Protective coatings for the reduction of AAR expansion need to have sufficient compatibility to concrete; i.e. adequate adhesion or penetration (in the case of silanes) to an irregular surface profile and moist alkaline substrate, and the ability to accommodate the expected ongoing AAR expansion Although certain epoxy coating formulations have sufficient adhesive strength to withstand the build up of water vapour pressures, they do not permit the outward movement of water vapour and hence the concrete remains sufficiently moist for AAR to continue. The selected protection needs to be sufficiently robust to withstand the rigours of normal service life, under which the coating needs to remain functional despite the effects of ultraviolet light, wave splash and abrasion (in the case of marine structures), wetting and drying, heating and cooling, etc.

CONCLUSIONS

As a result of research on AAR in the past decade, many cases of AAR-affected concrete structures have been discovered in Australia, and a considerable number of potentially reactive aggregates identified. Because of a poor correlation with service performance of aggregates, it has been found that the current standard test methods for assessing AAR susceptibility of aggregates are inadequate and require revision. Newly developed accelerated mortar bar and concrete prism tests are considered more reliable and should replace the existing test methods.

Further research is needed to determine the influences of various parameters (e.g. temperature, alkali content, mineral additives) on the mechanisms of reaction in the concrete, and their implication for the evaluation of aggregates.

The lack of published Australian data related to repair and maintenance of AAR-affected concrete structures indicates the need to implement long-term investigations to examine such issues as the effectiveness of different treatments, longevity of repairs, the structural consequences of repairs, and the cost-effectiveness of repairs versus replacement.

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AAR IN PORTUGUESE STRUCTURES. SOME CASE HISTORIES

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ABSTRACT

Alkali-siliceous reactions are in the origin of an increasingly larger number of major deterioration in concrete structures. Geological materials that, since long ago have been considered as excellent aggregates for manufacturing concrete are nowadays distrusted because they are the cause of such phenomena. Today, the knowledge of the adequateness of geological materials for that purpose and the measures that became indispensable to avoid or reduce the consequences of their use are a field of research and standardization of undeniable importance and economic validity. This paper presents some considerations on the nature of climatic and geologic conditioning factors in Portugal and some cases of concrete structures affected by AAR are refered to.
Keywords: AAR, climatic factors, geologic factors, case studies.

INTRODUCTION

In 50's and 60's there was a great increase in public works construction in Portugal, mainly dams and bridges. To satisfy the demand there was also an important development of the cement industry and different factories were built all over the country. In spite of the variability of the quarries, the composition of the different Portuguese portland cements did not differ significantly, bearing in general low level alkalis, sporadically raising 0.8% Na_2O equiv.. Otherwise, Portugal has a mild climate and so it could be thought that only in some restricted zones, where special weather conditions prevail, namely high level of humidity, AAR deterioration of aerial structures could be sustained. Although all these circumstances, some cases of AAR have arised in several concrete structures, predominantly in dams.

In Portugal, apart from a small number of ASR cases where reactions have occurred at high rates immediately after construction, most of the cases of AAR deterioration correspond to ASSR of slow-late-expansion type. Taking into consideration the nature of existing aggregates (Silva, H. 1992), (Silva et al. 1996), a future increase of structures damaged by AAR is expected.

This paper presents some considerations on the nature of climatic and lithological AAR conditioning factors in Portugal and describes two cases of damaged concrete structures whose causes of deterioration were investigated at LNEC.

AAR CONDITIONING FACTORS

Climatic factors

Concerning aerial structures in particular, the climatic factors to which works are submitted must be taken into account. The Portuguese climate does not generally present conditions too favourable for the development of this type of pathologies in aerial structures. The country is situated in a region with an Atlantic temperate climate. In average, the maximum annual rainfall reaches in some Northern zones of the country about 2000mm, while in Southern zones rainfall rarely reaches 600mm. The rainfall frequency is also higher in the North, especially in the Western border, close to the sea, where it reaches values higher than 100 days/year.

Furthermore, in the coastal zones there is a larger number of aerial concrete structures due to the higher population density. It is also in the North that is concentrated the highest hydroelectric potential and therefore a large number of concrete hydraulic structures is situated in this region.

Lithologic factors

In Portugal there are almost all types of rocks. The predominant igneous rocks belong to the family of granites and gabbros, mainly basalts, and occupy about 30% of the mainland. The predominant sedimentary rocks are of a carbonate nature, such as limestones, marly limestones, marls, dolomitic limestones and dolomite, which outcrop in about 10% of the territory and of a clastic nature, especially sandstones, conglomerates, sand, and clays, which are distributed over about 20% of the mainland. The metamorphic rocks outcrop in the other 40% of the territory, and are predominantly composed of schists, almost always associated to metagreywackes, quartzites, marbles and rocks of gneissic and amphibolic nature. In some regions calcedonic and opaline silica may occur, such as lyddite, chert, fhtanite and silicifications.

The macrozoning of the geologic aggregate resources which can be sources of reactive silica forms and of alkalies for ASR was carried out and presented also in this Conference (Silva et al. 1996). The best conjugation of geologic and climatic factors for alkali-silicious reactions development in aerial structures has a high probability in the Northern region of the country.

SOME CASES HISTORIES

Very different types of damaged concrete structures have been investigated, ranging from dams (Cabril, Alto Ceira, Pracana among others) to bridges (Duarte Pacheco and Arrabida) and it is assumed that many others are also affected by AAR. Besides structures at mainland it was studied also the case of Santa Maria's runway (Azores) where basaltic aggregates from the island were used.

Next mention is made of two cases where it was diagnosed the occurence of ASR and that prove the reactivity of aggregates, initially not considered as potentially harmful to concrete. The cement used was, in both cases, ordinary portland cement.

Case n° 1 - Alto Ceira Dam

Characterization of the structure and of damage

The dam, on Ceira river in the center of Portugal, was completed in the end of 40's, and started operating in 1950. It is a 36m high cylindrical arch dam with a 103m crest length and a thickness of concrete between 1.5m in the crest and 4.5m in the base of the central cantilever. The aggregates are essentially composed of quartzite in sizes higher than 20mm, quartzite and philonean quartz and siliceous metapelite in sizes between 2 and 20mm, and quartzitic, feldspathic (especially microcline) and rarely metapelitic and micaceous sands in sizes less than 2mm. Quartzite is strongly laminated with high undulatory extinction angles and intergrowths of microcrystalline quartz.

Since its early lifetime, the dam has been presenting progressive vertical displacements of the crest and horizontal displacements of the arch towards upstream. Simultaneously, intense cracking was mainly developed in the horizontal plan due to the liberation of tension stresses throughout the downstream face and in an area corresponding to the oscillation of the operation levels of the reservoir in the upstream face.

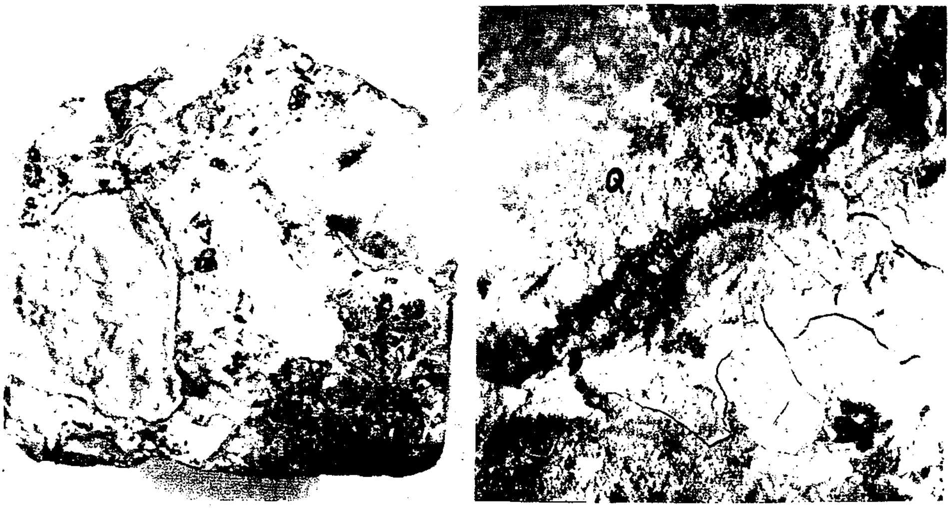
Research results and conclusions

Besides a detailed visual inspection, the methodology of damage diagnosis involved laboratory testing on concrete cores, including in particular, chemical analysis, X-ray diffractometry (XRD), petrographic and SEM/EDAX analysis, reactivity tests of the aggregates and expansion tests of concrete cores (Reis 1991), (Silva & Rodrigues 1990).

Petrographic analysis revealed the existence of ASR phenomena caused by siliceous aggregates, such as microcrystalline quartz occurring between lamellas of quartzite, metapellites and microcline. SEM/EDAX analysis have confirmed the presence of alkaline calcium silicates, predominantly potassic, as gels and crystalline products. Gels in cracks and in reaction rims round aggregate grains and white deposits filling voids and aggregate/paste interfaces were detected directly by visual inspection of concrete cores (Fig. 1). XRD analysis of these white deposits revealed the presence of a crystalline product, characterized by an intense reflection about 12 Å, which was identified as a zeolite type mineral corresponding to an hydrated calcium and potassium silicate. This type of compound is often refered in the literature, as the crystalline ASR rosette-like products (Cole et al. 1981), (Shayan 1988). These rosette-like products were really detected by SEM (Fig. 2).

In order to evaluate the possibility of expansive reactions prosecution accelerated expansion tests on concrete prisms stored in NaCl saturated solution at 50° C were performed (tests based in Danish mortar bar method). The results have shown residual potential reactivity of the aggregates.

Experimental results allowed to conclude that the concrete deterioration process is related to the occurrence of ASR, essentially due to reactive silica of cataclastic quartz (measurements have shown undulatory extinction angles between 25 and 37°) and cryptocrystalline quartz (occurring between lamellas of quartzite and metapellites) and alkalis, mainly potassium, from feldspars, namely microcline.



a)

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Fig. 1 a) Reaction rims round quartzitic grains, b) detail showing a gel rim (G) in the quartzite (Q)/paste interface.

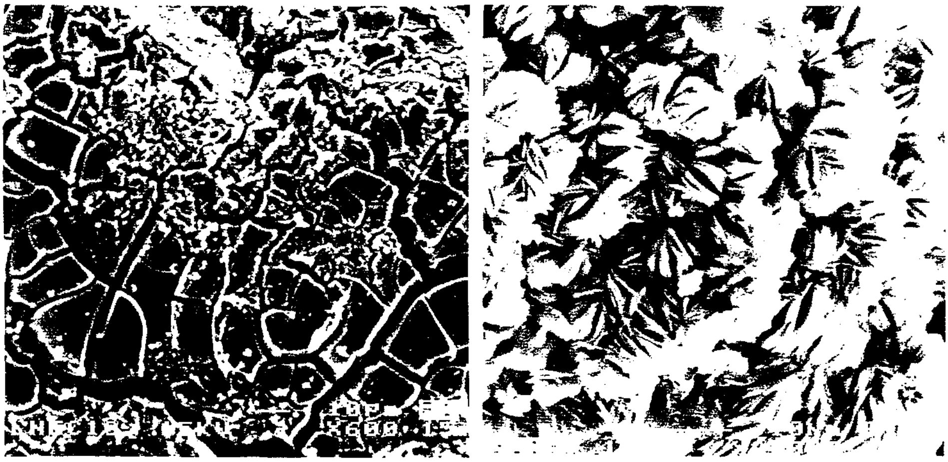


Fig. 2 ASR products in a reaction rim round a quartzite grain, gel and rosette-like crystals.

Case n° 2 - Duarte Pacheco Bridge

Characterization of the structure and of damage

The bridge is located in the Alcântara Valley, a short distance from river Tagus estuary in Lisbon. It was built in the 40's. It is a reinforced concrete structure formed by a roadway supported by slender piers which are supported, in the central zone of the valley, by a circular arch with a span of about 100m linked by 85m long transition portal frames to two lateral arches with a 43m span each. Almost all the information about the structure was lost, but it is known that in its manufacturing coarse limestone aggregate from the region (turonian limestone) and alluvial sands, mainly composed by quartz and some feldspars (orthoclase, albite and oligoclase--albite), were used.

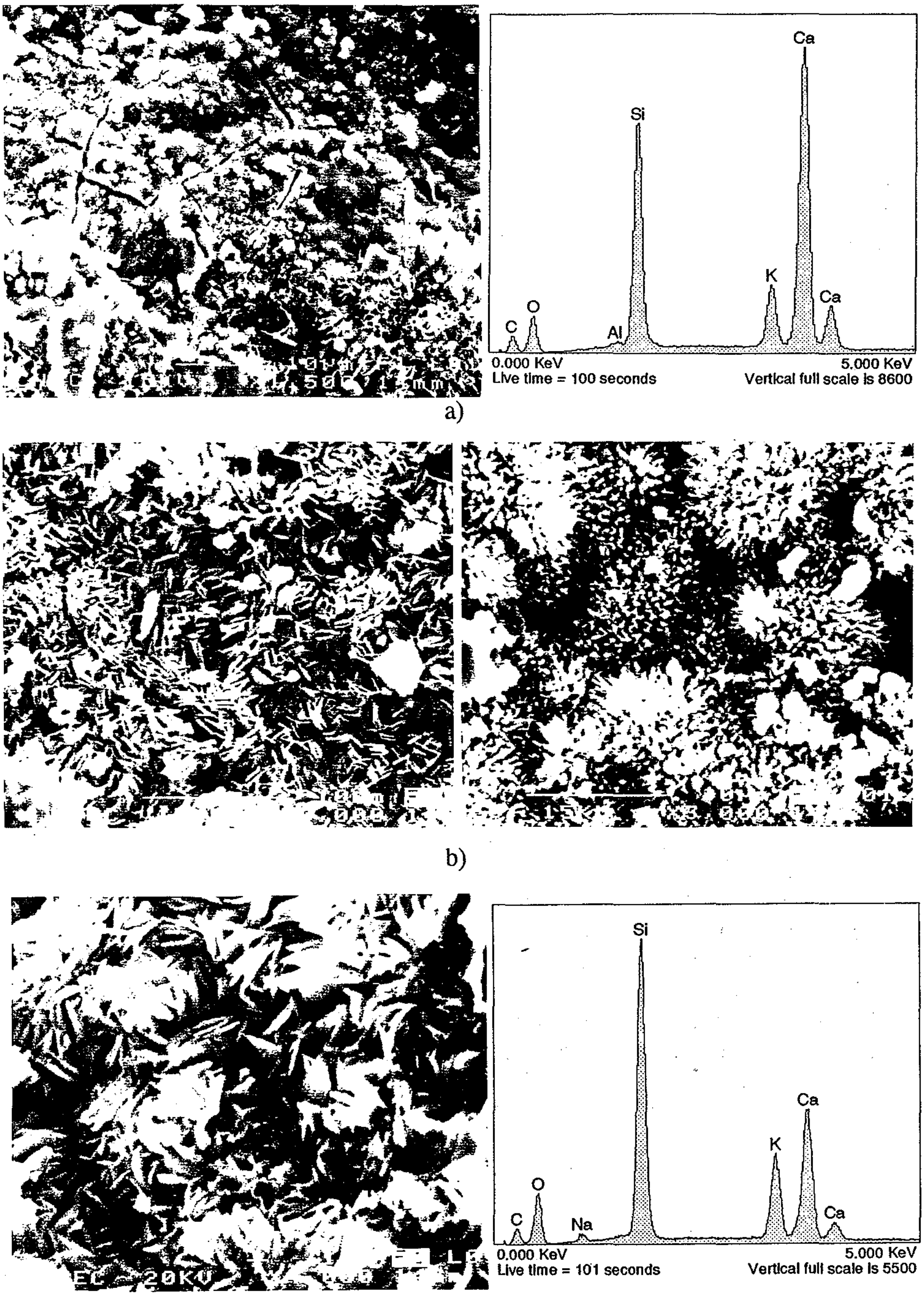
Visual inspection has shown dispersive cracking in arches and piers, more pronounced at the river-side. In the piers it was observed also aligned cracking following the main reinforced bars from 0.1 to 1mm wide. Localized spalling of the concrete is evident. The areas of greater deterioration are located in the central arch over the road where traffic is more intense, specially in zones more exposed to rain and sunshine. The preliminary inspection of the structure had suggested concrete deterioration due to chloride-induced corrosion of the reinforced steel.

