrete, kg/m^3	-	$\leq$ 3.0	$\leq$ 3.0	$\leq$ 3.0
Chloride content class	Cl 0.40 Cl 0.20	Cl 0.20	Cl 0.20	Cl 0.20 Cl 0.10
Min. air content in fresh concrete (%) from total volume	4.5	4.5	4.5	4.5
Min. air content in hardened concrete (%)	4.0	4.0	4.0	4.0
Frost resistance test	-	good	good	good

However, studies and collected data show that there is no cement with low alkali content in Latvia, or in the other Baltic countries. Thus, imported cement may cause a significant increase in the cost of concrete, possibly up to 30%.

3.3 Reactivity of aggregate

Silica can be found in many types of sediment with wide amplitude of crystalline structure and particle size. Only mixed structure of silica can cause damages in construction while the smallest particles are able to react significantly with alkali of cement. Thus, there is "Bridge Specifications 2005" which defines additional requirement for aggregates (Table 2).

Table 2 – Additional requirement for aggregates.

Feature	Aggregate type	Precise requirement
Reactivity of alkali, max. expansion in 52 weeks (%)	Coarse aggregates	0.04
Reactivity of alkali, max. expansion in 14 days (%)	Coarse aggregates	0.1
Max. water absorption % from aggregate volume	Coarse aggregates	1.00
Max. expansion in 20 weeks %	Fine aggregates	0.10
Max. content of particles which size < 0,063 mm, from fine aggregate, %	Fine aggregates	3.00

As there is no low alkali content cement in Baltic countries, the only option to limit ASR in concrete is to use of non-reactive aggregates.

To select non-reactive aggregates, the laboratory of Riga Technical University carried out a test on local aggregates. The aim was to check the alkali-aggregate reaction in accelerated testing conditions (RILEM AAR-2). There were 2 concrete mixtures with following proportions (Table 3).

Table 3 – Proportions of tested concrete mixtures.

	Cement+Granite	Cement+Sand
Cement	1000 g	1000 g
Aggregate	2250 g	2250 g
Water	500 g	487 g
Concrete slump test	215 mm	220 mm

Modelling an accelerated ageing process, fabricated samples were prepared and tested in lye solution (NaOH) for 14 days (Fig 8).

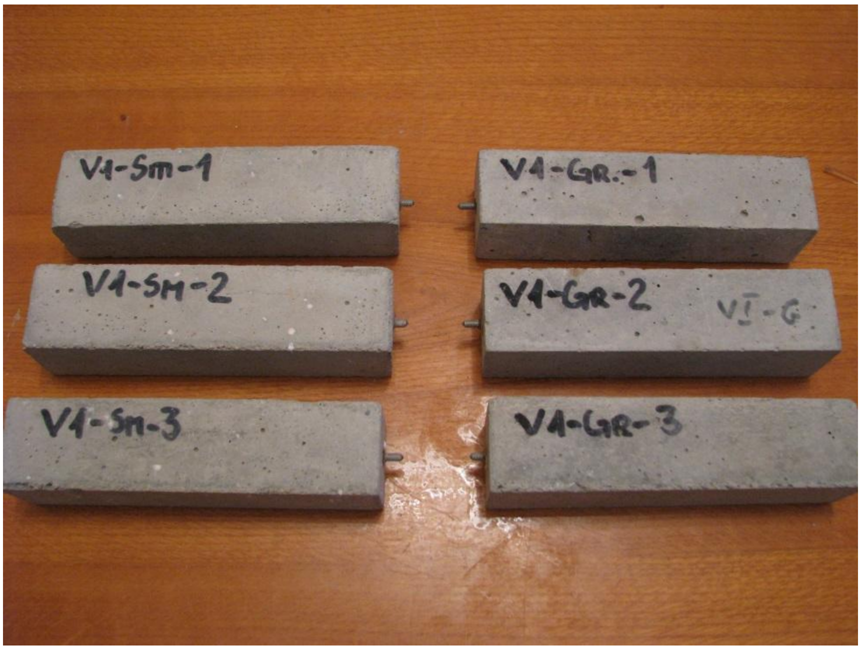
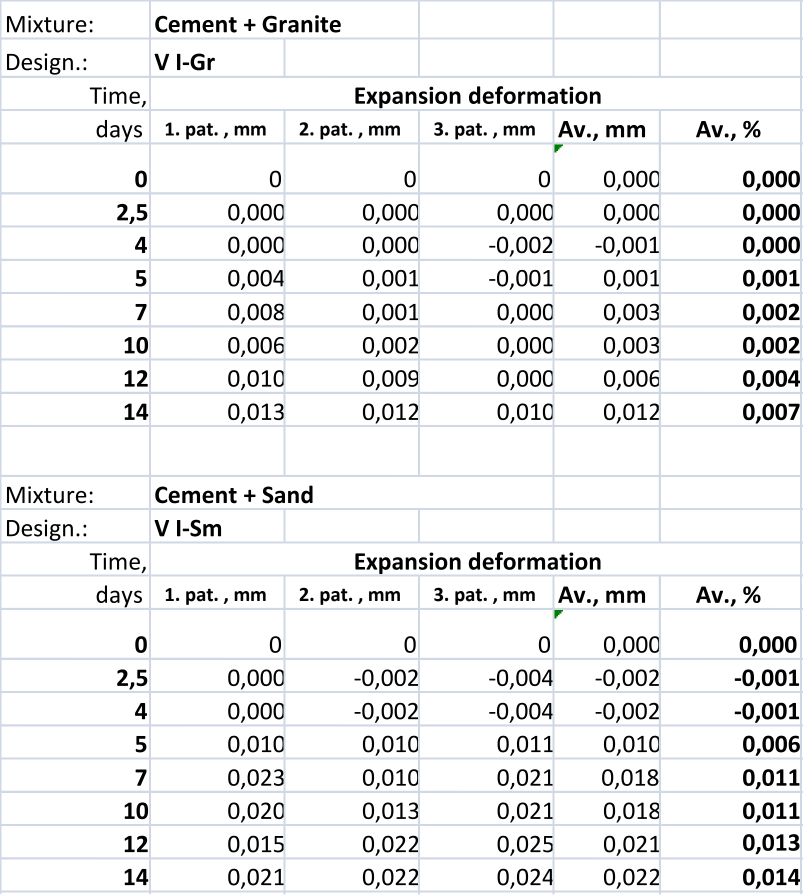


Figure 8 – Concrete mixtures samples for RILEM AAR-2 test.

Gained results showed that critical expansion deformations had not been exceeded and that tested aggregates were appropriate for concrete production. Expansion measurement results are showed on Table 4 and Figure 9.

Table 4 – Expansion measurement results.



Mixture:	**Cement + Granite**				
Design.:	**V I-Gr**				
Time,	**Expansion deformation**				
days	**1. pat. , mm**	**2. pat. , mm**	**3. pat. , mm**	**Av., mm**	**Av., %**
0	0	0	0	0,000	**0,000**
2,5	0,000	0,000	0,000	0,000	**0,000**
4	0,000	0,000	-0,002	-0,001	**0,000**
5	0,004	0,001	-0,001	0,001	**0,001**
7	0,008	0,001	0,000	0,003	**0,002**
10	0,006	0,002	0,000	0,003	**0,002**
12	0,010	0,009	0,000	0,006	**0,004**
14	0,013	0,012	0,010	0,012	**0,007**
Mixture:	**Cement + Sand**				
Design.:	**V I-Sm**				
Time,	**Expansion deformation**				
days	**1. pat. , mm**	**2. pat. , mm**	**3. pat. , mm**	**Av., mm**	**Av., %**
0	0	0	0	0,000	**0,000**
2,5	0,000	-0,002	-0,004	-0,002	**-0,001**
4	0,000	-0,002	-0,004	-0,002	**-0,001**
5	0,010	0,010	0,011	0,010	**0,006**
7	0,023	0,010	0,021	0,018	**0,011**
10	0,020	0,013	0,021	0,018	**0,011**
12	0,015	0,022	0,025	0,021	**0,013**
14	0,021	0,022	0,024	0,022	**0,014**

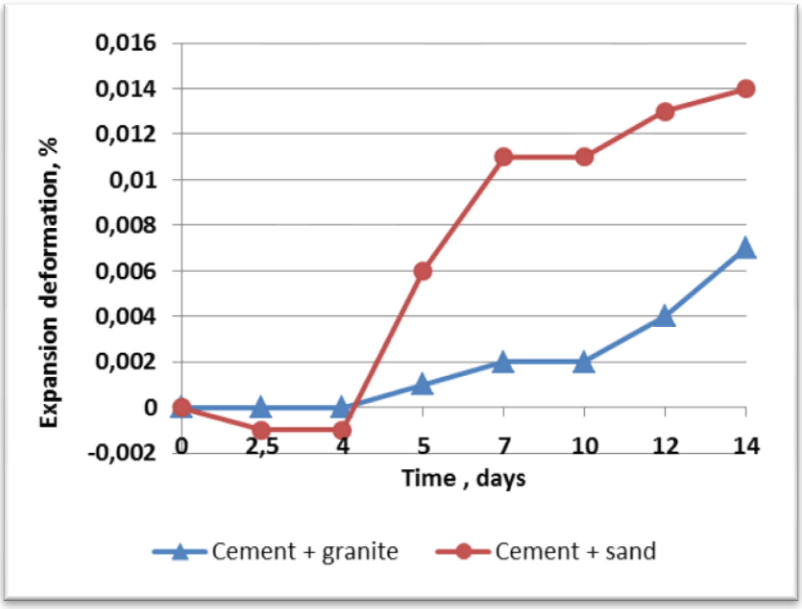


Figure 9 – Expansion magnitude diagram.

4 CONCLUSSIONS

ASR is able to develop just in case if sufficient alkali content in cement, reactivity of aggregates and humidity are met at the same time.

Low alkali cement cannot be obtained in Baltic countries. Consequently, due to imported cement the cost of concrete might rise significantly.

As alkali content in cement and humidity level cannot be influenced to any great extent, the only way to prevent ASR is by using non-reactive aggregate.

Hence, guidelines and standards are being updated and improved, but aggregates are being tested.

The measurement gauge for aggregate expansion was defined in “Bridge Specifications 2005” and in a National annex of Latvian standards to European standards for concrete structures. For cement, in Latvia, it is limited by 0.6%. The total content in concrete must be less than 3 kg/m^3.

To continue with the concrete durability and performance research cycle, research will be carried out on the quality and whole life performance of concrete bridge structures. The key research question is a thorough investigation of ASR. The petrographic test will be used as the main analysis method of concrete structure damage, which allows collecting of a wealth of comprehensive information about cement aggregate reactions.

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Challenge of ASR in the Use of Waste Glass Slurry

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ABSTRACT

Currently the majority of industrial companies in the world develop and implements clean production technologies where manufacturing wastes are recycled and are used for new building materials production. Due glass debris sorting and recycling are practiced at minimum level in Latvia, there should be found alternative ways how to reduce the accumulated glass wastes. The worldwide application of waste glass in concrete was quite limited due to alkali-silica reactivity (ASR) for almost forty years, but this supplementary cementitious material gained again interest in its alternative use in concrete industry in the last decade. It was found that the ground glass particles (<150 μm) initiate pozzolanic reactions without harmful expansion deformations. Size reduction of glass to enhance its chemical reactivity is the key enabling step for converting the landfill-bound waste glass into a valuable product capable of partially replacing cement in concrete. The coarse and fine aggregates can trigger ASR in concrete whereas glass powder can suppress the tendency to ASR and produce an effect similar to that of supplementary cementitious materials such as pozzolan. Waste glass is cementitious in nature when it is finely ground, and especially when it is ground in a wet environment. The obtained waste glass slurries with finest particle size up to 10 μm show pozzolanic characteristics. The ASR tests results indicate that the presence of finely ground waste glass in a wet environment has a relatively small influence on ASR expansion.

Key words: ASR, waste glass slurry.

1. INTRODUCTION

1.1 About glass

The term "glass," in a general sense, is applied to the hard, brittle, non-crystalline, transparent, opaque or translucent vitreous substance which results from fusing silica with active mineral solvents or fluxes, such as the alkalis, earthy bases, or metallic oxides. Silica exists in great abundance, in a free natural state, in the form of flints, quartz, and sand; and in the latter form it is now most generally used for glassmaking. When sand alkali and lead oxide are heated together to a high temperature, the sand is dissolved by the solvent action of the fused alkali and lead oxide until the whole becomes a molten mass of glass. The solvent action of the alkalis, soda potash or lead oxide, is very energetic whilst being heated, and the mass boils with evolution of gases until, at last, the solution, becoming complete, settles down to a clear quiescent molten liquid metal, which is quite soft and malleable, after the nature of treacle. In this condition it is ready for working. The time and temperature necessary for melting such mixtures vary according to the proportions and composition of the ingredients. Silica, combined with alumina and other oxides, is freely distributed in nature in the form of clays, granites, and feldspars, which are also available for use in glassmaking. Originally glass was made by using crushed and ground flint stones as the source for the silica: hence is derived the old name of "flint" glass; but now the large extensive deposits of white sand present a much more convenient and less expensive source, and sand has become universally used [1].

From the early days of the Ancient Egyptians the art of glassmaking was known, and it is now one of the most important industries, supplying as it does many articles for common domestic use and convenience. Nowadays, the large number of glass types can be classified in several ways, for example, by chemical composition use in the manufacture of glass products, or processing behaviour. The most widely used classification is by chemical composition which leads to three chief groups: soda-lime glass, lead glass and borosilicate glass. Glasses in these categories account for at least 95% of all glass types [2]. The remaining 5% are special glasses manufactured for the most part in very small quantities [3]. The European glass industry is very diverse and covers a variety of very different types of products and technologies, including bottles, jars, flat glass, continuous filament glass fibres (CFGF – not to be confused with insulation mineral wool), flaconnage, tableware, insulation mineral wool, optical fibres and special glass (cathode ray tubes, glass for televisions and monitors, lighting glass, optical glass, laboratory and technical glassware, borosilicate and glass ceramic cookware, etc) [2]. But at the same time due to progressive growth of urbanization increased glass production increases the rate of glass waste. Waste is a problem for the environment. But often, waste is also a resource which should be utilized in an environmentally efficient way [4]. Post-consumer waste glass can be recovered with the purpose of recycling in the glass manufacturing industry, to make products such as container glass, flat glass, etc. The process of recycling consists of a series of steps to condition the waste glass for further use. Typical steps are collection, crushing, sorting, contaminant removal, transport, and final use. Some of these steps can, if appropriate, be by-passed. When no use is found for glass, it is stored or disposed of. The most frequent disposal options are landfilling and incineration. Incineration implies that glass is mixed with other materials in slag, which then can be either used (e.g. in civil works) or landfilled. Disposal of glass waste is a complex problem for many countries in the world. Many governmental and nongovernmental organizations are dealing with this problem, but in spite of efforts, the recycling of glass in many countries is insufficient.

1.2 Glass waste recycling in Latvia

Approximately 50,000 tonnes of glass packaging waste is generated annually in Latvia (see Table 1). There are no recycling facilities for glass in Latvia [5], most of the glass waste is landfilled or exported by several companies dealing with preparation of glass for re-use to the neighbour countries. The opportunities for the recovery and recycling of different packaging waste vary between the countries. In Latvia, there is the only one lamp recycling centre in the Baltic States located in Liepaja, which in 2012 recycled 159 tonnes of lamp glass waste (see Figure 1). In Estonia, glass packaging factory exists (Jarvakandi klaas), but it requires a very good quality (the quality from the glass collected through the deposit system is good, but the quality from the glass collected through the containers is mostly not good enough (mixed glass)) and does not accept the green glass. Some of the glass is used to produce street pavements, stones etc, the rest is still exported. Several other recycling projects are in a certain stage, getting 50 % support from the EU Cohesion program. But they are still unable to proceed, as there is no clarity around the waste sector is still too big for the financial institutions to provide rest of the financing [6]. There are four glass recycling companies in Lithuania (CJSC “Kauno stiklas”, JSC “Warta Glass Panevėžys”, CJSC “Alytaus keramika”, CJSC “Stikloporas”) which have quite a high glass recycling capacity (54.7 kt/a) [7], thus, this waste is even imported.

Table 1 – Packaging waste recycling statistics in Latvia within 2003- 2011 years [8-16]

Year	Latvian purchased glass packaging waste (t)	The imported glass packaging waste (t)	Exported glass packaging with goods (t)	Generated glass packaging waste (t)	Reused glass packaging waste (t)	The collected glass packaging waste (t)	Glass packaging waste exported for recycling (t)	Glass packaging waste recycled in Latvia (t)	The total volume of recycled glass packaging (%)	Reclaimed glass packaging waste (t)	Reclaimed glass packaging volume (%)
2003	32585	45034	15826	62193	29010	15829	3333	12523	26	15856	26
2004	44416	55817	18307	73771	27201	24014	7474	14503	30	11851	16
2005	260	4	110	58565	----	----	5786	21558	47	----	----
2006	9682	19445	16609	63639	5173	----	10970	23998	55	----	----
2007	9202	30592	4555	78973	87	----	13122	22917	46	10	----
2008	----*	----	----	66788	----	----	15324	35424	----	----	----
2009	----	----	----	44323	----	----	6692	19690	----	----	----
2010	----	----	----	51896	----	----	7648	24704	----	----	----
2011	----	----	----	51967	----	----	7731	26814	----	----	----

* ---- data is not reftected in the reports

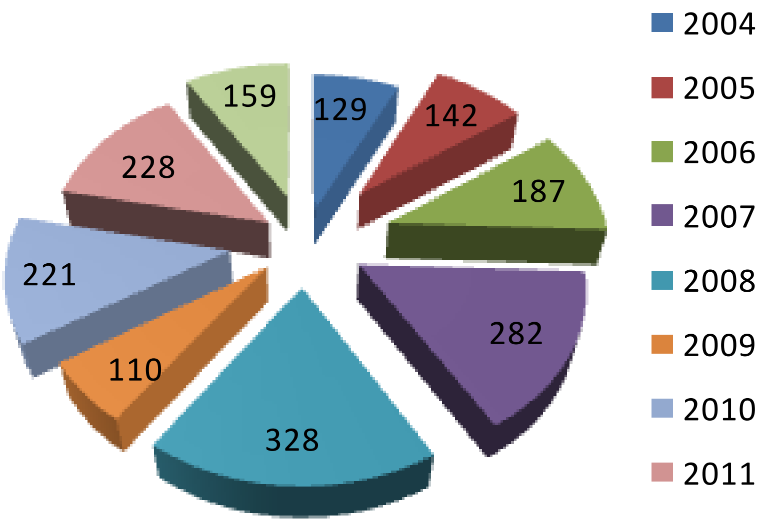

Figure 1 – Incandense lamp and fluorescent tubes recycling statistics [17]

In the middle of the 1990's the Baltic States developed and adopted National Environmental Strategies/Plans in which improvement of the waste management system was defined as one of the main priorities of the national environmental policy. Recognising waste management as a priority for environmental policy, various strategies and plans have been elaborated with the aim of developing an efficient waste management system complying with the European Community's and other relevant international waste management policy principles and requirements. Considering the economic value of the secondary raw material, recovery and recycling schemes for packaging waste were established in many countries even before comprehensive EC legislation was adopted. With the Directive 94/62/EC on Packaging and Packaging Waste, the European Community, as a first priority, aimed to prevent the production of packaging waste and, secondly, to encourage the reuse, recycling and other forms of recovery of packaging waste, to reduce the amount that is disposed of in landfills. Based on the actual amount of packaging waste generated and the availability of recovery and recycling facilities, Latvia has developed directive specific implementation plans for achieving the targets laid down in Directive 94/62/EC [18]. In conjunction with economic growth and increased consumption, packaging amounts (defined as all products made of any materials of any nature to be used for the containment, protection, handling, delivery and presentation of goods that, when discarded) ending up in waste streams during all this time were increasing. Packaging waste was generated along with municipal waste (household and commercial waste). Due to a lack of a separate collection system, packaging waste has mainly been disposed of at landfills. Although certain steps to promote recovery and recycling have been taken, comprehensive reporting on the whole packaging waste management chain has not been performed. The total amount of separately collected packaging waste was less than 5% of the total municipal waste stream in 2002. And that indicated a weakness in the existing reporting/collection system. When, for example, Estonia introduced an excise tax on beverage packaging in 1997, with the goal of increasing recovery and recycling, due to high tax rate, almost all of the companies fulfilled packaging waste recovery targets. In 2003 a new instrument – a deposit system for glass bottles - to increase reuse of refillable glass packaging was introduced in Latvia. The packaging waste management system in Latvia became based on the "producer responsibility" principle - producers and importers are required to organise the packaging waste management system. There were recently founded packaging management organisations whose aim is to improve the situation by establishing separate collection systems also promoting the collection and primary sorting of packaging waste due to that mostly depends on the goodwill and awareness of the public, as so called habit of each citizen to sort wastes at the first place. In 2013 waste sorting (mainly waste glass and plastics) has become more popular in Latvia than it was in the previous years. This is evidenced by presence of special waste separation containers and drop-off points for wastes in Latvia. However, there is still absence of factories that could recycle glass wastes and Latvian recycling infrastructure is based mostly on limited operations like: waste glass collection, sorting and export to EU. As there is absence of data in statistics, as it can be seen from Table 1, there is no complete statistics on waste glass collected and recycled amount in Latvia, according to the regulations the total amount which should be recycled yearly is 50%, and that is shown but other items are not reflected in the last year reports.

1.3 Impact on greenhouse gas emissions

The rapid deterioration of global environmental conditions indicated to society the increasing necessity to debate and react to environmental issues. One of the most important and debated issues is the enhanced greenhouse effect. The Intergovernmental Panel on Climate Change (IPCC) has predicted global rise in temperature and carbon dioxide is a major greenhouse gas responsible for global warming. The Kyoto Protocol was opened for international support on December 11, 1997 at Kyoto, Japan. The main objective of the Kyoto Protocol is to stabilize the greenhouse gas concentrations in the atmosphere, by bringing them to a level that will not interfere with the climate system. Under Kyoto protocol some countries committed to reduce their Greenhouse Gas (GHG) emissions 5.2 % compared to 1990.

According to the International Energy Authority World Energy Outlook 1995, worldwide cement production was responsible for 7% of the total CO_2 emitted around the world [19]. Approximately, half of the CO_2 emitted by the cement industry originates from the fuel and half from the calcinations (chemical reaction) that will convert raw materials into clinker. To produce one tonne of Portland cement clinker in a rotary kiln, some 100 kg of fuel oil is used for the electric energy generation. To make Portland cement, clinker has to be ground to fineness in excess of 300 m^2/ kg. Grinding requires between 35 and 40 kilowatt hours (i.e. 126 - 144 MJ) of electric energy per tonne of clinker, or about 10 kg of fuel oil. Apart from oil, pulverised coal and natural gas, or dual fuels, such as oil-coal, oil-natural gas, and coal-gas are commonly used for clinker making. An average CO_2 emission factor in the fossil fuel combustion process is of the order of 0.22 kg of carbon dioxide for every kilogram of fuel. The electrical energy requirement to produce one tonne of cement is of the order of 440 kWh or approximately 1.580 MJ [20]. Calcination of one tonne of $CaCO_3$ generates 440 kilograms of CO_2, according to the stoichiometry of this chemical reaction. Energy demand for the calcination of one tonne of clinker is about 490 kWh, or approximately 1.750 MJ. Consequently, for every tonne of Portland cement produced, cement plants generate approximately one tonne of carbon dioxide - one of the main greenhouse gases [21]. This figure varies, depending on the type of kiln and the fuel used. For a wet process, common in the developing countries, it is near 1.1 tonnes of CO_2 per tonne of cement; for a more energy efficient dry process it is of the order of 0.9 tonnes of carbon dioxide per tonne of cement, but a tonne per tonne is a good average.