Research results and discussion

Inspection methodology included "in situ" tests (carbonation depth and potential corrosion measurements) and core drilling for lab testing. Lab tests included mainly compressive strength of cores and chemical determination of chlorides and sulfates. Later, experimental work was complemented with macroscopic and microscopic examination (binocular, polarising microscope, SEM/EDAX), X-ray diffraction and expansion tests of concrete cores (Reis et al. 1993).

The results of the corrosion potentials measurements have not shown, even in the more cracked areas, the existence of active corrosion at the steel surface. The low concrete chloride contents, allied to the fact that there was not detected by SEM/EDAX chlorides in the rebar corrosion products and that the carbonation near the rebars was not significant, have pointed out to the fact that rebar corrosion was not the initiating mechanism of concrete cracking

SEM/EDAX analysis revealed the presence of typical ASR products, gels and crystalline forms, in paste and limestone/paste interfaces (Fig. 3). These products are calcium and potassium silicates, beeing the gels richer in Ca than crystalline products. The occurrence of these ASR products in limestone/paste interfaces suggest the existence of reactive silica inclusions in the limestone as will be expected in turonian limestones. In fact, petrographic examination of thin sections of concrete samples have confirmed the presence of reactive silica forms (chert and silex) inclusions in the limestone aggregate. It was also observed the occurrence of ettringite, in acicular crystals and massive forms, disseminated in the paste and filling voids and microcracks (Fig. 4). The presence of ettringite was also detected by XRD analysis. SEM examination of the thin sections prepared for petrographic analysis have shown also some interesting aspects revealing signals of ASR and eventual sulfate attack (Fig. 5,6).



a)

b)

c)

Fig. 3 ASR products in limestone/paste interfaces. a) Gel with microcracking and EDAX spectra, b) Lamellar and acicular crystalline products co-existing with gel, c) Lamellar crystals and EDAX spectra.

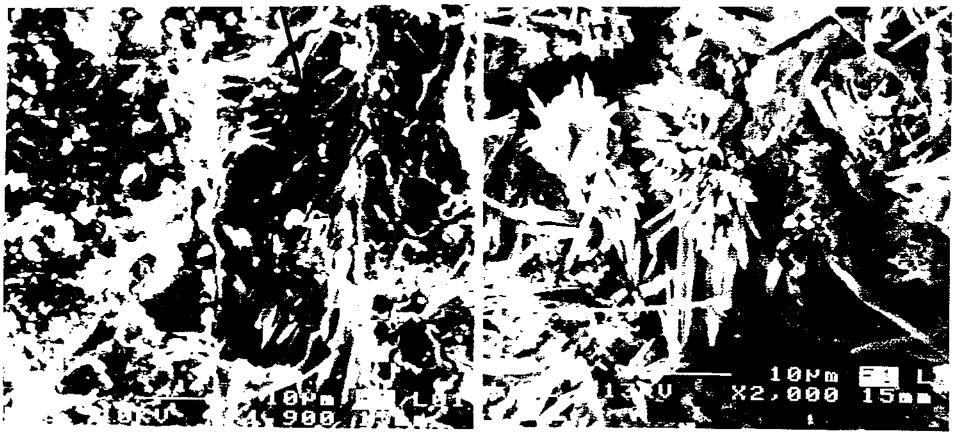


Fig. 4 Ettringite crystals near a steel rebar/cement paste interface. Vein of massive ettringite (arrow) and acicular crystals bunches.

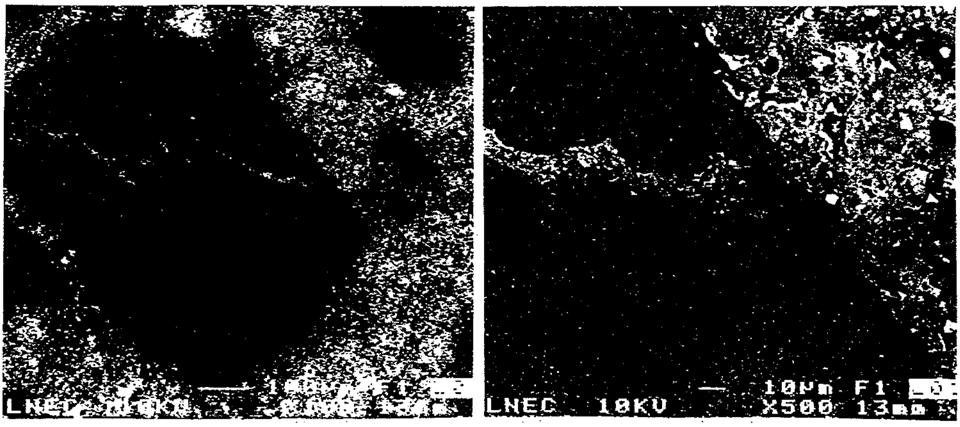


Fig. 5 Microcracked silica grain. Detail showing a vein of calcium alkaline silicate filling the crack.

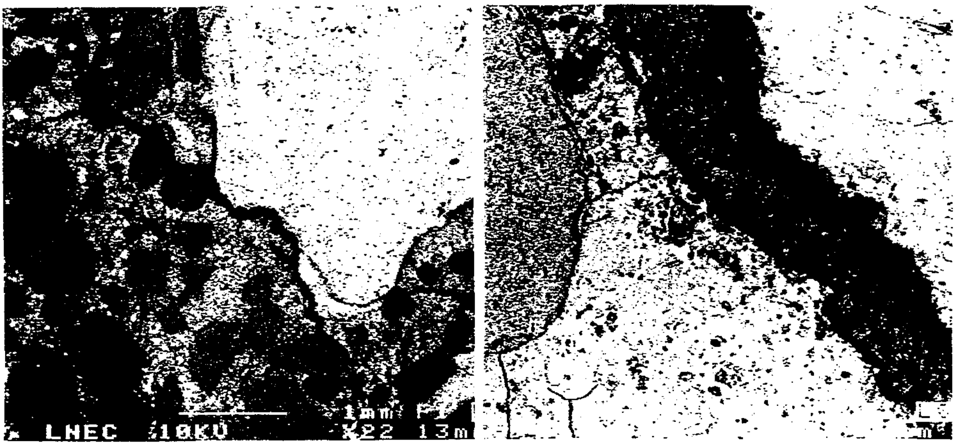


Fig. 6 Microcracking in the paste. Detail showing ettringite crystals filling a crack.

Ettringite formation is often associated to ASR in the concrete. But in the present case the frequency of occurrence may be an indication of sulfate attack caused by sulfates coming from outside. In fact the results of sulfate determinations in concrete have shown a little increase of the content at the concrete surface. Attending to the intense traffic in the area, in particular under and over the central arch of the bridge, the contamination of the concrete by sulfates may be explained by the liberation of SO_2 in the motor vehicles escape gases.

Expansive tests of concrete cores in KCl and NaCl saturated solutions at 50° C have shown residual potential reactivity of the aggregates.

Conclusions

From all experimental results it may be concluded that the concrete deterioration would have its origin in an internal cause in result of ASR, due mainly to the reactive silica inclusions in the limestone aggregates. An eventual attack by sulfates of external origin, as a result of the atmospheric pollution, would also contribute to the deterioration. Subsequently, concrete cracking would provided, in some places, conditions of reinforcing steel depassivation and consequent appearance of corrosion, causing thus the increase of cracks width and concrete spalling.

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CANMET / INDUSTRY RESEARCH CONSORTIUM ON ALKALI-SILICA REACTIVITY (ASR)

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ABSTRACT

In 1991, CANMET initiated a major research project dealing with preventive measures against alkali-silica reactions (ASR) in concrete. The main objective of the above project is to develop a comparative field and laboratory engineering data base on the long-term effectiveness of supplementary cementing materials (SCM) in controlling and/or reducing expansion and cracking in concrete due to ASR. As part of this program, a number of organizations in the private and public sectors have joined CANMET in this research, and as a result an international research consortium has been formed.

Keywords: Alkali-aggregate reactivity, fly ash, granulated blast-furnace slag, preventive measures, silica fume, supplementary cementing materials.

INTRODUCTION

The Canada Centre for Mineral and Energy Technology (CANMET) has been active in the area of alkali-aggregate reactivity for the past 25 years. Earlier research was directed at identifying rock types in Eastern Canada which could be potentially reactive. In recent years, the research program has been expanded to evaluate the usefulness of various test methods in determining the potential alkali-reactivity of concrete aggregates, and to evaluate the effectiveness of various preventive measures against alkali-aggregate reactivity (AAR) (Malhotra & Fournier 1995). In 1991, CANMET initiated a major laboratory and field research program to develop an engineering data base on the long-term effectiveness of supplementary cementing materials (SCM) in reducing expansion and cracking in concrete due to ASR. As part of this program, CANMET invited a number of organizations in the private and public sectors to participate in this research; a consortium was then formed which now includes 15 partners from Canada, U.S.A., Mexico, Japan, and Australia (Fournier & Malhotra 1996).

In this study, a number of reactive aggregates have been selected to be tested in control concrete mixtures, and in mixtures incorporating various types and proportions of supplementary cementing materials (SCM) and other chemical additives. From each of these mixtures, specimens of different types and sizes were cast, and the expansion and cracking will be monitored over a minimum of three years in different accelerated storage conditions in the laboratory, and for ten years for those subjected to natural environmental conditions. The above combinations were also tested in ASTM C 1260 Accelerated Mortar Bar Test to evaluate the usefulness of this method in predicting the long-term effectiveness of SCM in controlling expansion due to AAR.

MATERIALS USED

Aggregates

A total of thirteen reactive coarse aggregates, eight from Canada, one from the U.S.A., three from Australia and one from Japan have been selected for this study. They represent different rock types showing various degrees of alkali-reactivity in concrete (Table 1). The fine aggregate used is non-reactive and of granitic origin.

Table 1: Petrography and results of the Accelerated Mortar Bar and Concrete Prism tests for the aggregates from Canada used so far in the mixing program.

Aggregates	Rock Type	Expansion (AMBT)* 14 days	Expansion (CPT)** 1 year
Gr (quarried)	Granite and granitic gneiss	n.a.	0.035
Su (gravel)	Sandstone, quartzwacke, arkose, greywacke and argillite	0.266	0.095
Po (quarried)	Siliceous sandstone	0.093	0.113
Al (gravel)	Sandstone, limestone, quartzite and fine-grained volcanics	0.293	0.113
Lm (quarried)	Siliceous and argillaceous limestone	0.259	0.133
Sp (quarried)	Siliceous limestone, chert	0.391	0.170
Con (quarried)	Greywacke	0.464	0.175
Spl (quarried)	Greywacke	0.463	0.271

* Accelerated Mortar Bar Method (CSA A23.2-25A; ASTM C 1260)

** Concrete Prism Test (CSA A23.2-14A M94)

Portland cements, SCM and chemical admixtures

High- and low-alkali ASTM Type I portland cements from Canada and Australia have been used in the program. The SCM consist of ASTM Type F fly ashes from Canada, the U.S.A., and Australia, silica fumes from Canada and the U.S.A. and granulated blast-furnace slags from Canada and Australia. A summary of the physical properties and the chemical composition of the cements and SCM used so far in the mixing program is given elsewhere (Fournier & Malhotra 1996).

A synthetic resin type air-entraining admixture was used in all the mixtures. A commercially available sulphonated, naphthalene formaldehyde condensate superplasticizer was used in high-volume fly ash (HVFA) and silica fume concrete mixtures.

MIXTURE PROPORTIONING

All supplementary cementing materials selected for this program were used as replacement, by mass, of the high-alkali cement. The replacement levels tested were 20, 25, 30, and 56% for fly ashes, 7.5, 10, and 12.5% for silica fume, and 35, 50, and 65% granulated blast-furnace slags. For practical reasons, the SCM were used according to regional considerations, i.e. local reactive aggregates being used with SCM available in the regional market.

The nominal cementitious materials content for the concrete mixtures made in this study was 420 ± 10 kg/m^3, except for the high-volume fly ash (HVFA) mixtures (i.e. those mixtures incorporating 56% fly ash) for which the total cementitious materials content was 375 ± 10 kg/m^3. Effective water-to-cementitious materials ratios ranged from 0.37 to 0.42 for all the mixtures except that for the HVFA mixtures for which the value was 0.32. A series of concrete mixtures incorporating selected reactive aggregates were made using a total cementitious materials content of 225 kg/m^3, a maximum size coarse aggregates of 50 mm, and a water-to-cementititous materials ratio between 0.60 and 0.65. This was done to reflect mass concrete for use in hydro-electric dams.

For a selected number of mixtures, reagent grade NaOH pellets were added to the mixing water in order to increase the total alkali content corresponding to the cement part of the concrete system to 1.25% Na_2O equivalent. All the concrete mixtures were air-entrained with the target air content being 6 ± 1%. The dosage of the superplasticizer in the silica fume and HVFA concrete mixtures was adjusted to give desired workability.

Table 2: Physical Properties and Chemical Analysis of the Cements and Supplementary Cementing Materials Used so Far in the Mixing Program

	Cements		Fly Ashes			
	C1 Low Alkali	C2 High Alkali	FA 1 Canada	FA 2 (U.S.)	Blast Furnace Slag Canada	Silica Fume Canada
A. Physical Tests						
Fineness • < 45 μm, %	93.11	90.79	78.19	71.54	98.96	97.48
• Blaine, m^2/g	410	399	262	273	436	...
Specific Gravity, g/cm^3	3.14	3.11	2.46	2.41	2.92	2.15
Compressive Strength, 28 days, MPa	47.3	41.5				
Pozzolanic Activity Index, 28 days, %			90.0	93.9	101.7	118.7
B. Chemical Analysis, %						
Silicon dioxide (SiO_2)	21.15	20.15	41.72	50.16	35.7	93.6
Calcium oxide total (CaO)	60.35	61.72	2.06	2.39	34.3	0.50
Aluminum oxide (Al_2O_3)	4.0	5.52	19.7	26.84	9.6	0.06
Ferric oxide (Fe_2O_3)	5.39	2.59	26.03	12.75	0.55	0.45
Magnesium oxide (MgO)	3.40	2.19	0.87	0.89	14.1	0.67
Sulphur oxide (SO_3)	2.46	4.98	1.08	0.78	3.69	0.32
Loss on ignition	2.25	1.54	3.38	2.80	1.59	2.26
Sodium oxide (Na_2O)	0.13	0.18	0.79	0.26	0.53	0.16
Potassium oxide (K_2O)	0.41	1.09	2.12	2.24	0.42	0.85
Alkalies, (Na_2O equiv.)	0.40	0.90	2.18	1.73	0.81	0.72

FIELD AND LABORATORY TESTING OF SPECIMENS

Laboratory test specimens

A number of concrete cylinders, 100 by 200 mm in size, and concrete prisms, 75 by 75 by 300 mm in size, were cast from each one of the mixtures. Concrete cylinders for compressive strength determination at the ages of 7, 28, 91 days and at later ages, i.e. one, two, and > two years were placed in a lime-saturated water bath for long-term storage at room temperature, i.e. 23 ± 2°C.