1.4 Waste glass in concrete

Rapid depletion of resources and concerns over degradation of the environment has led to a growing emphasis on sustainable development [22]. As a result, the construction industry is required to adopt environmentally friendly practices and make judicious use of natural resources. The concrete industry can address growing environmental and energy concerns by minimizing the cement content in concrete required to meet the required performance levels; this goal can be accomplished by replacing as much of Portland cement as possible with pozzolanic supplementary cementitious materials (SCMs) [23]. In order to reduce the carbon footprint and the consumption of energy in production of concrete, one can reduce the consumption of concrete, reduce the cement content of concrete, or reduce less clinker for cement production [24]. The consumption of concrete can be reduced through implementation of innovative architectural concepts and structural designs for new construction and for rehabilitations of old structures. The cement content in concrete can be reduced through changes in concrete mix designs (e.g., use of high-range water reducers and aggregate size optimization), partial replacement of cement with supplementary cementitious materials (SCMs) (fly ash, blast furnace slag, natural pozzolans, waste glass). When compared with alternative SCMs, waste glass actually offers a more favourable chemistry for effective use as an SCM in concrete. Glass, which is rich in amorphous silica, has the proper chemistry and reactivity for pozzolanic reactions with the lime released during hydration of cement to produce highly stable end products with desired binding qualities. The recycling of glass in concrete as partial replacement of cement offers three major benefits. First, waste glass has negative value when disposed of in landfills, requiring tipping fees; it assumes a considerable positive value as partial replacement for cement in concrete. Second, as partial replacement of cement, waste glass reduces the significant polluting effects, energy consumption and cost of producing Portland cement. Third, the use of waste glass in concrete as partial replacement of cement yields enhanced physical, mechanical and durability characteristics through pozzolanic reactions with cement hydrates.

Recycling of post-consumer glass for use as raw material in production of new glass is very limited by difficulties related to the classifying operations of the mixed waste glasses. Landfilling of waste glass is not only an expensive way of disposal with a tipping fee of $65 per ton (2001 data), it also occupies increasingly scarce landfill space available in urban areas [25]. Since glass is not biodegradable, its disposal in landfills is not an environmentally friendly solution. There is a strong need to utilize the waste glass in an environmentally friendly way. One option is to recycle waste glass in construction materials, including concrete, where the chemistry of glass can make contributions towards the end product quality [26-31]. So far, the construction industry has made low-value use of waste glass (cullet) as replacement for aggregate.

In the sixties, many studies have been devoted to use crushed glass waste as an aggregate for concrete production [32-34]. This aggregate was also applied in road construction. The glass waste was also used for production of glass tiles and bricks, wall panels, glass fibre, agriculture fertiliser, landscaping reflective beads and tableware [35]. Many other alternatives have been proposed to use waste glass as glasphalt, abrasives, filtration media, binders in ceramics and bricks, additives and fluxes in metal foundry work and as additive for cements [25, 27]. The properties of glass seemed comparable to those of large aggregate in terms of constitution, strength and durability, and the larger size of the glass meant lower processing costs. These early attempts however, were unsuccessful due to the alkali–silica reaction (ASR) which takes place in the presence of the amorphous waste glass and concrete pore solution with marked strength reduction and simultaneous excessive expansion [36]. A pozzolanic material is a siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide to form compounds possessing cementitious properties. It has been demonstrated that such materials require high silica content, an amorphous state and a large surface area [36]. Many industrial wastes or by-product powders, which possess pozzolanic properties, have already been successfully used in concrete mixtures such as fly ashes, granulated blast furnace slags, steel slags [37] and silica fume [38]. Due to high disposal costs of glass wastes, the use of glass as concrete aggregate again attracted the attention of researchers [26, 39-41]. Waste glass offers a highly desirable chemistry for use as a SCM. Size reduction of glass to enhance its chemical reactivity is the key enabling stop for converting the landfill-bound mixed-colour waste glass into a valuable produce capable of partially replacing cement in concrete. This highly promising concept has not, however, been picked up by the cement and concrete industry.

1.5 ASR

Alkali-silica reaction (ASR) is the reaction that occurs between hydroxyl ions in concrete pore water and certain forms of silica which may be present in some aggregates. The product of this reaction is a gel which imbibes water and swells. If sufficient reaction and swelling takes place, the pressures induced cause micro-cracking, expansion and ultimately deterioration of the surrounding concrete.

Recent publications, whilst not specifically supporting the use of glass in concrete, have led to a great understanding of ASR parameters and methods by which it can be suppressed and major recent research in the USA and UK [42; 43] has made it possible for recycled glass to be viewed as a potentially “fit-for-purpose” concrete construction material.

Experimental results have demonstrated that both particle size and chemical composition have an influence on the expansion during alkali-silica reaction [42, 44]. Generally, the finer glass particles exhibit considerably lower expansion; pozzolanic activity increases as fineness increases. Fine particles of waste glass powder also tend to perform a relatively rapid pozzolanic reaction with Portland cement on the contrary to the much slower alkali-silica reaction.

The particle size of glass aggregate was found to have a major influence on ASR expansion. Since the ASR reaction is clearly a surface-area dependant phenomenon, one would expect the ASR associated expansion to increase monotonically with aggregate fineness [42]. However, there exists a size of the aggregate at which the maximum expansion occurs. This is called “pessimum” size, which for regular finer than the pessimum size, processes other than the surface-area dependant become predominant and the expansion decreases with further fineness. In fact, when waste glass was ground to mesh size #50 (300 µm) or finer, no expansion of glascrete mortar bars was observed. This means that the ASR expansion increases with increasing fineness of glass particles up to certain point, and then decreases afterwards [42]. The practical implication of this finding is that waste glass, ground to at least mesh size #100 (150 µm), is not likely to cause unacceptable expansion due to ASR.

It was found that if glass was ground to a particle size of 300µm or smaller, the ASR induced expansion could be reduced and in fact, data reported in the literature that if waste glass finely ground under 75µm, this effect does not occur and mortar durability is guaranteed [36]. Despite the high alkali content of glass, which may represent a typical concern for its use in concrete, studies [44, 30] have shown that finely ground glass does not contribute to alkali-silica reactions. When glass ground to about the same fineness as Portland cement or even finer, below 100 µm, for example in a wet environment [45], waste glass powder has advantageous pozzolanic behaviour: the amorphous silica (SiO_2) reacts with portlandite ($Ca(OH)_2$) generated during cement hydration to form gel of calcium silicate hydrate (CSH).

2. MATERIALS AND METHODS

2.1 Supplementary cementititous material (SCM) – Waste Glass Slurry

Since the chemistry of different types of glass is different, one would expect different ASR expansions. Various types of glass waste cullet were tested, including: soda-alkaline earth-silicate glass (LB) coming from low pressure mercury-discharge lamp waste chips and borosilicate glass (DRL) coming from incandescent light bulb and low pressure sodium lamp waste chips were received from a local lamp recycling centre located (kindly supplied by Agris Magone, Liepaja, Latvia); soda-lime glass cullet was obtained from beverage containers ((F) flint colour) which was collected from a local glass bottle return point and manually by hand crushed into chips under laboratory conditions.

In order to obtain waste glass slurries, washed and dried waste glass chips with total volume of material of 500 grams in each of four containers of laboratory planetary ball mill Retsch PM400 were ground with rotation speed 300 min^{-1} for 120 minutes in a wet environment with water to glass proportion 1:1 by weight (see Figure 2). In total were prepared four kinds of slurries: DRL, LB and F.

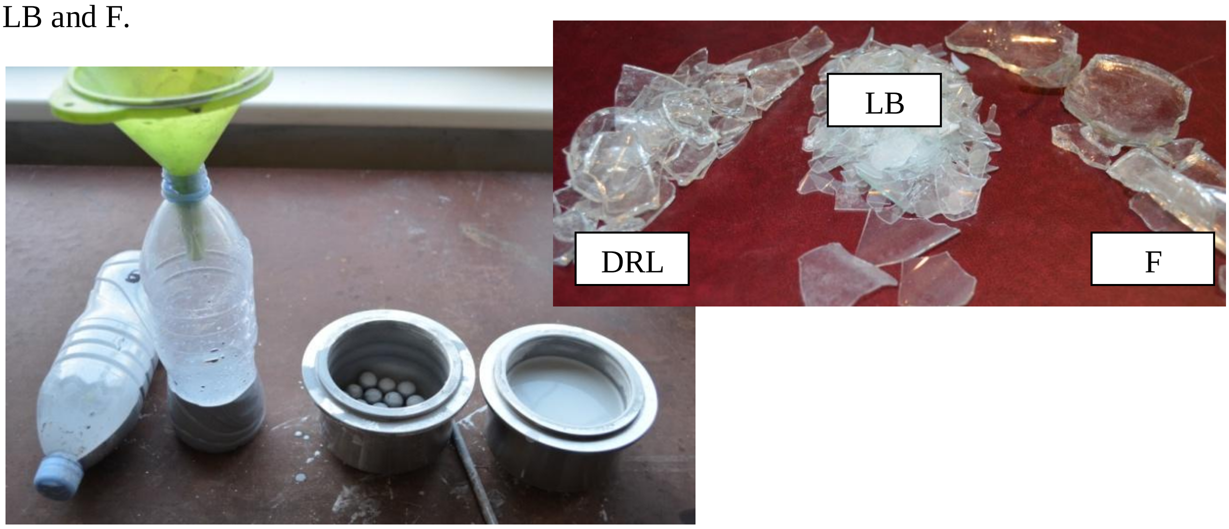

Figure 2 – Waste glass cullet and slurry

Borosilicate glasses

Silicate glasses containing boric oxide have a higher percentage of SiO_2 (70-80%) than soda-lime and lead glasses. Characteristic of this type is the presence of substantial amounts of silica (SiO_2) and boric oxide (B_2O_3> 8%) as glass network formers. The amount of boric oxide affects the glass properties in a particular way. Apart from the highly resistant varieties (B_2O_3 up to a maximum of 13%) there are others that – due to the different way in which the boric oxide is incorporated into the structural network – have only low chemical resistance (B_2O_3 content over 15%) [46]. Glasses having such a composition show a high resistance to chemical corrosion and temperature change. For this reason, they are used in process plants in the chemical industry, in laboratories, as ampoules and vials in the pharmaceutical industry, as bulbs for high-power lamps and low pressure sodium lamps. But borosilicate glasses are also used in the home; backing and casserole dishes and other heat-resistant items can be made from it. The family of borosilicate is extremely broad, depending on how the boron compounds within the glass melt interact with other metallic constituents.

Alkali-lead silicate glasses

Such glasses typically contain over 10% lead oxide (PbO). Lead glasses containing 20–30% PbO, 54–58% SiO_2 and about 14% alkalis are highly insulating and therefore of great importance in electrical engineering. They are used in lamp stems. Lead oxide is also of great importance as an X-ray protective component (radiation shielding glass and cathode ray tube components). In the beginning of 19th century lead silicate was composed of 100 parts sand and 66 parts of red lead fused together. This silicate was mostly used in the manufacture of soft enamels and artificial gems, and went under the names of “Rocaili flux”, “strass metal” and “diamond paste” [1].