The concrete prisms, in sets of three, were tested in the following conditions: (A), 38°C and relative humidity > 95% (control condition), (B), 1N NaOH solution at 38°C, (C), 1N NaOH solution at 80°C, and (D), 5% NaCl solution at 38°C. The length change of the concrete prisms in each of the above storage conditions is being monitored at the ages of 1, 2, 4, 8, 13, 18 26, 39, 52 weeks, and at every three months after up to three years. Petrographic examination of the prisms is also performed to monitor the development of external signs of deterioration.

Test specimens for field exposure conditions

Two blocks, 0.40 by 0.40 by 0.70 m in size, and one slab, 0.70 by 0.70 by 0.15 m in size, were cast from each of the above concrete mixtures. For monitoring the length changes, eight stainless steel threaded studs, 9 mm in diameter by 75 mm long, were partially embedded in the concrete prisms; in the case of concrete slabs only four studs were embedded.

The large prisms and slabs were installed on a gravel pad consisting of a minimum of 0.3 m of well-compacted 0-25 mm crushed limestone material; the first prism is placed directly on the gravel while the second one is placed above ground, on two 200 mm in diameter by 0.40 m long concrete cylinders cut lengthwise (Fournier & Malhotra 1996). The concrete slabs are placed directly on the compacted gravel pad, with granular material being placed around and between each of them so that only the top surface of the slab is exposed (Fournier & Malhotra 1996).

A number of blocks made incorporating selected reactive aggregates and SCM were transported to Treat Island, Maine, U.S.A., for long-term exposure to severe marine environment (Fig. 1D). Slabs were also made from a selected number of concrete mixtures and were placed in compacted 0-25 mm granular material so that only their top surface was exposed; these were made to evaluate the effect of de-icing salt applications on the acceleration of AAR (Fournier & Malhotra 1996).

TEST RESULTS

Since no expansion data are currently available from the field testing program, the following paragraphs will summarize the results obtained so far from the laboratory investigations.

Compressive Strength Testing

The main conclusions for the compressive strength determinations performed so far as part of this study are summarized below:

- Compressive strengths were generally higher for test cylinders made from control mixtures incorporating the low-alkali cement than for those made with the high-alkali cement. Slight to significant reductions in compressive strengths were often observed with the addition of NaOH to the mixing water. Such results have also been reported previously (Shayan & Ivanusec 1989).
- As expected, compressive strengths of fly ash concretes are generally lower than that of control mixtures incorporating the high-alkali cement at early ages, with the difference decreasing with time. Higher compressive strengths are generally obtained with high-volume fly ash (HVFA) concretes than with the corresponding control mixtures at the same ages, except at 7 days. The difference in the compressive strengths of fly ash concretes with and without added alkalies was not found to be that pronounced at the 20% replacement level, being generally within 2 MPa. Significant reductions in the compressive strengths, i.e of 8 to 10 MPa, were noticed when large amounts of alkalies were added to concrete mixtures incorporating 30% fly ash.
- The compressive strengths obtained for the concrete mixtures incorporating 35% slag were similar to those obtained for the control mixtures made with the high-alkali cement; however, the values for the 35% slag concretes were significantly higher than those for the mixtures incorporating 50 or 65% slag. The difference in the compressive strengths between the 35% slag concrete mixtures and both the 50 and 65% slag mixtures was also found to increase with time for a given aggregate. The addition of NaOH does not have a clear effect on the compressive strength of the slag concretes.
- For a given aggregate, the compressive strength values obtained for the silica fume concrete mixtures were generally similar or slightly higher than those obtained for the control concrete mixtures made with the high-alkali cement. Also, for a given aggregate and a given age, only small differences in the compressive strength values, i.e. within 2 to 3 MPa, were obtained between the concrete mixtures incorporating 7.5, 10 or 12.5% silica fume. The addition of alkalies seems to have no significant effect on the compressive strengths of most silica fume concretes.

AAR Concrete Prism Expansion Testing

According to Appendix B of Canadian Standards A23.1-M94, the most suitable method for assessing the efficacy of supplementary cementing materials in reducing expansion due to AAR is the concrete prism test. Current experience is that a testing period of two years (with a 0.04% expansion limit) is sufficient for the evaluation of concretes containing fly ash or slag but particular attention should be paid to the rate of expansion toward the end of the testing period (Duchesne & Bérubé 1992, Thomas et al. 1992). Longer testing periods are required to assess the performance of concretes containing silica fume (Bérubé & Duchesne 1992, Oberholster 1989).

The immersion testing in the 1N NaOH and the 5% NaCl solutions at 38°C were used to determine if better control on the storage or testing conditions could be achieved by using immersion test conditions. No expansion limits are currently available for the above immersion tests at 38°C. The immersion testing in 1N NaOH solution at 80°C has been suggested for reducing the proposed testing time period for AAR. The test is very severe; its real significance and the expansion limits to be used are still open to debate.

The expansion test data obtained in the different storage conditions were analysed and compared in order to determine a series of expansion limits at specific ages; these were then used to evaluate the effectiveness of the various cementitious systems investigated in controlling ASR. The test results are summerized in Table 3.

The significant observations from the length change measurements performed so far are as follow:

- Increasing expansion with increasing alkali content in the concrete mixture is generally observed for control test prisms stored at 38°C and R.H. > 95% (Fig. 2A). Test prisms cast from the control mixtures incorporating the high-alkali cement and added alkalies generally expanded significantly more when tested at 38°C and R.H. > 95% than when immersed in the 1N NaOH solution at 38°C (Fig. 2A vs 2B).
- Test prisms cast from the mixtures incorporating the low-alkali cement generally gave the highest expansion values among the control mixtures when tested in 1N NaOH at 38°C (Fig. 2B). Test prisms made with the high-alkali cement and added alkalies generally gave the lowest expansion levels among the control mixtures when tested in 1N NaOH at 38°C (Fig 2B).
- Expansion values obtained for test prisms stored in the 5% NaCl solution were always found to be significantly less than those obtained for the companion series of test prisms stored either at 38°C and R.H. > 95% or in 1N NaOH solution at 38°C.
- Concrete prisms and mortar bars incorporating 30 or 56% fly ash invariably met the expansion limits proposed for the different test conditons used in this program (Table 3). Test prisms cast from the concrete mixtures incorporating the highly reactive Spl greywacke and 20% fly ash show a slow but steady increasing expansion during the first 104 weeks of testing both at 38°C, R.H. > 95% and in 1N NaOH at 38°C (Fig. 2C and 2D; Table 3). Additional testing will be performed to determine if the failure of the 20% fly ash system with the Spl aggregate was due to the particular composition of the fly ash used or due to the high reactivity level of the aggregate; however, these results confirm previous CANMET observations that indicated that a minimum of 30% fly ash is generally required to control deleterious expansion with highly-reactive aggregates (Malhotra & Fournier 1996).
- None of the test specimens cast from the 50 and 65% slag concrete or mortar mixtures have reached expansions greater than the expansion limit proposed for the different test conditions (Table 3). The use of 35% slag is not considered appropriate for concretes incorporating a highly-reactive aggregate (such as the Sp limestone); however, it may control adequately deleterious expansion with marginally reactive aggregates (such as the Su gravel) (Table 3).
- Expansions for the silica fume concrete test prisms subjected to the various storage conditions were generally found to decrease with increasing silica fume content. Between 7.5 and 10% silica fume seems necessary to reduce deleterious expansion of concrete prisms or mortar bars under the proposed expansion limits for marginally-reactive aggregates such as the Lm limestone and the Su gravel; however, 10% silica fume does not seem to control deleterious expansion with moderately- to highly-

reactive aggregates, such as the Po sandstone, the Sp limestone and the Spl greywacke (Fig. 2E and 2F; Table 3).

- The accelerated mortar bar test, using an expansion limit of 0.10% at 14 days, generally gives a good indication of the effectiveness of SCM in controlling expansion due to ASR; however, there were some exceptions as indicated in Table 3. This 0.10% expansion limit has also been proposed by Davies & Oberholster (1987) and Duchesne & Bérubé (1992).

CONCLUSIONS

Laboratory test results obtained so far have confirmed the beneficial effect of supplementary cementing materials in reducing expansion due to AAR in concrete. The effectiveness of these materials depends on several factors, including the chemical composition of the SCM and the potential alkali-reactivity of the aggregates.

CSA A23.2-14A Concrete Prism test seems satisfactory for evaluating the effectiveness of SCM in reducing expansion due to AAR; however, the two-year testing period is considered long. Testing is currently underway to evaluate the usefulness of other accelerated concrete and mortar methods for predicting the long-term effectiveness of SCM in controlling the expansion due to AAR.

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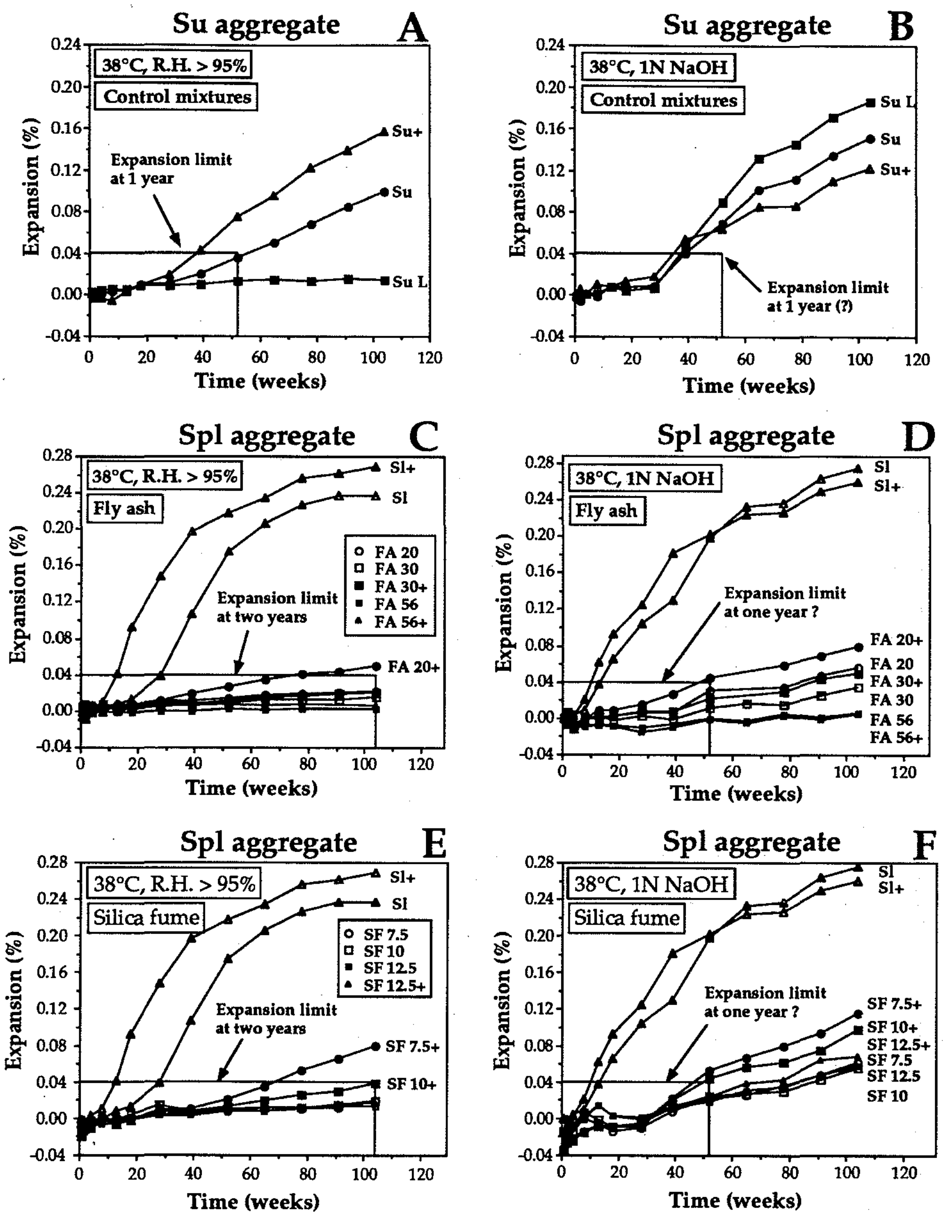

Fig. 2 Expansion curves for concrete prisms cast from concrete mixtures incorporating the Su (A&B) and Spl (C to F) aggregates.

A. Control mixture; Storage condition A: 38°C and R.H. > 95%
B. Control mixture; Storage condition B: 1N NaOH solution at 38°C
C. Fly ash mixture; Storage condition A: 38°C and R.H. > 95%
D. Fly ash mixture; Storage condition B: 1N NaOH solution at 38°C
E. Silica fume mixture; Storage condition A: 38°C and R.H. > 95%
F. Silica fume mixture; Storage condition B: 1N NaOH solution at 38°C

Table 3: Summary of the Laboratory test results obtained in this study

Expansion of controls in ≠ storage conditions (see below):	**Aggregate Types**				
	Limestone Lm	**Gravel Su**	**Sandstone Po**	**Limestone Sp**	**Greywacke Spl**
Concrete Prism (condition A):	0.087% (2 years)	0.157 (2 years)	0.226% (2 years)	0.171% (2 years)	0.269% (2 years)
Concrete Prism (condition B):	0.070 (1 year)	0.063 (1 year)	0.072 (1 year)	0.097 (1 year)	0.202 (1 year)
Concrete Prism (condition C):	0.124% (8 weeks)	0.160% (8 weeks)	0.281% (8 weeks)	0.201% (8 weeks)	0.190% (8 weeks)
Mortar Bar (condition D):	0.173 (14 days)	0.278 (14 days)	0.093 (14 days)	0.391 (14 days)	0.463 (14 days)

Cementitious Materials		Limestone Lm				Gravel Su				Sandstone Po				Limestone Sp				Greywacke Spl			
		Testing condition				Testing condition				Testing condition				Testing condition				Testing condition			
Type	%	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
Fly Ash	20					OK	OK	OK	OK					OK	OK	OK	•	•	•	OK	OK
FA 1: Po & Spl	30					OK	OK	OK	OK	OK	OK	OK		OK	OK	OK	OK	OK	OK	OK	OK
FA 2: Su & Sp	56					OK	OK	OK	OK					OK	OK	OK	OK	OK	OK	OK	OK
Slag	35					OK	OK	OK	•					•	•	•	•				
	50					OK	OK	OK	OK					OK	OK	OK	OK				
	65					OK	OK	OK	OK					OK	OK	OK	OK				
Silica Fume	7.5	•	OK	•	OK	OK	OK	OK	•	•	OK	•		•	•	•	•	•	•	•	•
	10	OK	OK	•	OK	OK	OK	OK	OK	•	OK	•		•	•	•	•	•	•	OK	•
	12.5	OK	OK	•	OK	OK	OK	OK	OK					OK	OK	OK	OK	OK	OK	OK	OK

Testing Condition and expansion limits:	
(A):	Concrete Prisms at 38°C, R.H. > 95%; Expansion limit of 0.04% at two years.
(B):	Concrete Prisms at 38°C, in 1N NaOH solution; Expansion limit of 0.04% at one year.
(C):	Concrete Prisms at 80°C, in 1N NaOH solution; Expansion limit of 0.04% at 8 weeks.
(D):	Mortar bars at 80°C, in 1N NaOH solution; Expansion limit of 0.10% at 14 days.

•: The combination tested failed to meet the expansion limit suggested for this test condition
OK: The combination tested met the expansion limit suggested for this test condition

ALKALI-SILICA REACTION IN THE NETHERLANDS

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ABSTRACT

In The Netherlands, the number of concrete structures showing damage caused by ASR has increased from 3 to about 35 over the last 5 years. In general, the concrete of these structures is composed of ordinary portland cement and local river-dredged gravel and sand. The reactive constituent is porous chert, which is present in the coarse aggregate in varying amounts (1-6% by mass). Secondary ettringite formation often accompanies the ASR.

Damage due to ASR has been found in two concrete structures, in which blast furnace slag cement with a low slag content of about 40% by mass, was used.

A recommendation has recently been published describing a procedure, which should be followed in designing concrete mixes, to prevent the occurrence of harmful ASR in concrete. A procedure with respect to the structural assessment of concrete structures affected by ASR is currently in preparation.

Keywords: alkali-silica reaction, blast furnace slag cement, ettringite, microscopy, recommendations

INTRODUCTION

The first case of deterioration as a result of ASR in a Dutch concrete structure was published in 1991 (Heijnen & Van der Vliet 1991; Heijnen 1992). Since then, about 35 cases of deterioration caused by ASR have been found in The Netherlands. Most structures affected by ASR are more than 30 years old. This suggests that the rate of ASR in Dutch concrete structures is relatively slow. Reasons for this slow reaction rate are to be found in the nature of the reactive components (coarse porous chert grains) of the aggregates, the relatively low content of potentially reactive aggregate particles, the generally low cement content of Dutch concrete and the generally low alkali content of Dutch portland cement.

Since the number of structures suffering from ASR is increasing, substantial effort is directed towards specifications and guidance for avoidance of damage due to ASR, as well as for evaluation of the structural consequences.

STRUCTURES AFFECTED

General

The structures that have been found or suspected to be affected by ASR include mostly viaducts, bridges and locks. The age of the structures involved is variable, but in general ranges from about 30 to 60 years. The damage is usually manifested in the form of map-cracking with the cracks sometimes filled with ASR-gel in combination with ettringite or other leached cement products.

Cement used

The cement used in all but two of the structures that have been diagnosed to be affected by ASR is ordinary portland cement. PFM (Polarising and Fluorescence Microscopy) analysis shows the grains of nearly all such cements to be coarse, with an estimated modal grain size of about 50 μm. To date, there has not been any case of concrete structure affected by ASR in which blast furnace slag cement (bfsc), containing more than 65% slag (by mass, as cement replacement) or composite cements of fly ash containing more than 25% fly ash (by mass, as cement replacement) was used.

In the two cases, namely a lock and an in-situ cast foundation block of a power pylon, blast furnace slag cement with a low slag content of 35-40% and 40-50% respectively was used (Fig. 1). In both cases also, most of the residual slag particles in the cement paste were found to be coarse-grained with an estimated modal grain size of about 100 μm; a significant number of the grains was found to fall in the 150-200 μm size range. The majority of the coarser grains (>100 μm) appeared not to have reacted at all. Reaction rims around such particles were clearly absent - they appeared virtually inert.

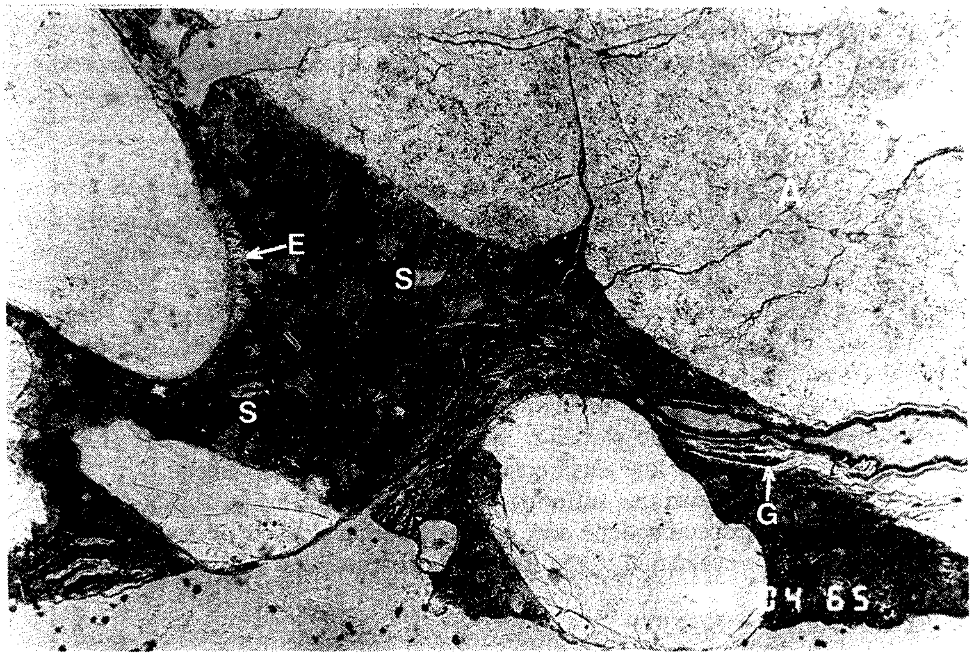

Fig. 1: Thin section micrograph showing alkali-silica gel streaming outward from a sericitic sandstone coarse aggregate particle through cracks into the adjoining cement paste. The concrete was prepared with portland blast furnace slag cement with about 40% by mass of slag. A = aggregate; S = slag particles; G = ASR-gel; E = ettringite. (plane polarised light; micrograph is 1.4 mm x 0.9 mm)

The extent of damage in the concrete, as diagnosed from the PFM analysis was in both cases found to be moderate to severe.

Blast furnace slag cements and composite cements of fly ash currently produced in The Netherlands contain more than 50% slag or more than 25% fly ash (by mass, as cement replacement) respectively. The modal size of the slag particles, determined from PFM analysis is 25-40 μm, with only a small percentage falling above the 100 μm size range. The alkali content (Na_2O-eq.) of Dutch cements varies between 0.6 and 0.8 % by mass. The modal size range of ordinary portland cement and composite cement containing fly ash currently produced in The Netherlands is 15-30 μm.

Concrete characteristics

The mixing of the concrete constituents and especially, the cement is very often found to be poor or at least not optimum, resulting in an inhomogeneous concrete. The packing and distribution of the aggregate particles in the cement paste is not the same throughout the concrete; very often there are zones rich in aggregate but low in cement content. This, however, does not apply to all concretes with an inhomogeneous microstructure. There have been a number of cases in which petrographic analysis of concrete specimens with inhomogeneous microstructure and suspected to be affected by ASR have shown no apparent signs of attack by ASR.

Cement paste characteristics

The cement paste is nearly always inhomogeneous; zones rich in water but poor in cement (zones with high water-cement ratios (w/c) are very often interconnected with each other from the surface of the concrete into the deeper parts) alternating with zones with low water-cement ratios are very common in nearly all the samples that have been examined. The water-cement ratios, determined with the aid of fluorescent microscopy vary considerably within a thin section specimen and in the concrete core as a whole. Typical values range between 0.40 and 0.70.

Degree of cement hydration

The degree of hydration of the cement (which is estimated from the amount of residual, unhydrated or partially hydrated C_3S and C_2S grains in the cement paste) is, in general, high. The occurrence of large clumps of coarse, virtually unhydrated C_3S and C_2S grains (signifying poor mixing of the cement) is quite common in most of the specimens. The modal grain size of the portland cement particles, estimated from the residual unhydrated cement and from reaction rims around the partially hydrated grains is about 50 μm. A considerable portion of the grains of such cements is usually found to fall within the 50-100 μm size range.

Amount of calcium hydroxide

The amount of calcium hydroxide, $Ca(OH)_2$ found in thin section specimens is very often considerably high. In some specimens, they occur as large crystals even at the paste-aggregate interface of particles that have already reacted indicating high levels of alkalinity of the pore fluid in the cement paste. Such high levels of alkalies are likely to originate from the cement used or in the case of bridges and viaducts, from a combination of the cement and de-icing salts. PFM and SEM analyses of specimens of concrete suffering from ASR very often do show a depletion of $Ca(OH)_2$ in the cement paste, especially at the paste-aggregate interfacial zones of the reacted particles (Regourd-Moranville 1989). In such cases, $Ca(OH)_2$ is believed to go into solution to buffer the alkalinity of the pore fluid after the alkali-silica reaction has taken place. The presence of large crystals of $Ca(OH)_2$ in the specimens indicates that the pore fluid of such concretes is still rich in alkalies and may have adequate potential for further reactivity under the appropriate conditions. $Ca(OH)_2$ in such cases does not seem to have contributed to the alkalinity of the pore fluid in the cement paste.

Alkali-silica gel

Manifestations of alkali-silica reaction usually includes the following:

a. cracked or fractured particles of chert, quartzite or sandstone, with the cracks occasionally filled with ASR-gel and radiating into the surrounding cement paste;
b. occurrence of reaction rims at the periphery of some of the affected aggregate particles;
c. simultaneous occurrence of fine, needle-like crystals of ettringite and ASR-gel in pores and cracks in the cement paste.

In thin section specimens, the ASR-gel is found to occur in one or more of the following locations in the concrete: confined to cracks within the reacted aggregate particle, filling cracks in the reacted aggregate and penetrating or streaming outward from the reacted aggregate through cracks into the adjoining cement paste or occurring solely or together with secondary ettringite in pores and cracks in the cement paste. The alkali-reactive particle or constituent in all cases examined so far, is one or more of the following: porous chert, sericitic quartzite, sericitic sandstone, quartzite containing microcrystalline quartz or chalcedony.

Simultaneous occurrence of fine needle-like and massive ettringite

In all cases, fine needle-like crystals of ettringite are found to occur simultaneously with ASR-gel in cracks and pores in the cement paste. This type of ettringite formation has not been observed in cracks within aggregate particles as yet.

The formation of massive ettringite from sulphate attack has been found to occur simultaneously with ASR in a few cases. The ettringite in such cases consists of compact crystals completely filling pores and large cracks (> 25 μm in width) at the interface of both coarse and fine aggregate particles (Fig. 2). In some cases, there is a complete 'enveloping' of the aggregate particles by the massive ettringite. Such cases are very often associated with intense microcracking and all the microcracks apparently radiating from the ettringite-filled cracks into the adjoining cement paste. The individual crystals of such compact ettringite are not easily distinguishable even at higher magnifications of 200x and 400x with the PFM. The occurrence of fine needle-like ettringite is not associated with deformation of the cement paste.

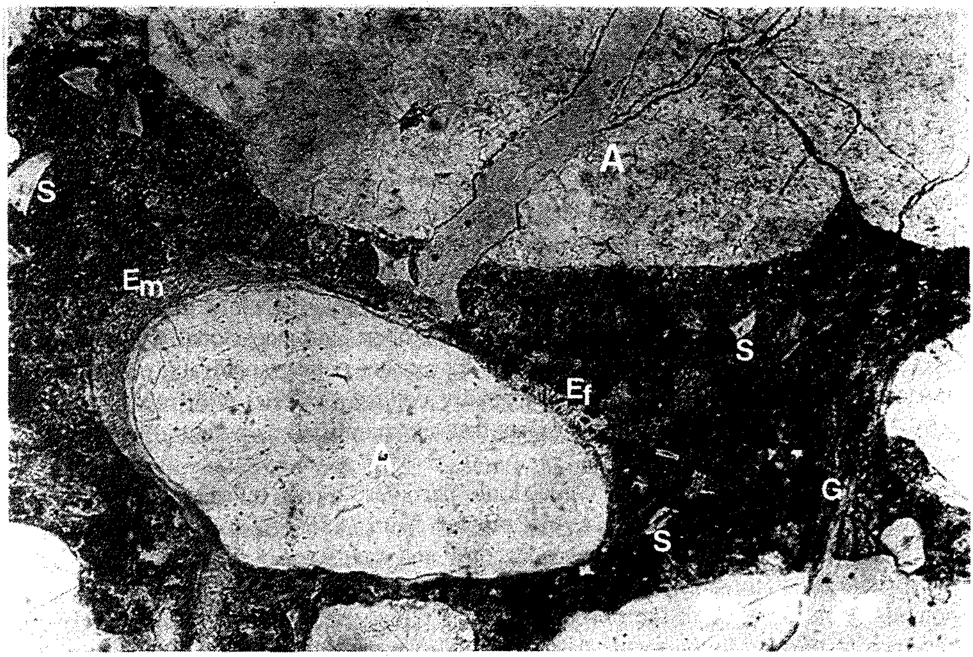

Fig. 2: Thin section micrograph showing alkali-silica gel streaming outward from a sericitic sandstone coarse aggregate particle through cracks into the adjoining cement paste. The concrete was prepared with portland blast furnace slag cement with about 40% by mass of slag. A = aggregate; S = slag particles; G = ASR-gel; Em = massive ettringite; Ef = fine needle-like ettringite. (plane polarised light; micrograph is 1.4 mm x 0.9 mm)

Zones of preferential occurrence of ASR and massive ettringite

In all cases studied so far, the occurrence of the ASR and the formation of massive ettringite seem to be confined to the zones in the cement paste with high water-cement ratios. Such observations are easily made with aid of fluorescent microscopy. In the zones with lower water-cement ratios, the integrity of the cement paste and the concrete is very much preserved. Pores and cracks in such zones are very often empty or contain only crystals of $Ca(OH)_2$.

CONCRETE AGGREGATES

General

Traditionally, the main aggregates used for production of concrete in The Netherlands have been river gravel and sand which are dredged from the rivers Maas and Rhine. The gravel is composed predominantly of particles of quartzite and sandstone and with small amounts of chalcedony, porous and non-porous chert. The sand is rich in quartz, but contains subordinate amounts of porous and non-porous chert, quartzites, sandstones, limestone, glauconite, feldspars and opaque minerals.

Most of the quartzites and sandstone particles are rich in mica and sericite. Crushed rocks such as limestones, granites, quartzites as well as sea-dredged sands and gravels are also used for concrete but in smaller amounts compared to river aggregates.

Reactive aggregates

Until recently, it was believed that only the porous chert and the chalcedonic particles, found mostly in the coarse aggregate fraction are potentially reactive. This assertion was made in 1992 after an inventory study of the structures that had been affected by ASR at that time. It appeared from investigations at that time that in all the ASR-affected structures only porous chert and occasionally chalcedonic particles were reactive. As such, to date, in routine petrographic analysis of river and sea-dredged aggregates (as part of quality control analysis) with regard to their ASR-reactivity, these are the constituents that are quantified as potentially reactive.

In addition to porous chert and chalcedonic particles, recent investigations with the aid of PFM on cores of concrete removed from ASR-affected structures have revealed that, constituents such as quartzites and sandstones, especially those rich in sericite and those containing micro- and cryptocrystalline quartz are also potentially reactive. Like the porous chert, all the reacted particles found so far are confined to the coarse aggregate fraction and are from river-source aggregates only.

Implications for quality control analysis of aggregates

In The Netherlands, CUR Recommendation 38 (CUR 1994) provides guidelines, for specifications and procedures for evaluating aggregates with regard to ASR. In that recommendation, concrete aggregates are distinguished on the basis of the mineralogical composition in relation to ASR into the following three categories:

a. aggregates with such a low amount of reactive components that no harmful ASR can occur, indicated as 'under critical'
b. aggregates with such a high amount of reactive components that no harmful ASR can occur, indicated as 'above critical'
c. aggregates with an amount of reactive components that harmful ASR can occur, indicated as 'critical'.

One of the means specified by the above recommendation for evaluating the potential reactivity of concrete aggregates is the mineralogical composition obtained from petrographic analysis. At TNO Building and Construction Research, the petrographic analysis is performed on routine basis with the aid of PFM in combination with point-counting analysis of the various fractions in the aggregate sample (sand, gravel or crushed rock).