Fluorescent lamps are low pressure mercury-discharge lamps which have a fluorescent substance coating the inside of the tube using only 20 to 25% of the energy required for incandescent and other lighting technologies. Installation of high-efficiency lamps reduces the demand for electricity, which in turn reduces the amount of mercury and green house gas (GHG) emissions from utility boilers, particularly coal-fired boilers. In the tube, made of soda-alkaline earth-silicate glass, mercury vapour is excited to produce ultraviolet radiation, which is transformed into visible light when it strikes the fluorescent material coated on the inside of the glass tube. Due to the high ultraviolet absorption of the fluorescent material as well as the glass, no ultraviolet light is emitted from the tube. Here again, lead glass is used to seal the electric leads into the sockets of the tube.

Alkali alkaline-earth silicate glasses (soda-lime glasses)

By far the greatest number of industrially produced glasses belongs to a group of glass types with very similar composition, collectively called the soda-lime glasses. As the name indicates, soda and lime play a major role along with the main component, sand. This is the oldest glass type. A typical soda-lime glass is composed of 71-75% by weight sand (SiO_2), 12-16% soda (sodium oxide from the raw material soda ash or sodium carbonate), 10-15% lime (calcium oxide from the raw material limestone or calcium carbonate), and a low percentage of other materials for specific properties such as colouring. Sometimes magnesium replaces a portion of the calcium contained in the limestone, or potassium replaces the sodium in the soda. Even so, these glasses are similar to and may be classified as soda-lime glasses. Soda-lime glass is primary used for bottles, jars, everyday drinking glasses and window glass.

2.2 Supplementary cementititous materials (SCMs) – Fly ash, wood ash and metakaolin

Fly ash Cemex 450-1 S (FA) was obtained in UK. Wood ash (WA) was obtained in Latvia. Metakaolin (MK) was obtained by heating kaolin at temperature of 800°C for 3 hours in the high temperature universal precise laboratory electric furnace (SNOL 8/1600L). The chemical composition and particle size distribution of SCMs are given in the Tables 2 and 3.

2.3 Cement

Two different cements were used to determine potential deleterious reactions between waste glass slurry and cement matrix. The chosen cements are locally available in Latvia: Kunda-Heidelberg CEM I 42.5N (Estonia) and Aalborg CEM I 52.5R (Denmark).

Table 2 – Chemical composition and Na_2O equivalents of cements and SCMs

Cement & SCMs	Bulk oxide, mass (%)											Na_2Oeq (%)	K*
	SiO_2	Al_2O_3	Fe_2O_3	CaO	Na_2O	K_2O	MnO	MgO	TiO_2	SO_3	P_2O_5		
Kunda	17.93	5.22	2.97	62.03	0.20	1.76	0.14	4.06	0.36	3.76	0.44	1.36	3.71
Aalborg	23.16	2.40	0.30	68.87	0.25	0.06	0.02	0.81	0.07	2.23	0.33	0.29	3.04
DRL	71.14	2.60	0.17	1.320	3.30	1.70	0.006	0.62	0.006	0	0.023	4.42	0.05
LB	65.52	1.22	0.11	5.11	12.35	1.88	0.01	2.95	0.027	0.14	0.038	13.59	0.08
F	69.61	1.34	0.08	11.30	11.21	0.56	0.008	0.46	0.028	0.13	0.021	11.58	0.16
FA	42.1	19.6	10.3	3.3	1.7	2.7	0.1	1.3	1.0	1.8	0.4	3.47	0.52
WA	40.69	14.1	8.29	13.5	0.43	1.34	0.178	3.49	0.631	0.55	0.745	1.29	0.67
MK	46.0	38.0	1.10	0.10	0.20	1.0	-	0.18	0.58	-	-	0.858	0.825

*K – the reactivity index defined as $K=(CaO+Al_2O_3)/(SiO_2+Na_2O)$ [47]

The fineness of cements and SCMs was obtained by automatic Blaine apparatus Zwick/Roell ToniPERM using the rapid method without the need to measure density of the powder beforehand with a pyknometer in accordance with BS EN 196-6 [48] and ASTM C204 [49]. The chemical analysis of the investigated materials was performed in conformity with BS EN 196-2 [50] and Na_2O equivalent as the most significant contributors to ASR expansion was defined (see Table 3).

Table 4 – Particle size distributions and Blaine fineness of cements and SCMs

Volume mean diameter,µm Cements & SCMs	d(0.1)	d(0.5)	d(0.9)	Blaine fineness (cm^2/g)
Kunda CEM I 42.5 N	3.536	21.213	62.051	3888
Aalborg CEM I 52.5R	3.078	14.045	45.330	3580
DRL 1/1/120	0.713	2.835	8.088	-
LB 1/1/120	2.473	8.970	20.088	-
F 1/1/120	1.151	3.306	8.932	-
FA	2.025	13.994	56.281	4360
WA	3.592	21.630	76.254	6600
MK	3.696	11.904	33.096	-

2.4. Preparation and curing of the mortar bar specimens

Mortar bar specimens were prepared following the procedure described in RILEM TC 106 AAR-2 “Detection of potential alkali-reactivity of aggregates – the ultra-accelerated mortar - bar test“[51]. In order to cast mortar bar specimens with dimensions 25x25x285mm, prismatic ply wood moulds were designed. Stainless steel pins of 6 mm diameter and 20 mm length used for length measurements were fixed in the centres of the end plates of the moulds before the casting procedure (see Figure 3).

There were prepared 39 mortar bar specimens for each cement type and three types of waste glass slurries: 3 reference mortar bar specimens, 9 mortar bar specimens with cement substitution with waste glass slurries at 20 wt. %, 27 mortar bar specimens with cement substitution with waste glass slurries at 20 wt. % and fly ash (FA) at 10 wt. % or wood ash (WA) at 10 wt. % or metakaolin (MK) at 10 wt. %. The mortars were mixed in accordance with the procedure for testing the strength of cement with plastic mortar according to LVS EN 196-1 [52]. Three specimens per each batch were made. The mortar workability was measured by flow table test according to LVS EN 1015-3 [53] and was in range of 210-220mm, a superplasticizing agent Sika Visco Crete D 132-2 was added to the mortar mixtures in order to achieve the required flow.

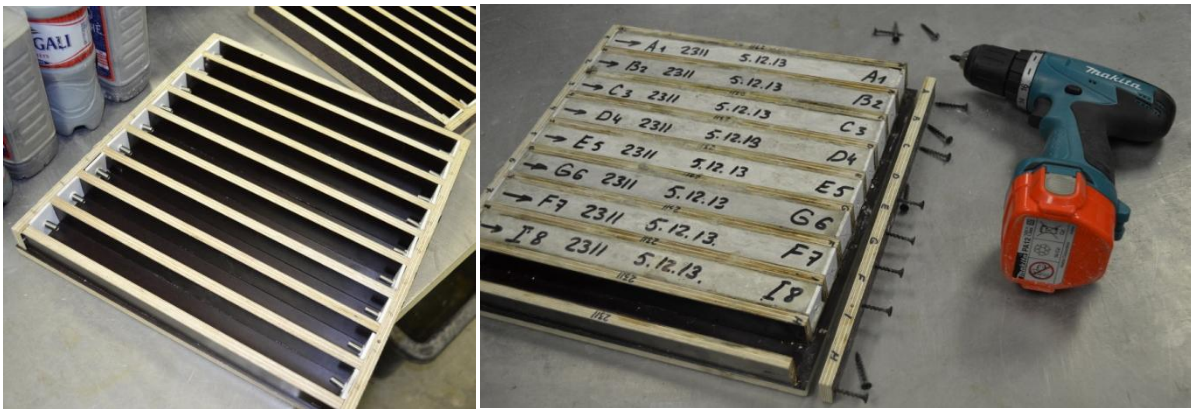

Figure 3 – Preparation of the mortar bar specimens for ASR test with dimensions 25x25x285mm

The mortar bar specimens were cured for 1 day at 20±2°C and 95% relative humidity in the moulds, after 24 ± 2 h they were demoulded and their initial length was measured. The specimens after were placed in water, transferred to an oven at temperature 80±2°C for 24 h, removed from the waster and the length measured immediately before the temperature has dropped substantially. After the specimens were placed in 1 M NaOH solution already at a temperature of 80±2°C, the containers were sealed and placed in an oven at 80±2°C (subsequently 14 days). Length measurements were taken only on 7th and 14th days by means of an electronic comparator (±0.001 mm accuracy).

3. RESULTS AND DISCUSSION

3.1 Expansion tests

The linear expansion of each specimen was obtained by calculating the difference between the length of the specimen at each period of measurement (L_n) and the zero measurement (L_o), to the nearest 0.001% of the effective length, as follows:

$$Expansion,\% = 100 \cdot (L_n - L_0) / \text{Gauge length} \qquad (1)$$

where L_n - reading taken at each period of storage in sodium hydroxide solution, L_o - measurement of specimen before subjection to sodium hydroxide solution, *Gauge length* - distance between inner ends of the metal pins, measures to the nearest 1mm.

According to AAR-2, the aggregates are classified:
- innocuous, if expansions are smaller than 0.10% at 16 days;
- potentially reactive, if expansions are between 0.10% and 0.20% at 16 days;
- reactive, if expansions are greater than 0.20% at 16 days.

The main obstacle to the study of ASR, specifically, the potential reactivity of certain aggregates, is the long-term nature of ASR. The resulting damage can take many years to manifest itself, therefore, accelerated test methods are needed that either compress the material's service life into a reasonable time frame or exaggerate certain reactions in such a way as to enable efficient comparative studies [44]. Nowadays the AAR-2 is considered as not as reliable as it should be due to the test is severe (due to the harsh test conditions which are not representative of the service life encountered by actual concrete structures) and many aggregates that have good behaviour in the field were classified as reactive, and some others that passed the test were found to induce cracking due to the alkali-aggregate reaction (AAR) in concrete [54]. However, it is the most effective of the RILEM methods across the whole range of European aggregates investigated, including the identification of "slowly" reactive aggregate combinations [55]. And it is still the only way to check reactivity of aggregates in a very short time period and several countries have proposed different limits and ways of performing the test.

Figures 6-9 show the expansion of 26 mortars at 16 days. According to the obtained results, it can be seen that only 6 mortars pass the test and are innocuous.

Borosilicate glass lamp waste (DRL) slurry performs the lowest expansions. When is used Kunda cement (with higher alkali level in comparison to Aalborg cement and also high expansions for the reference mortars in range of 0.14-0.15%), DRL performs lower expansions in the combination with metakaolin (0.039% for Kunda) and fly ash (0.06% for Kunda). When is used low alkali cement (Aalborg) the expansion is 0.05%, the expansion of the reference mortar is 0.06%.

Kunda cement has low Na_2O_{eq} (see Table 3) therefore the expansion results for DRL slurries are lower (see Figure 6) and the application of fly ash and metakaolin in the mortars shows the best performance for the experimental programme in the present study.