For chert particles, a combination of the polarising and fluorescent microscopy is crucial in establishing which of the particles are potentially reactive in the aggregate samples submitted for investigation. By means of polarising microscopy, the various mineral constituents (including the porous and non-porous chert) are identified. The porosity of the chert particles are subsequently established with the aid of fluorescent microscopy. By means of point-counting, the porous chert and chalcedonic particles are then quantified in the various fractions examined and eventually the material as a whole. The material is then classified into one of the three groups above on the basis of the amount of reactive constituents in the sample examined.

The recent revelations made about the potential reactivity of quartzites and sandstones in the coarse fraction of the river aggregates call for re-examination of the present guidelines and criteria for evaluating the potential reactivity of river- and possibly sea-dredged aggregates. There may be the need to develop procedures to re-evaluate the sericitic quartzite and sandstone particles in the light of their potential reactivity with regard to ASR so that they could be adequately classified as potentially reactive or not. In order to be able to achieve this, the following set of questions may have to be carefully examined and possibly answered:

- are all sericitic quartzite and sandstone particles in the river-dredged aggregates potentially reactive or are these reactive aggregates associated to only one source or mining location?
- should sericitic quartzite and sandstone particles in sea-dredged aggregates be considered potentially reactive?
- do they occur in significant amounts in river aggregates to warrant extra examination?
- what should be the criteria for defining the critical limits?

In addition to the above questions, one other issue about the sericitic quartzite and sandstone particles that needs to be resolved is the type and nature of the constituent in the particles that is potentially reactive or causes the alkali-silica reaction. At the moment, the reactive constituent involved is strongly suspected to be opal or its constituents occurring possibly together with the sericitic components in the joint or fracture planes of the particles. Because of its amorphous character and the fact that it occurs in small amounts, it is hardly identifiable in thin sections with the aid of optical microscopy. In order to remedy this shortcoming in the PFM analysis of aggregates, extension of the microscopic analysis with the gel-pat test is under consideration.

MEASURES TO PREVENT DAMAGE DUE TO ASR

In The Netherlands, the susceptibility of aggregates to ASR is considered to be of relatively minor importance, because the concrete mix design and cement type can be chosen in such a way, that the risk of harmful ASR is negligible (CUR 1994). For the following cements, it can be assumed that a harmful ASR will not occur:

* portland blast furnace slag cement with more than 65% (by mass) slag and an alkali content less than 2.0% (by mass)
* portland blast furnace slag cement with more than 50% (by mass) slag and an alkali content less than 1.1% (by mass)
* portland fly ash cement with more than 25% (by mass) fly ash and an alkali content less than 1.1% (by mass)

More than 70% of all concrete in The Netherlands is produced with the cements mentioned above.

STRUCTURAL ASSESSMENT

The number of concrete structures suffering from ASR now is larger than was expected originally. So now it has been decided to establish a Dutch guidance for the structural assessment of concrete structures with damage due to ASR. The basis of this guidance will be formed by existing knowledge and information as present in the Dutch Concrete Codes and the ISE-guidance (Institution of Structural Engineers 1992). From pilot structural assessments, it will be verified whether this knowledge and information is sufficient. At this moment it is already clear that there are questions to be answered with respect to:

- the internal forces in a static independent structure; the magnitude of the internal forces depends also on the deformations of the structure and as a consequence also on the swelling of the concrete structure
- the buckling behaviour of columns; due to non-uniform swelling of the concrete the eccentricities of axial forces can increase dramatically
- the residual service life; for ordinary concrete structures the insight in service life is for a large part based on experience, for structures with damage due to ASR this insight is very restricted and must be improved.

The Dutch guidance for the structural assessment which is currently in preparation is scheduled to be ready and available by the beginning of 1998.

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PRESENCE OF ALKALI-SILICA REACTIVE AGGREGATES IN KOREA

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ABSTRACT

Forty different aggregates commonly used in ready-mixed concrete plants in Korea were tested for their alkali-silica reactivity by petrographic examination, chemical method and mortar bar method, respectively. Test results indicate that several aggregates be alkali-silica reactive. And alkali-silica reaction(ASR) products are also confirmed to be present in the cement composites of these aggregates by scanning electron microscope (SEM) and electron dispersive spectroscope(EDS).

It is found from this investigation conducted in the area, where the seemingly reactive aggregates had been used in producing ready-mixed concrete that several structures seem to have been deteriorated by ASR, indicating the presence of alkali-silica reactive aggregates in Korea.

Keywords: alkali-silica reaction, alkali-aggregate reaction, field investigation

INTRODUCTION

In Korea, mass construction started in the late 1960's with the rapid economic development. Numerous concrete structures have been constructed during the past thirty years. The annual cement consumption in the 1990's exceeded 1,000kg per person. Nevertheless, the deterioration of concrete structures caused by alkali-aggregate reaction(AAR) has not been reported in Korea, because alkali content of cement is not high enough and crushed aggregates had not been used until 10 years ago.

The natural river aggregate was exhausted in the latter of the 1980's in Korea. And 200 million tons of aggregates had been consumed due to the rushed construction demand for housings and infra-structures, use of crushed aggregates was legally permitted. Therefore, the risk of AAR has been increasing in Korea due to the rapid increase in crushed aggregates consumption. Figure 1 shows a pattern of several aggregates consumption to be expected in Korea until 1998.

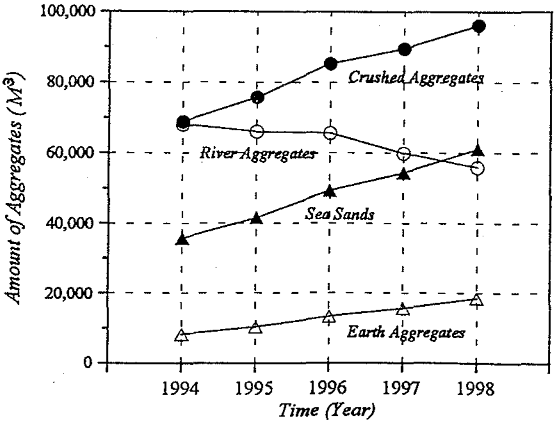

Fig. 1 Aggregates consumption in Korea

Forty samples of crushed stone being used at the ready-mixed concrete plants in Korea(*see Fig.* 2) were tested by petrological method using X-ray diffractometer (XRD), SEM and polarized light microscope, chemical method and mortar bar method to identify whether they are alkali-silica reactive or not.

From these results, several aggregates are found to be alkali-silica reactive. Cracks of concrete structures were discovered from investigation in the area where deleterious crushed stones were used as ready-mixed concrete aggregates.

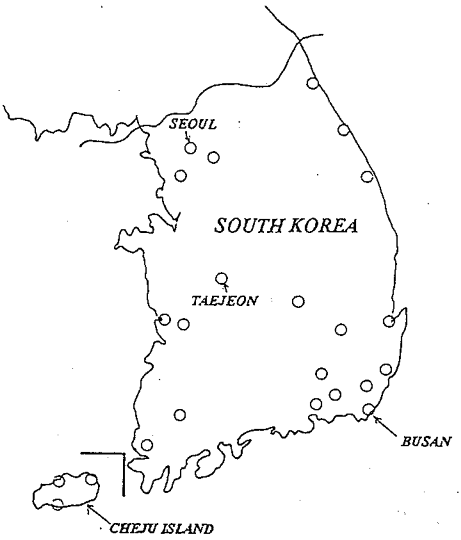

Fig. 2 Location of sampling aggregates in Korea

EXPERIMENTS FOR ALKALI-SILICA REACTIVE AGGREGATES

Forty crushed aggregates in this study were used in the ready-mixed concrete plants in Korea. The location of sampling aggregates are shown in *Fig.* 2.

The petrological properties of aggregates were investigated by chemical analysis, x-ray diffractometer(XRD), and polarized light microscope. Then, alkali-silica reactivity of aggregates was tested by chemical method(ASTM C 289).

Mortar bar method(ASTM C 227) was performed with the aggregates identified as deleterious in chemical method.

SEM and EDS were used to observe the microstructure of expanded mortar bars.

Mortar bar tests were then performed with various types of alkalies, contents and alkali-silica reactive aggregates. Cement was replaced with pozzolanic materials to test the effects to reduce the mortar bar expansion.

Petrographic examination

From the results of chemical and XRD analyses(*see Fig.* 3a), the major ingredients are found to be quartz and feldspar, and the minor ones are muscovite, olivin, and etc. for the most part of aggregates.

However, several volcanic aggregates have alkali-silica reactive, high temperature phased cristobalite (*see Fig.* 3b). They may have some amorphous glass because of the low peak intensity of these aggregates.

Fineiy grained silica crystallines were observed by the polarized light microscope analysis in these volcanic aggregates (*see Photo* 1) that are identified as alkali- silica reactive aggregates in the future results of chemical method and mortar method.

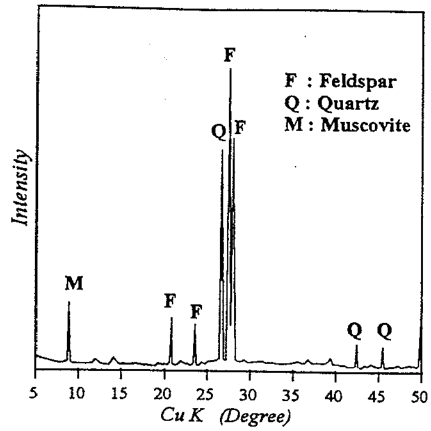

Fig. 3a XRD pattern of granite aggregate

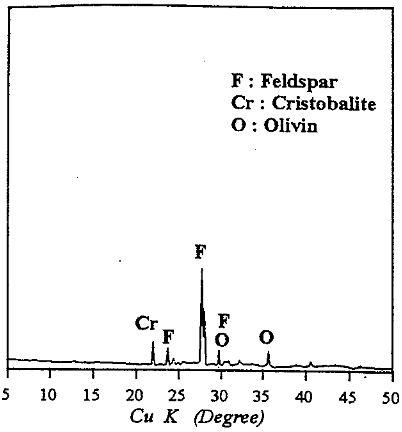

Fig. 3b XRD pattern of volcanic aggregate

Photo 1a Microscopic image at open nicol in thin slide of volcanic aggregate

Photo 1b Microscopic image at crossed nicol in thin slide of volcanic aggregate

Chemical method

Crushed, washed and dried to 0.15~0.30mm aggregate particles were reacted with 1N NaOH solution at 80°C for 24hours to get the reduction in alkalinity and dissolved silica content as in ASTM C 289.

As shown in the *Fig.* 4, most part of aggregates is innocuous, however, several aggregates of Cheju island are deleterious, or potentially deleterious to be alkali-silica reactive.

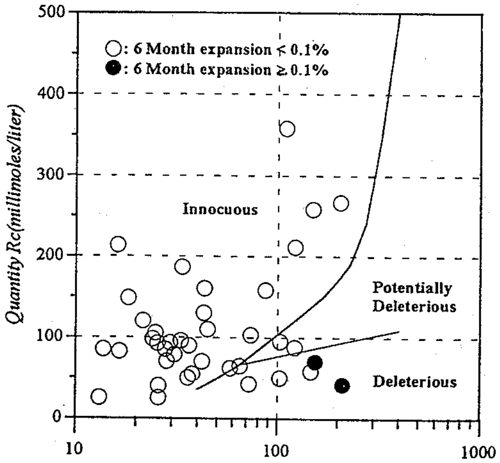

Fig. 4 Chemical method result

Mortar bar method

Mortar bar was molded at W/C 50%, and stored at 40°C, RH 95% according to ASTM C 227. Total alkali content of cement used in mortar bar method was controlled to 0.8%, 1.2%, 1.6%, 2.0% by adding NaOH, KOH, NaCl into cement with 0.72% alkali.

The result of mortar bar test was compared with chemical test in *Fig.* 5. Innocuous aggregates in mortar bar method were innocuous in chemical method. In some cases, deleterious in chemical method but innocuous in mortar bar method. The expansion of mortar bars with deleterious aggregates is over 0.1% in 6 months at 0.8% total alkali in cement. The more alkali content, the more expansion of mortar bar.

Fig. 6 shows the expansion of mortar bars adding NaOH, KOH and NaCl into cement. With equal alkali content, the mortar bar with NaCl added cement has large expansion than the mortar bar with NaOH or KOH added cement [1, 2].

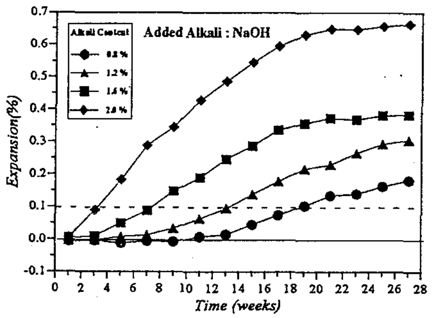

Fig. 5 Mortar bar expansions at various alkali content with a Cheju aggregate

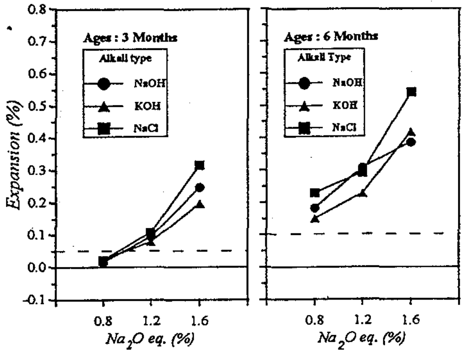

Fig. 6 Mortar bar expansions at various alkalies with a Cheju aggregate

Scanning Electron Microscopy

SEM and EDS were used to observe the microstructure of reaction products of a mortar bar expanded over 0.1% in 6 months. Photo 2a shows massive gel of primary reaction product and photo 2b shows spongy gel of secondary reaction product which is natrium and potassium rich silicates material in the dried mortar bar with alkali-silica reactive aggregate [3,4].

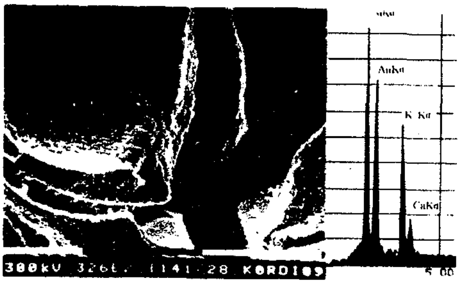

Photo 2a SEM and EDS of massive gel

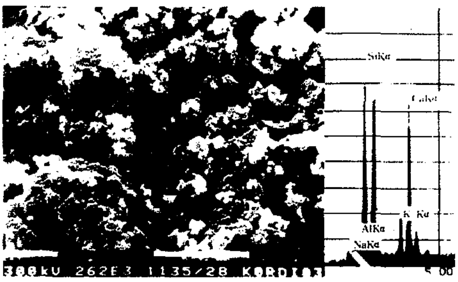

Photo 2b SEM and EDS of spongy gel

EFFECTS OF POZZOLANAS ON ASR

As shown in *Fig.* 7, the expansions of mortar bars with slag cement and fly-ash added cement were less than with type 1 portland cement at equal alkali contents. Therefore, slag cement and fly-ash added cement can prevent the damages by ASR in the case of using crushed stone in Cheju island.

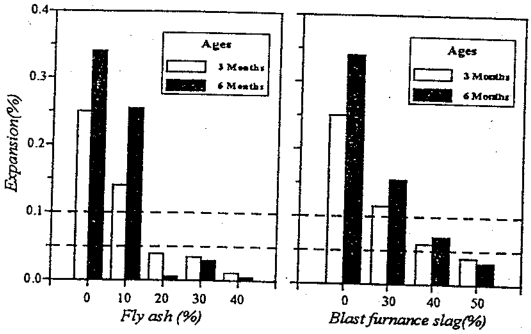

Fig. 7 Expansion of pozzolanas added mortar bar

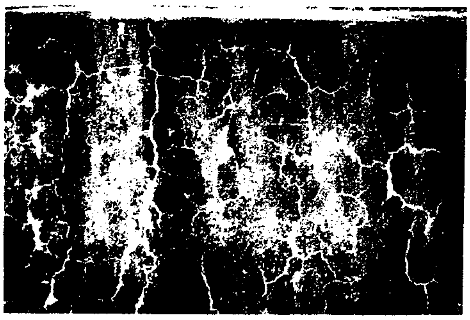

Photo 3a Damages of a concrete by AAR on the wall of a building in Cheju university

Photo 3b Damages of a concrete by AAR on tetraport in Cheju island

FIELD INVESTIGATION

Field investigation was performed on the concrete structures in Cheju island where alkali-silica reactive volcanic rocks have been used as aggregates for ready-mixed concrete. Crushed coarse aggregates are supplied by 10 makers being located in west, northwest, east and northeast in Cheju area. All fine aggregates are water-washed sea sand.

As shown in *Photo* 3, concrete cracks which is thought to be due to ASR were found in university buildings, hospitals, bridges, and offshore structures. There are not so old concrete structures enough for the occurrence by ASR in Cheju island. Because of the small amount of rainfall and the porous volcanic ground let the water flows under ground level, ASR in field concrete structures may proceed slowly.

However, only alkali-silica reactive aggregates and sea-sand are available as ready mixed concrete aggregates in Cheju island. Therefore, there can be more damages by ASR in the future.

CONCLUSION

There are numerous reports on the damage of concrete structures caused by using alkali-silica reactive aggregates in many parts of the world after the report of T. E. Stanton. However, the presence of alkali-silica reactive aggregates has not been yet reported in Korea.

Forty different aggregates commonly used in the ready-mixed concrete plants in Korea were tested for their alkali-silica reactivity by petrographic examination, chemical bar and mortar bar method, respectively. Test results indicate that several aggregates be alkali-silica reactive. ASR products were observed in the cement composites of these aggregates by using SEM and EDS.

Concrete structures in this area, where these seemingly reactive aggregates have been used in producing ready-mixed concrete, were thus investigated. It is found in this investigation that several structures seem to have been deteriorated by ASR, indicating the presence of alkali-silica reactive aggregates in Korea.

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CASE STUDIES OF THE PRACTICAL AND ECONOMICAL IMPACT OF ALKALI-SILICA REACTION IN SOUTH AFRICA

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ABSTRACT

Alkali-silica reaction has affected the serviceability and durability of a significant number of concrete structures in South Africa, especially in the South Western Cape where the current worth of structures affected is estimated to be more than R1 600 million (R1 = US$0,2743). Several million rand has already been spent on rehabilitating some of the affected structures in South Africa and some of those in the South Western Cape are discussed as case studies in this paper. Indications are that the cost of rehabilitating and partially replacing the structures that are discussed, could be between R50- and R100 million.

Keywords: Alkali-silica reaction, repair and rehabilitation, economical impact.

INTRODUCTION

In South Africa, ASR was first identified in the 1970s (Oberholster & Brandt, 1974) as the reason for the expansion and cracking of concrete structures in the Cape Peninsula. Currently, enough is known of the phenomenon to minimise the risk of ASR in a new structure. The purpose of this paper is to review the impact of ASR on the management and rehabilitation of affected structures and some of the consequences of measures taken to minimise the risk of its occurrence.

DETERMINATION OF THE WORTH OF AFFECTED STRUCTURES

The incidence of ASR is highest in the South Western Cape. Although widespread, it occurs on a smaller scale in the Eastern Cape and Gauteng. During the investigation of the problem in bridges in the South Western Cape, it was found to be totally impossible to assess the cost of damage in terms of the replacement cost of the structures or parts thereof. The only approximation that could be made was the *present worth* of the structures which was calculated as follows:

$$\text{Present Worth Factor} = \frac{\text{Construction Price Index for 1977}}{\text{Construction Price Index for Particular Year}}$$

$$\text{Construction Price Index} = 0.66(\text{Building Materials Index}) + 0.33(\text{Consumer Price Index})$$

Taking into account only those bridges and the 27 km concrete pavement that were affected by the reaction, the *present worth* estimate of the investment was R37,3 million in 1977 (Semmelink, 1981). Taking an average annual escalation of 15 %, this would amount to about R530 million in 1996. If other structures which have subsequently been identified as being affected by ASR are included, the figure could conservatively exceed R2 billion. However, the real significance of ASR can only be assessed if one analyses the situation for specific structures.

REHABILITATION AND MANAGEMENT OF AFFECTED STRUCTURES

In a number of instances, structures affected by ASR had to be demolished or parts replaced, because they had become unserviceable or dangerous. For example, part of the upper beam of a reinforced double-storey portal frame of the M1 motorway in Johannesburg, constructed in 1966 (Alexander et al. 1992) was demolished and reconstructed in 1991 at a cost of R1 million. However, where problems with the serviceability and durability of concrete structures are encountered, the aim should first be to carry out a proper condition survey to establish the extent and cause of the problem as outlined in a number of publications (ACI, 1990; British Cement Association, 1992). Once ASR has been identified in a structure, a future course of action should be established as described, for example, in the publication of The Institution of Structural Engineers (1992) or in Fulton's Concrete Technology (Oberholster, 1994). Depending on the extent and severity of ASR, this could involve monitoring only of the structure at intervals, repair, or repair and strengthening, in each case combined with regular monitoring. In extreme cases, demolition of the structure may be necessary.

The following examples describe the procedures that have been followed and the economical and practical implications, for a number of different structures in the South Western Cape.

The N2/1 concrete freeway between Cape Town and Somerset West

In 1970, approximately 27 km of National Route N2/1, between Cape Town and Somerset West were realigned and reconstructed as a dual-carriage freeway. The carriageway consists of a continuous unreinforced concrete slab 7,3 m wide and 200 mm thick placed with a slipform paver in a single lift. It has a longitudinal joint along the centre of the slab formed by a polythene tape insert (50 mm x 0,1 mm) and the two halves are tied together with high tensile dowel bars 600 mm long and 16 mm in diameter at 750 mm intervals. Skewed transverse joints at 80° to the longitudinal axis were formed by sawing to a depth of 50 mm with a diamond saw. Joints were sealed with a preformed neoprene seal. Malmesbury Group metasediment was used as coarse aggregate.

By 1975, an unusually high incidence of hairline cracking was observed in the pavement. It was established that this was due to ASR. The action of traffic aggravated the cracking so that by 1979 a number of failures at transverse joints had occurred. These failures took the form of spalling at the joints and required patching. It was, however, apparent that the pavement might fail progressively under traffic. A programme was commenced to: (i) evaluate the combined effects of ASR and traffic; (ii) determine the potential life of the pavement (Freeme & Shackel, 1981); (iii) evaluate all the experimental data on the pavement; and (iv) based on the findings, propose maintenance and remedial procedures and draw up proper contract documents (Van der Walt et al. 1981).

The first phase of the investigation involved (i) heavy vehicle simulator (HVS) testing on two selected sites; (ii) recording and quantifying the development of surface cracking; (iii) measuring the structural behaviour in terms of deflection and deformation; and (iv) rehabilitation analysis. It was concluded that all of the pavement was potentially liable to fail under traffic regardless of whether ASR progressed or abated and that the estimated remaining life of the pavement was between two and five years for the most heavily trafficked sections. Also, the construction of an experimental section, employing a number of various rehabilitation alternatives, would be essential to arrive at the ultimate solution.

In the second phase, therefore, an experimental section of overlays was constructed using three different types of materials, namely asphalt, crushed stone with an asphalt overlay and OPC concrete. The asphalt overlays were built at three different thicknesses using a semi-gapgraded asphalt with precoated chips at the surface. Two types of rigid overlays were constructed, namely, a jointed and a continuously reinforced concrete pavement. Performance evaluation of the different overlays was expedited by using an HVS.

Based on the results of the evaluations, rehabilitation of 22 km of the freeway was commenced in 1986 and was done in the following stages:

- Repair of the joints by breaking out the concrete and replacing with a 40 MPa Portland fly ash cement concrete.
- Installation of 4 300 cylindrical no-fines concrete shoulder drains 150 mm in diameter in the asphalt shoulder to drain water collecting on top of the clay working platform under the concrete pavement, thus retarding further ASR.
- Cleaning cracks and joints and sealing with a bitumen-rubber sealant.
- Applying a 40-mm bitumen-rubber overlay with a single seal bitumen-rubber stress absorbing interlayer between the concrete and the overlay.

The rehabilitation of an additional 3 km was carried out separately as follows:

- 1981, cracks, transverse joints where neoprene sealers had been pressed below the surface and longitudinal joints between the concrete and the premix, cleaned with compressed air and sealed with pli-astic. Spalls patched with premix. Surface sealed with a 13-mm and 7-mm double seal.
- 1991, repairing the joints by breaking out the concrete and replacing with a 40 MPa concrete and applying a 13-mm bitumen-rubber single seal.
- 1994, applying a 40-mm bitumen-rubber asphalt overlay, continuously graded with 19-mm maximum aggregate size.

The costs of the different stages of the freeway project are as follows:

• 1970: Construction cost of pavement and ancillaries	R 2 x 10^6
• 1979: HVS investigations and rehabilitation analysis	R 1.8 x 10^6
• 1983: Experimental overlays and HVS investigations, about	R 2.3 x 10^6
• 1985: Repairs to 22 km of existing road and bitumen single seal	R 2.3 x 10^6
• 1986: Bitumen-rubber asphalt overlay	R 6 x 10^6
• 1981: Repairs to 3 km (costs not available)	-
• 1991: Joint repairs and 13 mm bitumen-rubber single seal to 3 km	R 2 x 10^6
• 1994: 40-mm bitumen-rubber asphalt overlay to 3 km	R 2.4 x 10^6
Total estimated cost	R16.8 x 10^6

Concrete railway sleepers in the Sishen-Saldanha line

The 861-km long Sishen-Saldanha railway line was constructed between 1973 and 1976 to connect the iron ore deposits being mined at Sishen in the Northern Cape with the nearest suitable export harbour at Saldanha on the West Coast. Approximately 1.7 million heavy duty FY prestressed concrete sleepers were manufactured for this purpose, of which approximately 1.4 million were used in the construction of the line. The remainder was stockpiled at Saldanha for maintenance purposes.

The sleepers were manufactured at Saldanha by the longline process, using local granite aggregate and RHPC from the Riebeeck Plant of PPC. Maintenance staff reported serious longitudinal cracking of the sleepers in 1985 in the line (Oberholster et al. 1992). Inspection revealed that exposed sleepers in the stockpiles were also cracked. Investigations following on the discovery of the cracking of the sleepers included:

- Laboratory investigation to establish the cause of the cracking.
- Tests on cores to establish the potential for continued expansion of the sleepers.
- Long-term tests on two halves of a cracked sleeper from the line, to compare the expansion of one untreated half to that of the other half treated with a silane.
- Spot checks of 100 sleepers per km over the entire length of the line between 1988 and 1989 to determine the extent of the problem.
- Measurements on specific sleepers every three months to determine the rate of increase in crack width.
- Dynamic load tests on uncracked sleepers and sleepers from the line with different degrees of cracking, to develop an evaluation model in terms of the degree of ASR. This model was used to predict the service life and draw up a programme for repair and replacement of the sleepers.
- A detailed survey over the total distance of the line to determine the percentage of sleepers in each category.

The detailed survey revealed that approximately 70 % of the sleepers in the line show symptoms of ASR varying from slight to severe. The rehabilitation consists of treating reparable sleepers with silane. Since about 30 trains per week with an average length of 210 wagons (2.5 km) use the line and there are crossing loops only every 85 km, sleepers cannot be removed from the line for treatment, but individual sleepers have to be opened up without disturbing the geometry, classified and treated.

The relevant costs for this project are as follows:

• Original cost of 1.4 million sleepers in line	R 8.4 x 10^6
• Investigations, replacement and rehabilitation of sleepers since 1987	R 0.5 x 10^6
• Estimated rehabilitation costs for next 10 years	R17 x 10^6
• Estimated replacement costs for next 10 years	R17 x 10^6
Total estimated cost	R34.5 x 10^6

The estimates are in 1994 Rand value. A sleeper currently costs about R115.

The Good Hope Centre, Cape Town

The Good Hope Centre was constructed between 1976 and 1977. It is the largest cross-vaulted concrete structure in the world. The roof structure is 80 m by 80 m on plan and 20 m high. It is made up of 900 triangular, precast, gunited forms supported by in-situ ribs which in turn are supported by two diagonal, reinforced concrete arches each spanning 113 m and four external, terminal, reinforced concrete arches each spanning 80 m. The arch beams come together at each corner of the structure and are connected by means of exposed in-situ reinforced concrete buttresses. The horizontal forces generated by the arch beams are resisted by four prestressed concrete perimeter beams, each tie beam able to resist a force of 2 000 tons by means of high tensile prestressing cable strands. A 75-mm thick layer of sand completely separates the structure from the foundations to allow for initial shortening of the tie beams due to prestressing, temperature variations and creep of the concrete. The four pile caps at the corners of the structure each support a vertical load of 5 000 tons and each pile cap consists of five ton large diameter piles, founded and keyed into hard Malmesbury Group rock at depths varying from 6 m to 20 m. For the concrete of the four buttresses a granite aggregate and a cement content of 395 kg/m^3 were used. For the concrete of the arches a Malmesbury metasediment coarse aggregate was used. The eastern terminal arch was cast with a cement content of 428 kg/m^3 and the three remaining terminal arches and the diagonal arches of concrete with a cement content of 371 kg/m^3. The cement was from the Riebeeck Plant of PPC and, according to statistics, the average Na_2O equivalent of the cement from this factory over the period of construction varied between 0.8 % and 0.9 % (Semmelink, 1981).

Slight cracking of all four buttresses and terminal arches was reported in 1981. The cracking of the buttresses containing granite coarse aggregate, was regarded as unusual because at that stage ASR had only been identified in structures containing Malmesbury metasediment as aggregate. In 1993, an assessment of the extent and seriousness of ASR in the structure was commenced to decide on remedial measures and also to institute a management programme for monitoring and controlling the effects of ASR in the future.

An extensive condition survey of the buttresses and arches was carried out and the crack widths and distribution mapped in detail on a plan of the structure. Cores from the concrete were examined petrographically for the presence of ASR. The estimated expansion of the concrete was calculated and a structural severity rating assigned to the different elements. Measuring points were installed and the dimensional changes monitored with a Demec strain gauge.

The cost statistics are as follows:

• Original cost of construction	R 12 x 10^6
• Condition survey, consultation, laboratory investigations, monitoring	R 38 x 10^3
• Silane/siloxane treatment of buttresses, epoxy injection of cracks, sealing of surface; silane/siloxane treatment of terminal arches, epoxy injection where required; monitoring and consultation, 1994/95	R250 x 10^3
• Future monitoring at 6-month intervals, per annum	R 10 x 10^3
Total estimated cost	R298 x 10^3

The Steenbras Pumped-Storage Power Station

The Steenbras Power Station was commissioned in 1979. In 1984 it was established that the cracking of the concrete of the power station buildings, machine foundations and ancillary works was due to ASR. Monitoring of the concrete for dimensional, humidity and temperature changes was carried out between 1985 and 1990. In 1987 the Cape Town City Electrical Engineer's Department set up a monitoring programme aimed at determining the movement of two machines in Shaft B by means of a precision distometer with invar wires and an electronic clinometer. By September 1992 the measuring grid for the distometer clinometers was redesigned and implemented for Shaft A. This gave more detailed information with regard to the effects of ASR causing movement between the turbine top cover and generator lower bracket of the machines in this shaft. The B-shaft system was upgraded to the same configuration in the following year.