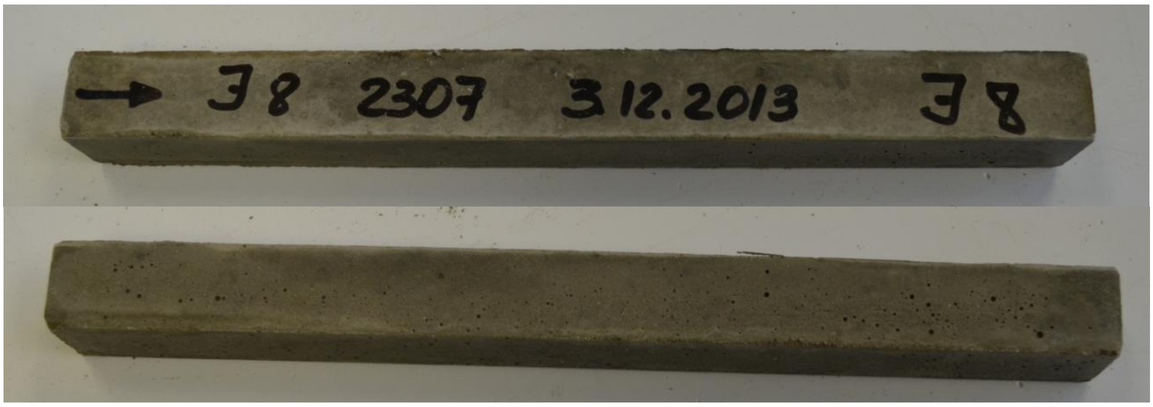

Figure 4 – The mortar bar specimens (Kunda+DRL slurry+MK) before ASR test

Figure 5 – The mortar bar specimens (Kunda+DRL slurry+MK) after ASR test

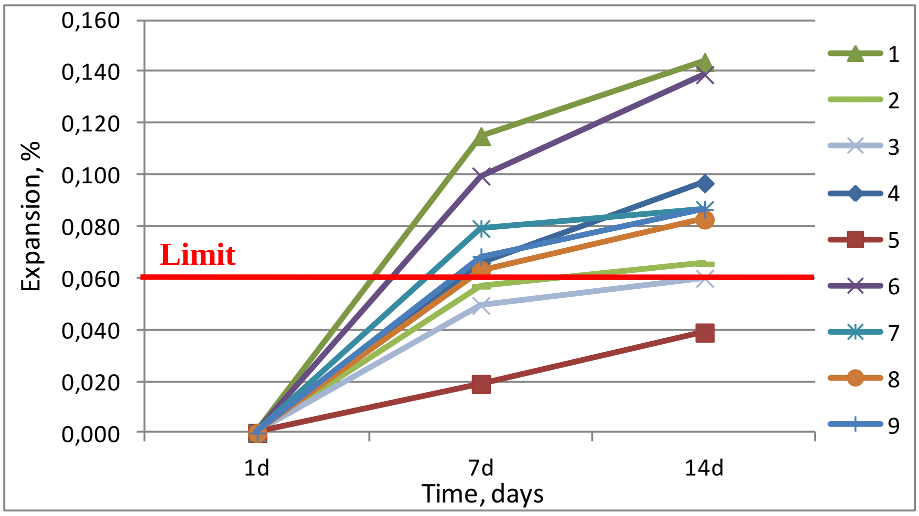

Figure 6 – ASR test results for the mortar bar specimens with Kunda Portland cement: 1) reference specimen, 2) DRL slurry, 3) DRL slurry +FA, 4) DRL slurry +WA, 5) DRL slurry+MK, 6) LB slurry, 7) LB slurry + FA, 8) LB slurry + WA, 9) LB slurry +MK

F slurry in combination with Kunda cement is potentially reactive, the application of WA and MK raise the expansion value over 0.08% (see Figure 7).

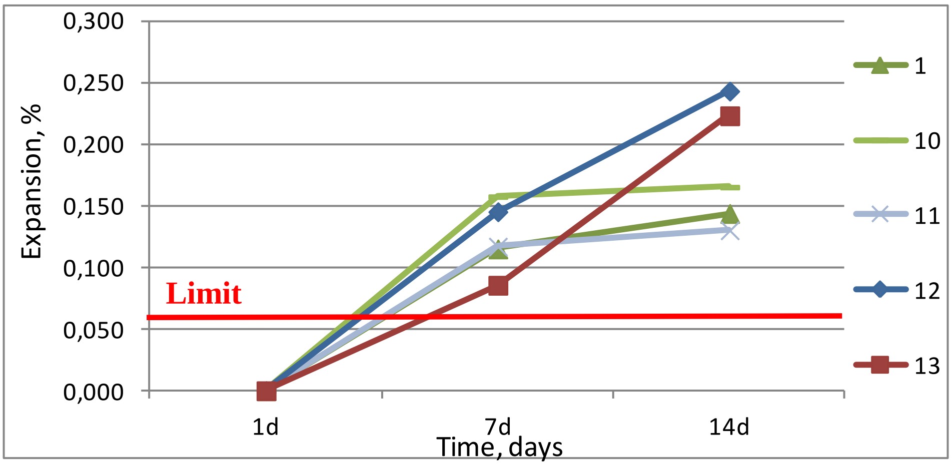

Figure 7 – ASR test results for the mortar bar specimens with Kunda Portland cement: 1) reference specimen, 10) F slurry, 11) F slurry + FA, 12) F slurry + WA, 13) F slurry +MK

Soda-alkaline earth-silicate glass (LB) slurry performs the lowest expansions with Aalborg cement (0.06%).

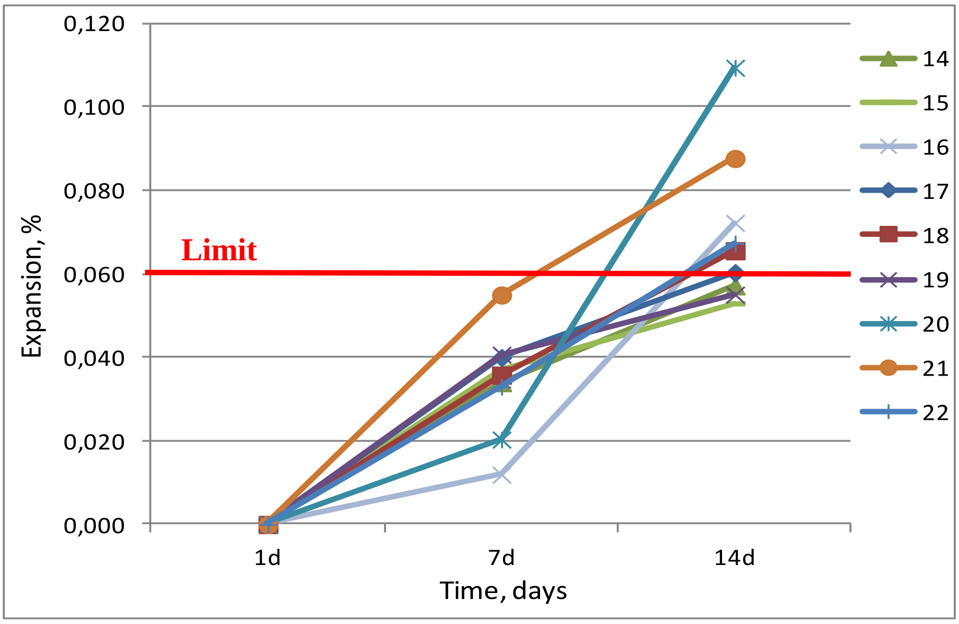

Figure 8 – ASR test results for the mortar bar specimens with Aalborg Portland cement: 14) reference specimen, 15) DRL slurry, 16) DRL slurry +FA, 17) DRL slurry +WA, 18) DRL slurry+MK, 19) LB slurry, 20) LB slurry + FA, 21) LB slurry + WA, 22) LB slurry +MK

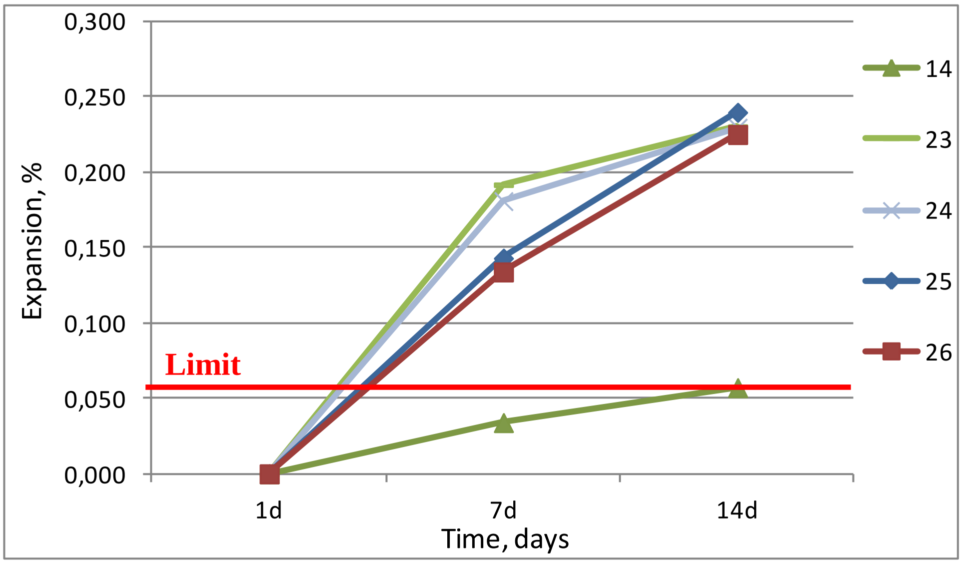

Figure 9 – ASR test results for the mortar bar specimens with Aalborg Portland cement: 14) reference specimen, 23) F slurry, 24) F slurry + FA, 25) F slurry + WA, 26) F slurry +MK

DRL and LB slurries in combination with Aalborg cement are innocuous. The application of fly ash and metakaolin to DRL and LB mortars with Aalborg cement raise the expansions, therefore its application is not reasonable (see Figure 8), however only DRL slurry can be applicable with WA. F slurry in combination with Aalborg cement are reactive, the application of FA, WA and MK gives no efficient impact of the expansion reduction (see Figure 9).

According to [36] the expansion of the mortar bar with 30% cement replaced by fluorescent lamp waste glass powder with particle size 38 µm was reduced to half of that in control. The particles of waste glass slurry (see Table 3) are finer and are below 38 µm and there was observed ASR expansion in the present investigation. However, fineness of aggregate size is no guarantee against expansion [56] and even highly pozzolanic materials cannot be guaranteed to be free of ASR, unless the particles are fine enough [44].

The pessimum size depends strongly on the reactivity of the aggregate, as the aggregate becomes more reactive, the pessimum (particle size that causes maximum expansion) shifts towards smaller particle size [44]. In the present investigation, it is evident that for example DRL has the finest particles therefore it should be the most reactive aggregate, but it is not, the expansion is less than to the reference mortar and LB, F mortars. The expansion of mortar bars is directly proportional to the glass content, higher substitute level – higher expansion, therefore for the present investigation was chosen optimum substitution at level of 20%.

4. CONCLUSIONS

The locally available Portland cement Kunda is potentially reactive; its Na_2O_{eq} doesn't correspond to the requirements according to LVS EN 197-1 [57]. The application of soda-lime glass cullet in concrete with Portland cement Kunda can cause negative effect on durability; however the application of low pressure mercury-discharge lamp, incandescent light bulb and low pressure sodium lamp waste cullet can significantly reduce the expansion making mortars with Kunda innocuous.

The combined use of other supplementary cementitious materials such as fly ash, wood ash and metakaolin not always has positive effect on mitigation of the expansion; as it was observed in the present investigation, it depends on cement type and waste glass used.

The colour of the container glass is normally obtained by adding certain oxides to the glass melt, therefore these oxides of waste glass slurry have significant effect on the ASR results. However, it cannot be postulated that the expansions of mortar bars are strongly dependent on the colour of the glass due to transparent glass (flint colour) can belong to soda-alkaline earth-silicate glass or to borosilicate glass or to soda-lime glass. Therefore, the expansions of mortar bars are strongly dependent on chemical composition of the glass and its fineness.

Substitution of cement with waste glass slurry can offer a slight economic advantage, however environmentally friendly materials, which are produced to save natural resources by using recycled materials, are gaining in popularity worldwide.

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Structural Modelling of ASR-affected Concrete: The approach developed in the PAT-ASR project

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ABSTRACT

The Alkali-Silica Reaction is a harmful reaction which can compromise the integrity and capacity of concrete structures. Due to its nature, a multiscale material model has been chosen to perform structural analyses. The model aims to couple the chemical and mechanical effects in order to characterize the affected concrete into the structures, which is considered as an evolving material. The model has been developed in the project Performance Assessment Tool for Alkali-Silica Reaction (PAT-ASR), which started in 2010 at Delft University of Technology and involves collaboration with the Norwegian Public Roads Administration.

Key words: Alkali-Silica Reaction (ASR), structural effects, material modelling, confinement, PAT-ASR project.