The results of the readings taken by the different authorities are briefly as follows:

- The concrete of the machine foundations, cooling water basement and anchor block is expanding and a total expansion of 0.550% and an expansion rate of 0.129% per annum have been recorded for the turbine floor in shaft B. The relative humidity in the concrete ranged between 71% and 100% at the different points.
- The measurements of the concrete movement over a period of 4.7 years have shown that: (i) Expansion of the concrete is causing the entire machine support structure to rise. This rise is restrained by the rock in contact with the shaft lining and is, therefore, greatest towards the centre of the shafts. The differential effect results in a tilting of the various floors and the machines themselves. (ii) Above the turbine floor (which is in the drier part of the support structure) a total vertical expansion of 1.5 mm has been recorded between levels 53.65 m and 66.12 m, which represents 27 microstrain per annum. (iii) There appears to be a local upward bulging of the turbine floor around the machines. (iv) Distometer measurements indicate that the entire shaft B is expanding and compressing against the surrounding rock. The rate of expansion seems to be decreasing, probably as a result of increasing resistance from the rock. (v) Expansion of the concrete is heterogeneous, i.e. at some points little or no movement is taking place, at others expansion is continuing at a steady rate and at others shrinkage is occurring.
- Measurement of machine movements carried out over a period of only 340 days since the new measuring grid was installed showed that the average machine movements correspond with the apparent movement of the concrete. There is a significant seasonal thermal movement of up to three times the apparent permanent effect.

The practical impact of the ASR in the concrete is misalignment of the four machines, each consisting of a reversible pump/turbine and motor/generator set rated at 45 MW. Consequently, these machines have to be realigned at intervals of approximately 5 years. An outage expressly for realignment purposes would take up to 8 weeks. The potential economic impact could be up to R2 million per machine, but in reality is more likely to be in the region of R1 million due to alternative loading strategies. Loss of machine availability over the sixteen year life of the power station to date would be approximately five to six machine years out of a possible sixty. This represents about 10 % unavailability due to direct and indirect consequences of ASR. During these outage periods where extensive repairs and refurbishment take place (which is not related to ASR) shaft realignment is checked and corrected as required. These outage periods could range from six to eight months, of which less than a month would be due to realignment as a consequence of ASR. In this way the economic impact of machine outages due to ASR is reduced to less than R1 million per machine per five year cycle, or R3- to R4 million for the station over a five year cycle. This represents less than 20 % of lost revenue due to machine down time for planned maintenance. The extra cost incurred due to extra labour and material is less than 10% of the total repair costs.

The approximate relevant cost statistics are summarised below:

• Original cost of Power Station	R 75 x 10^6
• Diagnosis of ASR, monitoring	R 10 x 10^3
• Consultation, installation of monitoring equipment, measurements	R130 x 10^3
• Cost of measurement systems	R200 x 10^3
• Loss of revenue due to ASR over 15-year period	R 12 x 10^6
• Cost of realignment/refurbishment due to ASR over 15-year period	R 12 x 10^6
Total estimated cost over 15-year period	R 24,3x 10^6

Dams

Reliable information of the cost implications of ASR in dams is not available. However, a brief overview of ASR and its effects in eight affected dams in South Africa has been given by Seddon and van den Berg, 1990. Subsequently ASR has also been diagnosed in the Ceres Dam.

Where cracking in dams has occurred, detailed inspections have been carried out, cores drilled for laboratory investigation to determine whether ASR is present and the stability re-evaluated. In several dams, cores have been extracted from the crest down into the foundations and also through the wall. At most of the dams where ASR has been identified, monitoring of total vertical strain has been initiated by precise levelling of points on the crest and/or sliding micrometers in boreholes.

The Churchill Dam is the only South African dam where major remedial work has been carried out. This multiple arch dam was built in 1944 and extensive, mainly horizontal cracking of the arches and buttresses was first observed in 1957. Some of the horizontal cracks penetrated right through the buttress at the upstream side. Initial investigations concluded that the cracking was caused by shrinkage of aggregates and thermal effects and that the cracking impaired the safety of the structure (Stutterheim et al. 1967). Consequently, as an immediate measure, the reservoir water level was restricted. Remedial action commenced and the buttresses were strengthened by the addition of reinforced concrete (colcrete) extensions on the downstream side. It was established in 1987 that the deterioration of the dam was in fact caused by ASR. Upward expansion of the arches at a greater rate than the buttresses caused the partial lifting and cracking of the upstream end of the buttresses.

MEASURES TO MINIMISE OR PREVENT THE RISK OF ASR

It is now well established that the risk of ASR in a new structure can be minimised or eliminated by taking appropriate measures such as using a non-reactive aggregate. Where an alkali-reactive aggregate has to be used, the alkali content of the concrete must be limited or the use of an approved cement extender should be considered (Oberholster, 1994). However, each of these might have some technical or economic implications.

Technical and economic implications of producing a low-alkali cement

All ordinary and rapid-hardening portland cements (OPC and RHPC) currently produced in South Africa are low-alkali cements as defined by SABS Standard Specification 471, although control of the alkali content is necessary for only a relatively small proportion of portland cement applications. For example, it is estimated that in the South Western Cape, only 10 % to 15 % of all cement produced in this region is used in concrete which could be susceptible to ASR. Controlling the alkali content of a cement at a specific limit, has certain technical and cost implications for a cement manufacturer, some of which are discussed below, using the PPC De Hoek plant as an example (Waterson & Harding, 1990). The measures applied at this factory to control the alkali levels, and the cost implications, might be substantially different from other cement plants in South Africa.

Screening levels and raw mix design

To produce a cement with an Na_2O equivalent of $\leq$ 0.55%, about 20% of the limestone (the fine, high-alkali portion) has to be screened out and discarded. Should a cement with an Na_2O equivalent of, for example, 0.8% be allowed, the screened rejects would be reduced to about 7.5% and this results in:

- Extension of limestone reserves by at least 14%.
- An annual saving in production costs of R1 million.
- Greater flexibility in raw mix design and savings on burning and milling costs.

Cement strengths

Dropping the alkali content of a cement results in a decrease in the early strengths but an increase in the 28-day strength. This effect of lower early strengths might become more pronounced with fly ash (FA) and Ground Granulated Blastfurnace Slag (GGBS) blends.

Cost of cement and concrete

Currently OPC and RHPC with an Na_2O equivalent of about 0.55% and a low-alkali sulphate-resisting portland cement (LASRC) with a guaranteed low Na_2O equivalent of 0.3% are being sold in the South Western Cape. For large, exposed concrete structures in which aggregate of the Malmesbury Group is used, it is usually required that a cement with an alkali content of less than 0.50% Na_2O equivalent, i.e. the LASRC, be used. It is calculated that the price differential between using the low-alkali OPC and the LASRC, would be about 9 %/m^3, in favour of the former for a 40 MPa concrete. The price difference between using a low-alkali OPC and an OPC with an Na_2O equivalent of 0.8 % in a 40 MPa concrete, could be of the same order in favour of the high-alkali cement. Currently, an RHPC with an Na_2O equivalent below 0.55 % is not available in the South Western Cape. This is a matter for concern in the precast concrete industry (where a high cement content is used as a rule) if Malmesbury Group metasediment aggregate is used.

An alternative measure for minimising the risk of ASR with an alkali-reactive aggregate, is to substitute a portion of a high-alkali cement with an extender, for example a minimum of 20 % FA or 40 % GGBS. Unfortunately, both of these industrial by-products are generated in areas where transport would add substantially to the delivered price in for example, the Western Cape. For example, a 40 MPa concrete in Gauteng costs about 4 %/m^3 less when a 50 % GGBS blend is used compared with 100 % OPC, while in Cape Town, the GGBS blend would cost about 4 %/m^3 more.

CONCLUSIONS

While enough is now known about ASR in South Africa to minimise the risk of its occurrence in a new structure, the implications of taking such precautions are significant in respect of the higher cost of concrete and the reduced life of limestone quarries.

The diagnosis of ASR in concrete, the assessment of the structural severity, the recommendation of remedial measures and the drawing up of a programme of maintenance and monitoring, require specialised and multidisciplinary skills.

In most instances where ASR has been diagnosed in structures, the durability and serviceability have been greatly affected and the service life in many cases significantly reduced. Public authorities and private owners of such structures are hard-pressed for funds for the high costs of reparation and maintenance.

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PRESENT EXPERIENCE WITH AGGREGATE TESTING IN NORWAY

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ABSTRACT

In Norway concrete aggregates are tested for possible alkali reactivity by the use of two successive test methods, namely a petrographic analysis (Norwegian method) and the South African accelerated mortar bar test (NBRI). The Canadian concrete prism test (CSA CAN 3-A23.2-14A) was used as test method up to 1993 but is no longer recommended for testing of concrete aggregates.

The paper describes the Norwegian petrographic method which correlates with the NBRI mortar bar test. So far, testing of Norwegian aggregates shows poor correlation between the CAN 3 concrete prism test and the petrographic analysis, when aggregates are dominated by sedimentary rock types, and between the CAN 3 concrete prism test and the NBRI mortar bar test.

It can be concluded that the petrographic analysis followed by the NBRI method is presently the best "tool" to assess alkali reactive Norwegian slow/late expansive aggregates. So far results show acceptable correlation between the two methods. Test results suggest that the Norwegian limits of 20 volume% reactive aggregates and the NBRI 14 day expansion of 0.10% are probably not conservative for Norwegian natural aggregates.

Keywords: Aggregates, Alkali Aggregate Reaction, Petrographic analysis, expansion tests

INTRODUCTION

Since the late 1980s alkali-aggregate reaction (AAR) has been recognised as a concrete durability problem in Norway. AAR in Norwegian structures is caused by the coarse aggregates derived from slow/late-expansive rocks. Diagenetic to low grade metamorphosed rhyolite, sandstones, siltstone, argillite (some carbonaceous), greywacke, and phyllite have reacted. More questioned AAR has been caused by granite, gneiss and hornfels. Cataclastic rocks e.g. cataclasite and mylonite are the most widely distributed reactive aggregate type in Norway. Common for Norwegian reactive aggregates are microcrystalline grain sizes e.g. due to cataclasis, developments of sub grains and recrystallization of quartz. The minerals *quartz-feldspar-muscovite (sericite)* are most frequently found in reacted Norwegian aggregates (Jensen 1990), (Jensen & Danielsen 1992), (Jensen 1993)

During the period 1990-1993 several laboratory test methods were investigated by SINTEF to find suitable test methods for Norwegian aggregates. This were the ASTM C 227 mortar bar test, the Danish accelerated mortar bar test TI-B 51, the Danish chemical shrinkage TK 84 test and the Japanese Fresh Con CBRI rapid test. Several aggregates proven to be alkali reactive in field concretes as well as some innocuous aggregates were tested. The most promising test methods found for Norwegian aggregates were the South African accelerated mortar bar test (NBRI) and the Canadian CSA CAN3-A23.2-14A concrete prism test. However, results suggested that the Canadian concrete prism test was not suitable for Norwegian alkali reactive sandstones and phyllite (Dahl et al. 1992), (Meland et al.1993), (Jensen 1993).

SPECIFICATIONS TO CONTROL AAR

Preventive measures, advisory notes and codes of practice to minimize the risk for AAR in Norway are limited. The Norwegian Standard NS 3420, L5 from 1986, dealing with aggregates for concrete, specifies that reactive aggregates in harmful amounts are not to be used in concrete. However no methods or recommendations to minimize the risk for AAR are presented in this standard.

In 1991 the Norwegian Concrete Society - aggregate committee, proposed methods for control and acceptance of concrete aggregates. Testing of aggregates were to be carried out in 3 steps; 1) a petrographic analysis, 2) an accelerated mortar bar test (South African NBRI method) and finally 3) a concrete prism test (CAN3-A23.2-14A). In 1991 it was believed that the concrete prism test was the most reliable test method relative to the more accelerated NBRI method.

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

In 1993 the concrete prism test was withdrawn from the DGB recommendation because of discrepancies between field experience and test results.

TEST METHODS

Two test methods are now used for assessment of alkali reactivity of Norwegian concrete aggregates, namely a petrographic analysis and the South African accelerated mortar bar test (NBRI). As mentioned earlier the Canadian Concrete Prism test CSA CAN3 A23.2-14A is no longer recommended for testing of concrete aggregates. However, this method is still recommended for assessment on cement type, admixture and concrete mix design.

Petrographic analysis

Before 1992 petrographic analysis followed the principles given in ASTM C 295, "Standard Practice for Petrographic Examination of Aggregates for Concrete". However, only 1 size fraction was counted from the sand and the coarse fraction respectively and results were given as the average grain% of the counted fractions and not as a weighted composition. Since most Norwegian alkali reactive aggregates are microcrystalline, identification and classification of aggregate grains is difficult without the use of thin section microscopy.

An improved petrographic analysis by use of point counting of thin sections has been developed to classify concrete aggregate samples (Jensen et al 1993). This method is now required by DGB for petrographic analysis of concrete aggregates. The petrographic analysis is used to classify both natural sand, coarse gravel and crushed rocks. Before point counting examination of the thin sections is carried out. Rock types are then classified according to geological nomenclature, microstructure of the rock, degree of deformation and alteration.

Results are reported as volume percentage of major rock/minerals in addition to alkali reactivity based on field experience. Based on this evaluation rocks have been divided into three main groups (Jensen et al. 1993), (Haugen & Jensen 1994), (Wigum & Lindgaard 1994):

- Reactive aggregates: Sandstone, siltstone, cataclastic rocks, acid volcanic rocks, argillaceous rocks, greywacke, marl and rock types with microcrystalline quartz (grain size less than 0.06 mm).

- Potentially reactive aggregates: Fine grained quartzite or rock types containing micro-very fine grained quartz (crystal size 0.06 - 0.13 mm).

- Innocuous aggregates: Gneiss, granite, coarse grained quartzite, crystalline limestone, gabbro and rock types with coarse grains and/or minor amounts of quartz.

Where more than 20 volume% reactive + potentially reactive rocks are found the material is classified as alkali reactive. If less than 20 volume% is found the material is classified as innocuous. The petrographic analysis is to be performed by an experienced geologist with knowledge of AAR. This is important because Norwegian rocks are varied and difficult to identify.

South African accelerated mortar bar test (NBRI)

The method follows the test procedures given by NBRI/CSIR (Davies & Oberholster 1987) but the size of mortar bars used are 4 x 4 x 16 cm.

Aggregates are assessed as potentially reactive when the 14 day expansion are higher than 0.10%, innocuous when less than 0.10%. Both sand, gravel, crushed rock and blends of natural and crushed aggregates are tested by the NBRI method.

Canadian concrete prism test CSA CAN3-A23.2-14A

The method follows the test procedures given by the Canadian standard CAN/CSA-A23.2-M90 but with increased cement content of 410 kg/m^3 concrete and prism dimensions of 100x100x400mm. After demoulding the prisms are stored in 100% humidity at 38°C with measurements up to one year.

If the expansion within one year exceeds more than 0.040% the aggregate is classified as deleterious.

ASSESSMENT OF AGGREGATE REACTIVITY

According to the DGB procedures aggregates are tested in two steps. The first step is the petrographic analysis. If less than 20 % of reactive + potentially reactive rock types is found, the aggregate is classified as innocuous and no further testing is necessary. If 20 % or more of reactive + potentially reactive rock types are found, a second step is recommended but not mandatory namely the South African accelerated mortar-bar test (NBRI). If the mortar bars expand more than 0.10% the aggregate is classified as reactive. According to the DGB recommendation the NBRI test overrule the result from the petrographic analysis.