1 INTRODUCTION

The concrete infrastructure comprising bridges, dams and other civil works may be at risk due to deterioration caused by alkali-aggregate reaction. In this group of reactions the Alkali-Silica Reaction (ASR) is considered one of the most harmful processes, because it generates an expansive gel. This reaction, which begins at microstructural level, may eventually cause serious damage with consequent loss of structural capacity.

Due to the nature of the phenomenon, it should be studied from different points of view in order to understand the impact. For this reason, in 2010 at the Delft University of Technology the project Performance Assessment Tool for Alkali-Silica Reaction (PAT-ASR) [1] has been developed. The main goal is an understanding of the consequences of ASR by employing experiments and modelling approaches, both at different scales.

A large experimental campaign focussing on the characterization of a reference material with various tests method has been carried out. Meso-scale modelling of the experiments with the Delft Lattice Model [2] is performed to investigate the reaction kinetics and the damage evolution. Meanwhile a material model for structural analysis [3, 4] is under development in order to couple the chemical and mechanical loading action in ASR-affected concrete structures.

In this paper the idea and the motivations behind the formulation of the material model for the assessment of the structural effects induced by ASR are explained. A literature review is included.

2 EXPERIMENTAL OBSERVATIONS

ASR is a chemical process between the alkali available in the cement and the silica originating from the aggregate. Its product is a hydrophobic gel which expands and builds up a pressure because it is confined by the concrete skeleton.

ASR has been studied by different researchers during several decades. Geologists and chemists investigated the mechanism of the gel formation; whereas civil engineers studied the mechanical impact both at material and structural level.

2.1 ASR mechanism and concrete expansion

The chemical mechanism can be described by a two-stage process [5]. In the first stage the silica on the surface of the aggregates dissolves and reacts with the alkalis in the pore solution, thus creating the alkali-silica gel. In the second stage the gel comes in contact with the moisture and expands.

When the reaction is established, the gel can be observed in different parts of the microstructure. Microscopic investigations show the presence of the gel within the aggregates as well as in the cement paste and at their interfacial zone. As reported by Saouma [6] “Reaction initiates inside some selected (i.e. reactive) aggregates, gel forms, and in the presence of water swells. As it swells in a confined environment, the aggregates eventually will crack to relieve the internal pressure and thus allow the gel to expand inside the newly formed void. One can speculate that the accumulated internal strain energy is much higher than the surface energy of the aggregate itself and that the excess energy drives the crack dynamically into the surrounding matrix and aggregates.”

The expansion of ASR gel can lead to a macroscopic swelling of the concrete with formation of cracks. However, before the gel expansion is visible as concrete swelling the gel flows and fills the existing porosity, afterwards the internal pressure leads to microcracks formation and swelling of the concrete. The swelling appears to stop when either the alkali content is too low to react or the water does not reach the reactive site.

The swelling process can be influenced by the environmental conditions. Elevated temperatures can lead to a faster reaction [7]. Besides, the moisture content is a relevant parameter. Laboratory tests have shown that if the relative humidity is lower than 50% the concrete does not expand, even if the gel has been formed [8].

The swelling process is also influenced by the microstructure. Zhang et al. [9] have reported that the smaller the aggregate size, the greater the concrete expansion, for aggregates between 0.15-10 mm. Multon et al. [10] have shown that if the concrete contains only large reactive aggregates its expansion will be slower with respect to a concrete with different reactive aggregate sizes; this phenomenon can be explained with the difficulty of the alkali solution to enter the aggregates. Moreover when multiple aggregate sizes are considered the behaviour can be different on the basis of the ratio between the coarse and fine particles. This ratio determines how large the porous crown zone around the aggregates is: the larger the porous zone, the smaller the expansion because the gel has more space before pressurizing the concrete skeleton.

2.2 Structural effects

Anisotropic behaviour

The swelling process of concrete affected by ASR appears to be characterized by an intrinsic anisotropic behaviour, as shown by Larive [7]. She observed that a sample in free expansion condition prefers to swell in the direction parallel to the casting direction; the expansion in this direction ranges from 1.3 to 2.8 times the expansion in the perpendicular directions (Figure 1(a)). Tensile tests on sound concrete specimens with the same aggregate size show, as well known, that the tensile strength is lower along the casting direction. This suggests that the distribution of pores with various shapes and orientations determines both the direction with the weakest tensile strength and the preferred expansion direction. Before microcracking occurs, the swelling is nearly isotropic. Afterwards the gel expansion will induce the propagation of the cracks in the weakest zone (perpendicular to the casting direction), which will mutually influence the further swelling. In conclusion, anisotropic cracking resulting from anisotropic strength properties influences the anisotropic expansion.

The same conclusion could be used to explain the swelling redistribution concept [11, 12]. In specimens subjected to uniaxial compressive loading or lateral constraining the imposed expansion is lower in the restrained direction. Once again the gel expansion induces the crack propagation in the direction which requires less energy dissipation. When the constraints are applied in the lateral direction the gel tends to expand along the longitudinal one (Figure 1(b)).

The anisotropic behaviour induced by the stress state is more relevant than the intrinsic one. This phenomenon is particularly important when the attention is focussed on the behaviour of ASR-affected concrete structures, where the concrete is always constrained or (pre-)stressed. It is thus expected that concrete in ASR-affected structures can be seriously influenced by the coupling between chemical and mechanical loading and constraints.

Mechanical degradation

The ASR reaction appears to degrade the mechanical properties of concrete differently than a mechanical loading [13]. Tests performed by Swamy and Al-Alasi [14] showed that: “the losses in engineering properties do not occur at the same rate or in proportion to the expansion undergone by the ASR-affected concrete”.

In Figure 2 the degradation of the Young’s modulus and the splitting tensile strength is collected using data reported in literature [7, 14-17]. The values of the mechanical properties have been normalized with respect to the values at 28 days. The comparison is made for concrete samples stored in high humidity at a temperature between 20 and 40 ºC. The stiffness appears to be the most sensitive property; it can reach degradation levels up to 80%. Moreover the stiffness and the strength degrade at a different rates.

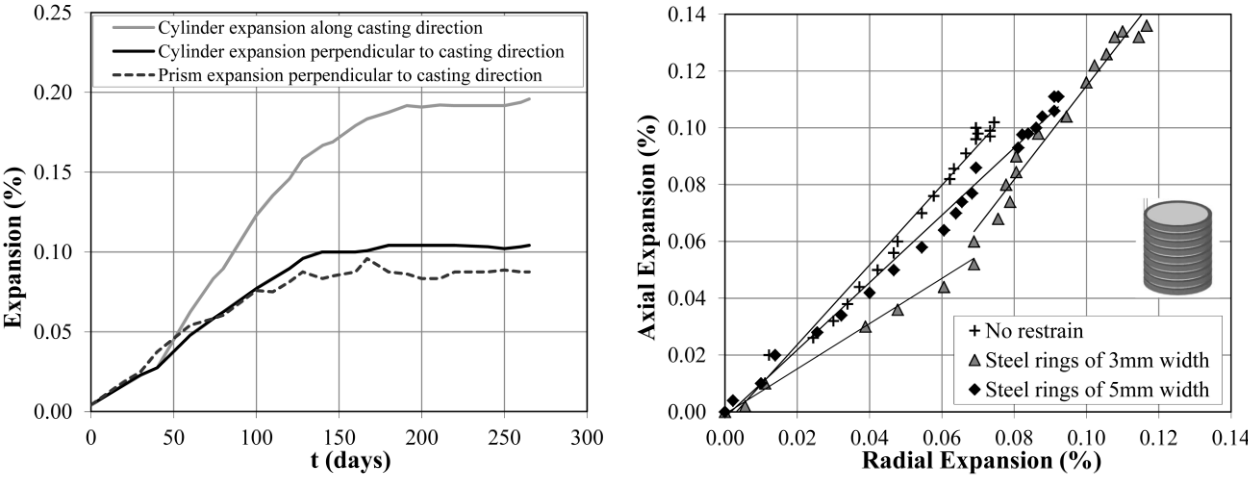
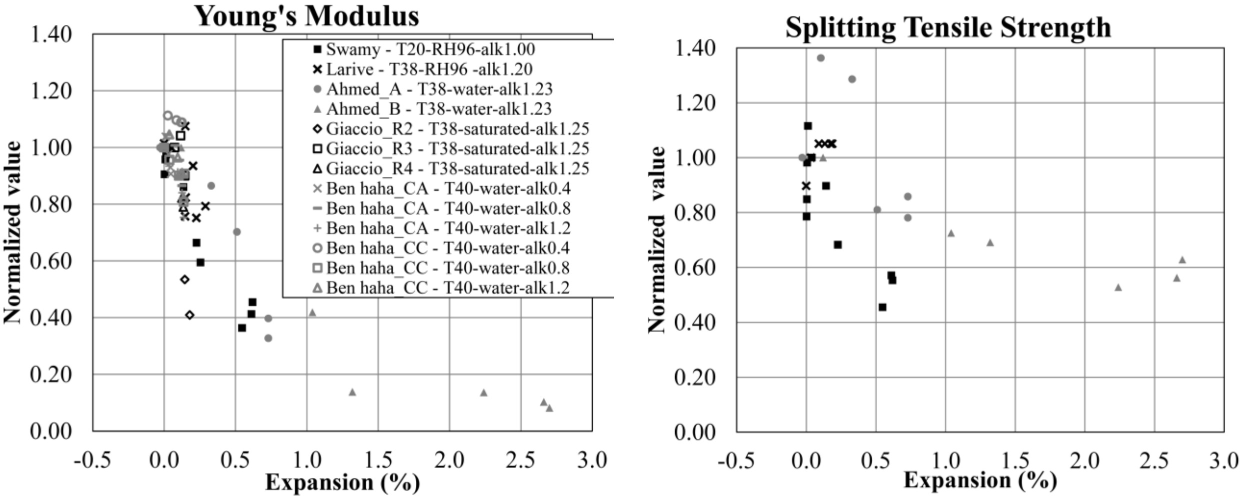


Figure 1 - (a) Expansion curves for cylinders specimens and prisms specimens in free expansion conditions [7], (b) Expansions for specimens subjected to lateral constrains.[11].



(a) *(b)*

Figure 2 - Degradation of mechanical properties in ASR-affected concrete samples in free expansion conditions using data reported in literature [7, 14-17]: (a) Young's modulus; (b) Splitting tensile strength. The legend indicates the name of the author and sample, the temperature in degrees Celsius, the moisture condition and the Na_2O_{eq} content in %.

3 MODELING ASR IN CONCRETE

The alkali-silica reaction was first observed in large massive concrete structures as dams. In an early approach a thermal equivalence concept was adopted to model the concrete expansion [18] and the stress-induced anisotropic behaviour was obtained by considering the local principal stresses [19]. Later, the influence of the environmental conditions was accounted by Léger et al. [20] and Larive [7] with phenomenological formulas.

Saouma and Perotti [12] proposed an engineering approach that accounts for the swelling redistribution due to the stress state; they improved the method presented by Charlwood by introducing anisotropic expansion coefficients on the basis of experimental observations. Capra and Sellier [21] adopted a probabilistic approach to model the evolution of cracks in the concrete matrix, due to the internal pressure generated by the swelling gel. The concrete is modelled like a damageable material having elastic and inelastic strains. ASR is modelled using global kinetics including temperature and humidity effects.

Recently, the attention has been shifted to describe the interaction between the gel and concrete skeleton at pore level, by investigating the kinetics of the phenomenon. Lattice models have been employed by Schlangen and Çopuroğlu [22] and Anaç et al. [2] to model the gel formation and its expansion. Concrete is modelled at meso-scale and the particles distribution is determined by image analysis of samples. Different expansion points are randomly selected into the micro-structures. Dunant and Scrivener [23] adopted an extended finite element framework to perform micro-mechanical simulations of free expansion tests. They explain the damage induced by the ASR by introducing growing gel pockets in the aggregates.

Ulm et al. [24] and Bangert et al. [25] employed the porous media theory to describe the gel swelling and the development of the internal pressure as well as the chemo mechanical coupling. Lemarchand et al. [26] were able to describe the kinetics beyond the S-shaped expansion curve and to capture the swelling redistribution effect by simulating the cracks’ closure [27].

Furthermore micro-mechanical models have been formulated to describe the “pessimum size” effect, to obtain more efficient expansion test procedures. This effect is strongly related to the aggregate size distribution and it defines the worst (pessimum) size which leads to the highest concrete expansion within a short time. Bazant [28] proposed a micro-mechanical fracture theory that explained the aggregate size effect in a 2-weeks accelerated test. Suwito et al. [29], Poyet et al. [30] and Multon et al. [31] employed analytically-solved microscopic models to predict the size effect of the aggregates on the concrete expansion in a 1-year accelerated test.

4 PAT-ASR PROJECT: STRUCTURAL MODELING

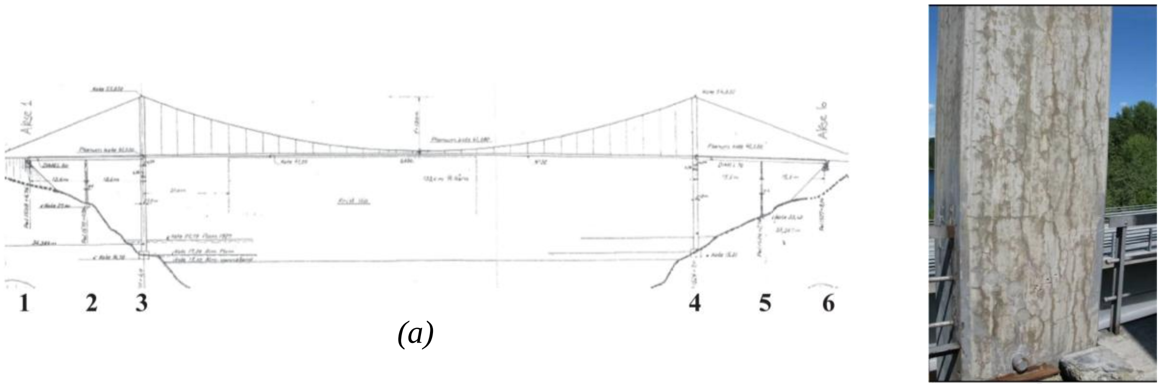
In 2010 the PAT-ASR project (Performance Assessment Tool for Alkali-Silica Reaction) has been established at Delft University of Technology in order to study the main aspects of ASR in concrete. The aim is to provide a tool able to characterize the reaction and its damage both at material level and in the sense of structural response.

4.1 Main case study: Nautesund bridge

Thanks to collaboration with the Norwegian Roads Public Administration, the main case study in the PAT-ASR project refers to the Nautesund bridge. The bridge was built in 1958 and demolished in 2009. Major signs of alkali silica reaction were found in the tower columns, which were extensively cracked. Prior to demolishing visual inspection and mechanical tests on cores and members were performed at the SINTEF Laboratory [32].

The ASR effects were stronger in the tower columns (sections 3 and 4 in Figure 3(a)) rather than in the support column (section 2 in Figure 3(a)). This can possibly be explained from the different confinement effects, induced passively by the reinforcements and actively by the loads. The damaging effect of a swelling ASR gel appears to be strongly influenced by compressive stresses, as reported in literature. The coupling effect between chemical and mechanical loading on ASR damage is a key point for testing this hypothesis.

Inspired by this observation, the PAT-ASR team performed an extensive experimental campaign to characterize the concrete adopted in the Nautesund bridge by performing micro to macro investigations. Furthermore modelling approaches at meso and macro level have been developed. The material model for structural analysis is based on the coupling between the chemical and mechanical loading and aims to be a complementary tool to be used in a structural assessment procedure. The experimental results regarding the mechanical tests on the Nautesund bridge are helpful data for the validation of the model.



(a)

(b)

Figure 3 – The Nautesund bridge: (a) configuration; (b) ASR damage in tower leg [32].

4.2 The structural modelling approach

Considering that a proper material characterization is extremely relevant, a multiscale material model [3, 33] has been selected to perform structural analyses (Figure 4). The model accounts for the micromechanical changes provoked by the ASR swelling. It is able to describe the stress-induced anisotropy effect of ASR, as well as the degradation of the mechanical properties resulting from the combined effect of chemical and mechanical loading. The coupled effect of chemical and mechanical loading is seen as a crucial point for explaining differences between results from laboratory tests and the observed behaviour of ASR-affected structures.

The concrete is modelled at micro level as a multiphase material in which aggregates, cracks and gel formations are considered as embedded inclusions in the matrix that is the cement paste. The development of the gel involves the erosion of the aggregate and its swelling, together with a possible macro-mechanical load, can lead to crack propagation. The overall mechanical properties of concrete are analytically determined with the Mori-Tanaka homogenization method [34]. This theory defines the average 3D stress and strain state of the concrete as well as the effective stiffness tensor, which depends on the amount, the shape and the orientation of the inclusions. The damage is related to crack families with different orientations. The damage evolution is based on the principles of linear fracture mechanics [35].

Even with a relatively simple micro-mechanical model, which allows analytical homogenization, the model turns out to well predict the behaviour of both sound and ASR-affected concrete and to achieve the goal of modelling the chemo mechanical coupling. An analytical homogenization is preferred in order to facilitate its implementation in a finite element program.

This approach allows employing the outputs of the model in a more straightforward approach to be used in practice. Moreover, the model can be helpful to interpret and supplement laboratory tests, which are usually requested to assess the material behaviour of an existing concrete structure suffering by ASR.

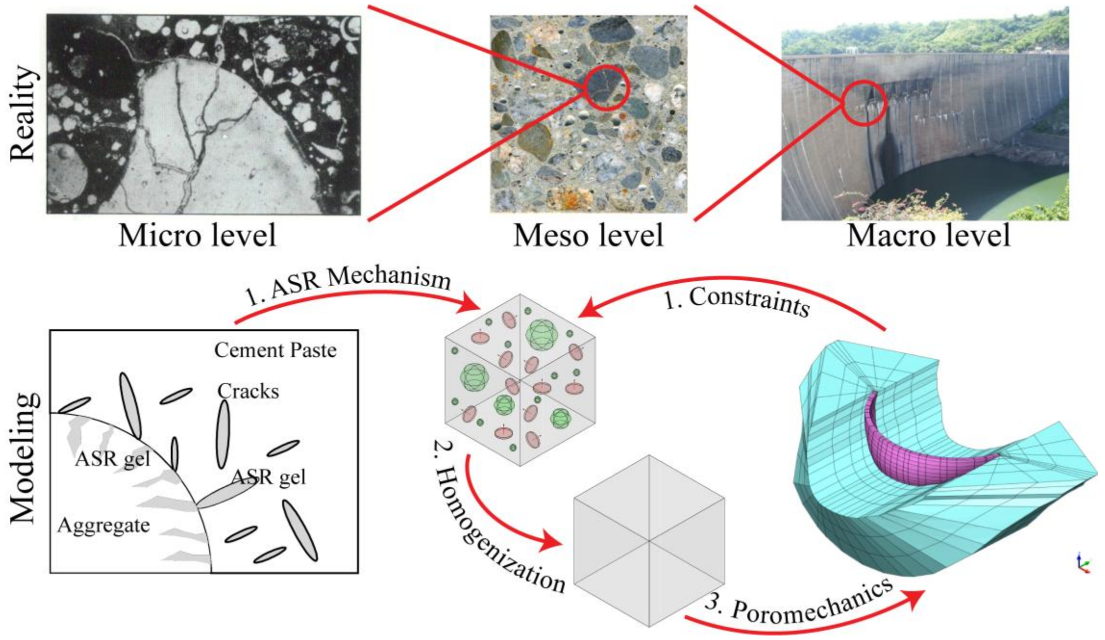


Figure 4 – Modelling procedure for structural analyses.

5 CONCLUSIONS

The alkali-silica reaction is a harmful reaction which can compromise the integrity and serviceability of concrete structures. Involving silica and alkali, respectively available in aggregates and cements, it creates a hydrophilic gel which expands and cause damage. This phenomenon is widely studied in different fields. Geologists and chemists investigated the mechanism of the gel formation; whereas civil engineers studied the mechanical impact both at material and structural level.

Regarding affected structures, one of the most important influencing factors are the mechanical boundary conditions. The damaging effect of the expansive gel appears to be subjected to compressive stresses, which can result in an anisotropic behaviour. In a structure the confinement effect on the gel can be induced passively from the reinforcements and actively from the loads. This is the major difference with most laboratory samples, stored in free expansion.

The main goal of the Performance Assessment Tool for Alkali-Silica Reaction (PAT-ASR) project [1] is an understanding of the consequences of ASR by employing both experiments and modelling approaches, at different scales. A large experimental campaign, involving various tests methods, has been carried out focussing on the characterization of a reference material, which belongs to the main case study: the Nautesund bridge. The experimental campaign has been established thanks to collaboration between Delft University of Technology and the Norwegian Public Roads Administration.

A multiscale material model for structural analysis [3, 33] is under development in order to couple the chemical and mechanical loading action in ASR-affected concrete structures. The model is able to couple the effects of chemical and mechanical loading, which appears to be a major characteristic of affected concrete in structures. The approach is based on a simplified micro-mechanical model to facilitate a further implementation in a finite element framework. The model appears as a complementary tool which can be adopted in combination with laboratory tests in structural assessment procedures.

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NCF Workshops 1975 – 2013

Friday, 13 September 2013

No.	Date	Place and convenor	Theme	Documentation
1	11/09 1975	Dansk Betongforening København (P. Nepper-Christensen, A. Nielsen)	Termiske problemer ved udførelse av massive betongkonstruksjoner	Kompendium utgitt av Dansk Betonforening, november 1975.
2	26/02 1976	Avd. Betongbyggnad, Chalmers Tekniska Högskola, Gøteborg (A. Losberg)	Lettballastbetong som konstruksjons-materiale	Rapport 77:3 fra Institusjonen for konstruksjons-teknikk CTH, 1977. (Ikke anmäldt i Nordisk Betong).
3	19/06 1976	Cement- og Betonginstituttet, Stockholm (U. Bellander, G. Fagerlund)	Accelerade provningsmetoder	Majoriteten av rapporter publisert i Nordisk Betong, 5:1976. Øvrige rapporter (forfattere og titler) listet i samme nr. av Nordisk Betong).
4	23/03 1977	Avd. for Konstruksjonsteknikk, Høgskolan i Luleå. (L. Elfgren, K Gylltoft)	Utmattning av betongkonstruksjoner	Teknisk rapport 1977:57 T fra Tekniska Høgskolan i Luleå.
5	09/06 1977	Avd. för Byggnadsmateriallära Lunds Tekniska Høgskola. (L.O. Nilsson).	Fukt i betong	Anmält i Nordisk Betong 6:1977 av L.O. Nilsson
6	18/04 1977	Forskningsinstiuttet for Cement og Betong (FCB)	Skjær (Skjuvning)	Nordisk Betongforsknings-seminar "Skjær i Betongkonstruksjoner" FCB Rapport STF 65 A77033.
7	29/09 1977	Aalborg Portland, Aalborg (P. Nepper-Christensen)	Bruddmekanikk	Rapport No 6:1977. Dansk Betonforening.
8	26/01 1978	Statens Tekniska Forskningscentral, Betongtekniska laboratoriet. Otnäs. (H. Poijärvi)	Uppvärmning och värmebehandling av färsk betong under arbetsplatsförhållanden	Seminarium för Uppvärmning och värmebehandling av färsk betong under arbetsplatsförhållanden. Rapport från VTT Esbo 1978.
9	24/04 1979	Statens Tekniska Forskningscentral, Betongtekniska laboratoriet. Otnäs. (A. Sarja)	Reparation av betongkonstruktioner	VTT Symposium 7, Esbo 1980.
10	Oktober 1979	Det Norske Veritas (N. Ellingsvåg).	Utmattning av betongkonstruktioner	FCB-rapport. STF65 A80025, datert 80-06-10.

11	16-17/10 1979	Cement och Betonginstitutet, Stockholm, (J. Byfors)	Betong och betongkonstruktioner i tidlig ålder	CBI-rapport finns. Kopia kan rekvireras från CBI.
12	26/11 1979	CBI, Stockholm (C. Molin)	Håltagning och rivning av betong	CBI-rapport finns ej och kommer ej heller att publiceras.
13	21/03 1980	DTH, København (K. Madsen)	Metoder och data för praktisk beräkning av svinn och krypning	Dialog Nordic Seminar on Deformations in Concrete Structures. Copenhagen March 1980, Nr 1-80
14	16/04 1980	Finska Betongforeningen och VTT, Tammerfors	Arbetsmiljø vid betongarbeten	Arbetsmiljøn – riskförebyggande åtgärder vid betongarbeten. VTT-symposium 14/1981
15	21/04 1980	SINTEF FCB, Trondheim	Dynamisk påkjente konstruksjoner	Dynamisk påkjente konstruksjoner. FCB rapport 5TF65 A80032, 1980
16	19/05 1980	Aalborg Portland, Aalborg (Lars Hjorth)	Cementsubstitutionsmaterialer	Aalborg Portland, CBL særtryk nr. 7
17	12/11 1980	CBI, Stockholm	Armering och armeringsarbetetn ergonomi	CBI-rapport
18	15/05 1981	NTH, Trondheim	Ikke-lineær analyse av armerte betongkonstruksjoner	Ikke lineær analyse av armerte betongkonstruksjoner. Institutt for statistikk, NTH Trondheim Rapport nr. 81- 2. Okt. 1981
19	10/12 1981	FCB, Trondheim (O.E. Gjørv / K.E. Løland)	Silika i betong	Condensed silica fume in concrete. Inst. for bygningsmateriallære Rapport BML, 82610 februar 1982
20	23/10 1982	CBI, Stockholm (G. Fagerlund)	Betongs frostbeständighet	CBI rapport 2:83
21	25/05 1983	CBI, Stockholm	Handtering av betong på byggarbetsplatser	
22	05/04 1984	SBI, Stockholm	Samverkanskonstruktioner stål-betong	Stålbyggnadsinstitutet. Publikasjon nr. 92, april 1984
23	23/05 1984	CBI, Stockholm	Organiske fibre i betong	Organiska fiber i betong. Sammenfattningar. Kompendium. CBI, Stockholm.
24	29-30/10 1984	Køge (Dirch H. Bager)	Beton & frost	Dansk Betonforening. Publikation 22:85

25	12/03 1985	VTT Helsingfors	Anslutningar mellan betongelement	“Connections between precast concrete elements”. VTT-symposium 62 (1985)
26	15/05 1985	VTT Helsingfors	Reparation, tilstandsvurdering	”Reparasjon av betongkonstruksjoner” VTT-symposium 66 (1986)
27	23/10 1985	CTH Göteborg	Vidheftning	Bond and anchorage of reinforcement in concrete. CTH, Div. of Cement Structures Publication 86:1, Göteborg
28	05-05/02 1986	Fortifikasjonförvaltningen, Stockholm	Dynamisk belastede betongkonstruktioner	Dynamisk belastede betongkonstruksjoner. Fortifikasjonsforvaltningen Forskningsbyraan. Rapport A4:86 Eskilstuna 1984
29	23/05 1986	FCB/NTH, Trondheim	Utmatting av betongkonstruksjoner	Fatigue of Concrete. Papers presented at a Nordic mini-seminar Trondheim, 1985.SINTEF-rapport 5TF65 A86082
30	21/10 1986	VTT Esbo	Betongkonstruksjoner under tvangsbelastning	Betongkonstruksjoner under tvangsbelastning. VTT-symposium 76
31	06/11 1986	LTH Lund	Bruddmekanik	Fracture Mechanics of Concrete. Division of Building Materials, LTH, Lund November 6/86
32	20/11 1987	Dansk Ingeniørforening, København	Hydrasjon av cement	Seminar om Hydration of Cement, Aalborg Portland, Februar 1988
33	18/02 1988	VTT, Helsingfors	Bestandighet, livslengde	Durable concrete with industrial by-products. VTT Symposium 89. Esbo 1988
34	15/03 1988	FCB, Trondheim	Fliskledt betong i våtrom – skader og utredning	Fliskledt betong i våtrom – skader og utbedring. SINTEF-rapport 5TF65 A88041, juni 1988
35	05/04 1988	DTH, København	Kraftoverførsel til armering i revnet betong	Notits i Nordisk Betong 2:1988
36	15/11 1988	FCB, Trondheim	Karbonatisering av betong	Karbonatisering av betong. SINTEF-rapport 5TF65 A88065

37	09/05 1989	Dansk Ingeniørforening, korrosionscentralen	Kloridinitert korrosjon	Ingen publisering på dette miniseminaret.
38	02/06 1989	FCB Trondheim	Betongkonstruksjoners brannmotstand	Fire Restance of Concrete. Papers presented at a mini-seminar Trondheim 1989
39	23-24/11 1989	CTH Göteborg	Utmatting av betongkonstruksjoner	Fatigue of Concrete Structures. CTH publikasjon P-90:8, Göteborg mai 1990
40	febr. 1990	Vegdirektoratet, Oslo	Høyfast betong i veier	-
41	14/08 1990	BML Trondheim	Fersk betongs reologi	-
42	20/11 1990	KTH Stockholm	Oforstorande provning av betongkonstruksjoner	-
43	17/04 1991	NTH/SINTEF, Trondheim (Ø. Vennesland)	Elektrokjemiske metoder for rehabilitering av armerte betongkonstruksjoner	
44	05/12 1991	HTH, Oslo (E. Sellevold)	Bindemidler i høyfast betong – reaktivitet, struktur og egenskaper	
45	24/01 1992	VTT, Esbo (Juha Saarima)	Mikroskopi och Bildanalys	Microscopy and image analysis of building materials. Espoo, Finland, 1992. VTT Symposium 136, Technical Research Center of Finland, Espoo 1993, 54 pp ISBN 951-38 4087-5
46	19/11 1992	Aalborg Universitetscenter (L. Pilegaard Hansen)	Utmatting av betong	Fatigue of Concrete Structures. Dept. of Building Technology and Structural Engineering, Aalborg Universtetscenter, AUC, Aalborg 1992, 126 pp, ISSN 0902-7513-R9307
47	13-14/01 1993	CTH, Göteborg (L.O. Nilsson)	Kloridtransport i betong	Cloride penetration into concrete structures. Institutionen för byggnadsmaterial, Chalmers Tekniska Högskola, Publication 93:1, Göteborg, Januari 1993
48	22-23/04 1993	Lund University (Göran Fagerlund, Erik J. Sellevold)	Nordisk Miniseminar NBS-MK Frost	Mødereferat

49	Juni 1995	Aalborg	Modern Design of Concrete Structures	
50	16-17/04 1996	Lund University (Sture Lindmark, Göran Fagerlund)	Frost Resistance of Building Materials	Report TVBM-3072
51	08.11 1996	Tekniska Högskolan i Luleå (Hans Hedlund, Patrick Groth)	Licentiatseminarier	
52	28/11 1996	NTNU/SINTEF, Trondheim (E. Sellevold, T.E. Hammer)	Early volume change and reactions in paste – mortar – concrete	
53	10/12 1996	CBI Stockholm	Livslängdbedömning inkluderande korrosionens propageringsskede	
54	10/07 1997	Lund University (Göran Fagerlund, Bertil Persson)	Self Desiccation of Concrete	Report TVBM-3075
55	22/08 1997	VTT, Esbo	Fuktmätning och fukttransport i betongkontstruktioner utsatta för temperatur- och fuktvariationer	VTT Symposium No 174 ”Moisture Measurements”
56	18/06 1999	Lund University (Göran Fagerlund, Bertil Persson)	Self Desiccation of Concrete	Report TVBM-3085
57	31/08+1/ 091999	Lund University (Katja Fridh, Göran Fagerlund)	Frost Resistance of Building Materials	Report TVBM-3087
58	07-08/10 1999	Skagen (Dirch H. Bager)	Water in Cement Paste & Concrete – Hydration and Pore Structure	Workshop Proceeding No.1, Nordic Concrete Federation 1999
59	01/09 2000	DTU / Lyngby (Mette Geiker, Henrik Stang)	Steel Fibre Reinforced Self-Compacting Concrete	
60	26-27/4 2001	Trondheim (Kåre Johansen)	Self Compacting Concrete	SINTEF Report STF22 A01614
61	22-23/05 2001	Chalmers (L.O. Nilsson)	Armeringskorrosion och chlorider i betong	
62	12-13/06 2001	KTH / Stockholm (Johan Silfwerbrand)	Fibre-reinforced Concrete Structures	Workshop Proceeding No.2, Nordic Concrete Federation 2001
63	21-23/11 2001	Hirtshals (Dirch H. Bager)	Durability of exposed Concrete containing Secondary Cementitious Materials	Workshop Proceeding No.3, Nordic Concrete Federation 2001
64	14-15/6 2002	Lund University (Göran Fagerlund, Bertil Persson)	Self Desiccation of Concrete	Report TVBM-3104
65	29/8 2002	CBI (Johan Silfwerbrand)	Diagnos av bärande konstruktioner	CBI rapport

66	22-23/5 2003	Teknologisk Institut (Claus V. Nielsen)	Concrete and Fire	Summary paper in NCR 2:03
67	6/10 2003	Veidekke ASA/Oslo (Terje Kanstad)	Design Rules for Steel Fibre Reinforced Concrete Structures	Workshop Proceeding No.4, Nordic Concrete Federation 2003
68	3-4/11 2003	Teknologisk Institut (Lars Nyholm Thrane)	Form Filling Ability of Self-compacting Concrete	Summary paper in NCR 2:04
69	31/3+1/4 2005	NTNU / Trondheim (T. Kanstad, Ø. Bjøntegaard, E.J. Sellevold)	Crack Risk Assessment of Hardening Concrete Structures	Workshop Proceeding No.5, Nordic Concrete Federation 2005
70	25-26/10 2007	Helsinki (Stefan Jacobsen/NTNU & AkerArktic)	Ice Abrasion on Concrete Structures	Workshop Proceeding No.6, Nordic Concrete Federation 2008
71	15/11 2007	NTNU / Trondheim (Terje Kanstad)	Fibre Reinforced Concrete	Workshop Proceeding No.7, Nordic Concrete Federation 2008
72	12-14/11 2008	Hirtshals (Eigil V. Sørensen)	Nordic Exposure Sites - Input to revision of EN 206-1	Workshop Proceeding No.8, Nordic Concrete Federation 2008
73	2-5/3 2010	Vedbæk (Dirch H. Bager)	Freeze-thaw Testing of Concrete - Input to revision of CEN test methods	Workshop Proceeding No.9, Nordic Concrete Federation 2010
74	13/10 2011	Chalmers / Gothenburg (Karin Lundgren, Mikael Hallgren)	Finite Element Analysis of Concrete Structures	Summary paper in NCR, December 2012
75	15-16/2 2012	Oslo (Bård Pedersen, Claus K. Larsen & Dirch H. Bager)	Durability aspects of fly ash and slag in concrete	Workshop Proceeding No. 10, Nordic Concrete Federation 2012
76	21-22/11 2013	Riga (Børge J. Wigum)	Alkali reactions in concrete	Workshop Proceeding No. 11, Nordic Concrete Federation 2014

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17	10	0	20	29