The Canadian concrete prism test CAN3-23.2A-14A (step 3 in the NB 19 publication) is no longer recommended for testing of concrete aggregates. Several tests using this method on aggregates containing sedimentary rocks (e.g.sandstones) gave results which classified the aggregates as innocuous even when field experience showed the opposite. Therefore, the concrete prism method is no longer recommended for testing aggregates and was withdrawn from the DGB recommendations in 1993.

CORRELATION BETWEEN TEST METHODS

Where more than one test method is required to be used, as suggested by DGB, it is required that both methods classify aggregates in the same way and correlation occur between the methods. However, for geological materials which often are inhomogeneous and tested by different methods the correlation should not be expected to be high. For test methods where limits are specified results can be given by one of two statements, namely, - *yes* - the aggregate is reactive or - *no* - the aggregate is innocuous. In graphical plot lines of the limit values will depict four quadrants and four types of results or statements will be possible. In figs 1-4 it can be seen that analyze results which plots into the quadrants 1 and 3 gives the same answer by both methods. In quadrant 1 both methods classify the material as reactive and in quadrant 3 as innocuous. On the other hand in the quadrants 2 and 4 the methods gives opposite answers.

Results from the petrographic analyses, NBRI mortar bar tests and CAN 3 concrete prism tests on Norwegian concrete aggregates are shown in Figs 1-4. The figures are in many ways self-explaining.

Petrographic analysis versus CAN 3-A23.2-14A

Figure 1 shows the relationship between the petrographic analysis and the concrete prism test CAN 3-A23.2-14A. The four quadrants are defined by the value 20% reactive aggregate and 0.040% expansion of one year.

In Figure 1 the scatter of results and lack of correlation between the two methods are apparent. The majority of samples fall into the quadrant 4. Only a few samples correlate by falling into the quadrants 1 and 3. The majority of sedimentary rocks (filled triangles) give low expansions even when large amounts of reactive aggregates occur in the samples. Several of the sedimentary aggregates depicted in fig 1 come from deposits which have caused AAR in structures. Only one sample, a sedimentary rock composed of impure carbonaceous rock (metamarl) was classified reactive as known from field experience. It can be concluded the results show poor correlation between the petrographic analysis and the CAN 3-A23.2-14A concrete prism test and the CAN 3-A23.2-14A method.

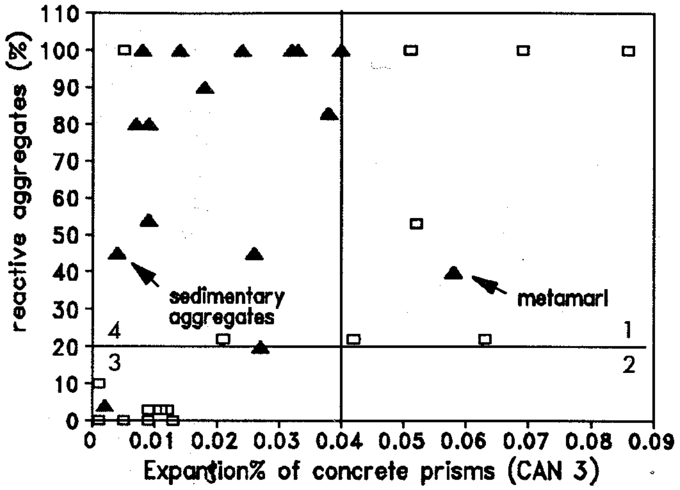

Fig. 1 Petrographic analysis versus CAN 3-A23.2-14A (34 samples)

NBRI versus CAN 3-A23.2-14A

Figure 2 shows the relationship between the South African accelerated mortar bar test NBRI and the CAN 3-A23.2-14A concrete prism test. The four quadrants are defined by the values 0.10% NBRI (14 days) and 0.040% CAN 3 (one year).

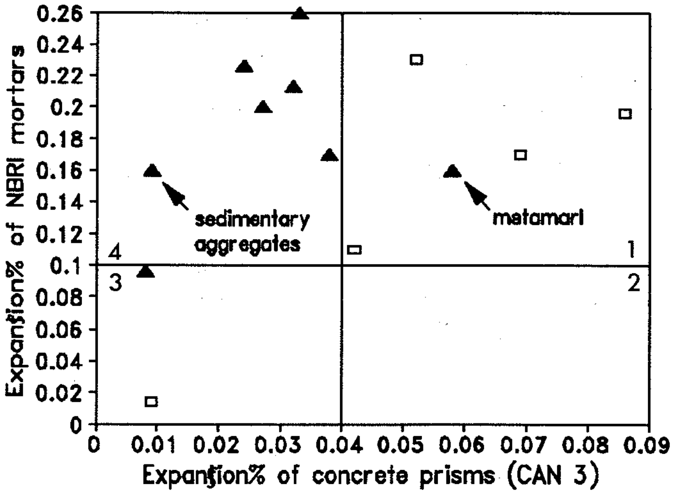

Fig. 2 NBRI mortar bar test versus CAN 3-A23.2-14A concrete prism test (13 samples)

In Figure 2 the scatter of results and lack of correlation between the two methods is apparent. Based on the 13 samples it can be concluded that there are poor correlation between the NBRI method and the CAN 3-A23.2-14A method.

Petrographic analysis versus NBRI mortar bar test of natural aggregates

Figure 3 shows the relationship between petrographic analysis and the NBRI mortar bar test of natural sand and gravel (crushed to give the NBRI grading). The four quadrants are defined by the values 0.10% NBRI (14 days) and 20% reactive aggregates.

In Figure 3 all 47 samples fall into the quadrant 1 and 3 and there is good correlation between the two methods. The correlation coefficient R^2 is only 0.46 which is rather low but not unexpected when the variability of natural aggregates is taken in account (note that the regression line goes through the intersection of limits).

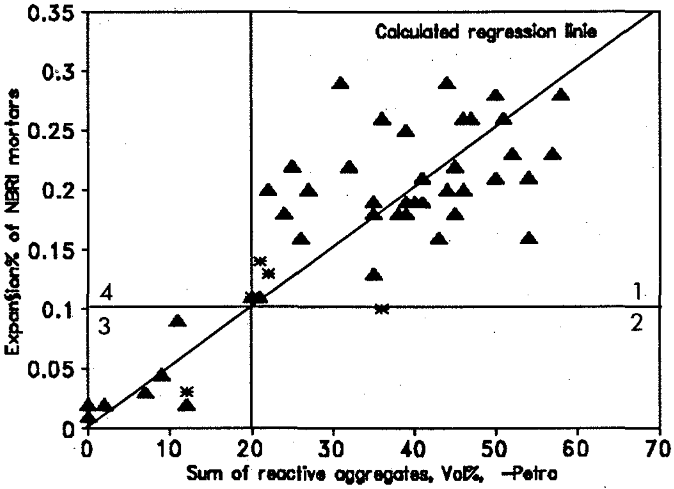

Fig. 3 Petrographic analysis versus NBRI mortar bar test of 47 natural sand and gravel.

Petrographic analysis versus NBRI test with crushed aggregates

Crushed aggregates are normally tested by petrographic analysis to identify if the aggregates are reactive. In some cases crushed aggregates are tested by the NBRI mortar bar method, too. So far there is good correlation between the two methods. This applies to for both innocuous and reactive aggregates. Testing of hornfels by the NBRI method has revealed both innocuous and reactive hornfelses. Reactive hornfels contain cryptocrystalline-microcrystalline quartz which is often difficult to identify by the petrographic analysis but is detected by the NBRI method.

Petrographic analysis versus NBRI test with blended natural/crushed aggregates

Figure 4 shows the relationship between petrographic analysis and the NBRI mortar bar test on blends of two natural sands, natural sand with crushed gravel and natural sand with crushed rock.

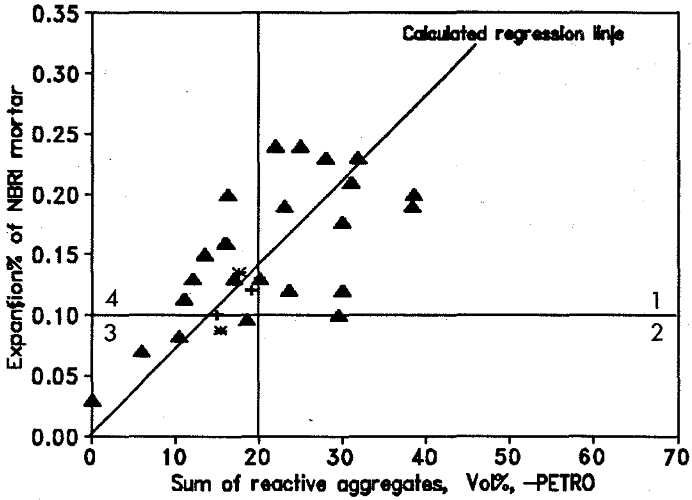

Fig. 4 Petrographic analysis versus NBRI test with 27 blends of natural/crushed aggregates.

In Figure 4 the results indicate only moderate correlation between the two methods as a number of points fall into quadrant 4. A regression analysis on 27 samples gives a regression coefficient R^2 of 0.49. Note that the regression line of the blended materials has a steeper inclination compared to natural aggregates shown in Figure 3. The reason why blends of natural aggregates and crushed rocks show poorer correlation between the petrographic method and the NBRI mortar bar test are unknown.

DISCUSSION

The research project carried out by SINTEF in 1990-1993 showed that the NBRI method was in good agreement with field experience and the CAN 3 concrete prism test was only in agreement with some types of aggregates and failed to detect reactive sandstone and phyllite. The results presented in this paper clearly suggest that the CAN 3-A23.2-14A concrete prism method does not correlate with the Norwegian petrographic analysis and the NBRI mortar bar test and more important does not agree with field experience of sedimentary aggregates. It has to be mentioned that sedimentary aggregates, sandstones, siltstone, greywacke and argillite are very common in glacio-fluvial aggregate deposits and located over large areas of Norway.

The petrographic analysis determines rock aggregates which by field experience have caused AAR in concrete structures and correlates with the NBRI test. The validity of the limit on 20 volume% reactive + potentially reactive aggregates recommended by the Norwegian Concrete Society (NB 19) and DGB can be discussed. However, for natural aggregates and crushed unmixed aggregate we have at present time not any disagreement between laboratory testing with the petrographic analysis and the NBRI method as well as field experience.

For blends of natural aggregates or natural aggregates with crushed aggregates where the amount of reactive particles have been diluted the NBRI method may give higher expansion than the 0.10% limit. This can occur even where there is less than 20 volume% reactive aggregates in the material as shown in Figure 4. The possible reason for this phenomena could be caused by pessimum relations as suggested by Shayan (1992) but could also be a testing problem caused by the NBRI method of blending materials.

It is important to note that all testing as well as the establishment of limit value in Norway with the NBRI method are based on the RILEM prism size of 4cm x 4cm x 16cm which differs from the ASTM C 490 prism size 2,5cm x 2,5cm x 28,5cm. Because the reaction in the NBRI test is influenced by diffusion of NaOH into the mortar bar the reaction will depend on the cross section of bars and the expansion time. This suggests that RILEM prisms will give lower expansion values relative to ASTM prisms at 14 days.

CONCLUSION

A combination of the Norwegian petrographic method of analysis and the South African accelerated test NBRI are presently the best tools to assess alkali reactive Norwegian slow/late expansive aggregates.

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GEOLOGIC CONDITIONING OF ASR DEVELOPMENT. A BRIEF EVALUATION OF PORTUGUESE MAINLAND

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ABSTRACT

In Portugal, up to now little attention has been given to AAR problems in concrete structures, in particular due to the fact that only few cases of this kind of deterioration have been detected. However, as the structures become older, some of them are beginning to show evident signs of ASR in concrete. The analysis and evaluation of the factors which can conditionate ASR in portuguese concrete structures, aiming at not only to diagnose the declared cases of affected structures but also to prognose the potential deterioration in others, is an important task that is being performed in LNEC. This paper deals with a simplified methodology to evaluate the geological conditioning of the use of aggregates for concrete based on a lithologic zonning of portuguese mainland, wich was applied to the geologic sources of aggregates.

Keywords: ASR, aggregates, lithologic zonning of portuguese mainland

INTRODUCTION

Actually, the development of deleterious chemical reactions, among the components of the concrete themselves or with aggressive agents coming from outside, with formation of expansive products is one of the most important process of concrete structures damage. Typical processes are those based on the alkali-aggregate reactions (alkali-silica, ASR, alkali-silicate, ASSR, and alkali-carbonate reactions) and on the sulfate-alumina reactions. In Portugal, some concrete structures have began in the last years to show evident signs of ASR occurrence and, in LNEC, quite a lot of work is being performed aiming at not only the diagnose of the declared cases of damaged structures but also the evaluation of the factors wich can conditionate ASR in portuguese concrete structures and also to attract attention for these problems (Silva, H. 1992), (Silva, H. & Rodrigues, J.D. 1993), (Reis, M.O.B. 1990).

As well known, the main conditioning factors for ASR in concrete are the existence of reactive silica and alkalis, in adequate proportions, a sufficiently high pH, usually higher than 12, and an environment where the relative humidity (RH) is higher than 80% (Coutinho, A. 1993), (CMCD 1989), (Idorn 1967). The source of reactive silica is usually the aggregate and the source of alkalis may be the cement, the additives and even the aggregates. The mixing water and the percolation water (of importance for example in dams) may also be a source of alkalis, in the first case with global distribution in the concrete mass and in the second case with a localised incidence, in the vicinity of seepage channels.

Potentially reactive silica forms are essentially those having an amorphous, subcrystalline and cryptocristalline nature and those that are strongly tectonised, presenting an undulatory extinction. The former, like opal, chert, silex, chalcedony, are considered as having an assured potential reactivity to alkalis, the ASR process occurring at short term, as long as the other conditioning factors (pH, RH, temperature) are propitious. The latter, like strained quartz, usually cause slow late reactions, whose development reaches highest rates only after ten or twenty years (Glasser & Kataoka 1981), (Hyward et al. 1988).

In the portuguese mainland there are some of the above mentioned potentially reactive silica forms, which form part of the mineralogical composition of rocks that provide aggregates for concrete. Consequently, an adequate selection of aggregates based on an early evaluation of national resources may help in avoiding or at least attenuate future distresses by ASR in concrete structures.

Studies of this nature can be carried out on several scales:

- on a national territory scale, giving access to a global knowledge, through the preparation of macrozonings;
- on a regional scale, introducing regional conditioning factors (sedimentologic, orogenic, tectonic, etc.), the knowledge being substantiated in microzonings;
- on a local scale, based on experimental criteria (mineralogical and petrographic analysis, alterability, expansibility and reactivity tests, etc.), leading to the characterization of the geologic formations to be used for concrete.

In this paper a global lithological framework is given, taking into consideration two important factors for the development of ASR in concrete (Gillot 1975), (Gogte 1981), existence of potentially reactive silica forms and of minerals with high alkali content. The zonings presented make a contribution to the global knowledge, on a national scale, of fundamental aspects related to the nature of aggregates used in the manufacturing of concrete, in the light of the possible occurrence of ASR.

MAIN LITHOLOGIC ASPECTS OF THE PORTUGUESE MAINLAND

General

Almost all types of rocks, namelly igneous, sedimentary and metamorphic ones, outcrops in the portuguese mainland (Teixeira & Gonçalves 1980), (Soares da Silva 1983). The predominant igneous rocks belong to the family of granites and gabbros (basalts mainly), and occupy about 30% of the portuguese mainland. The predominant sedimentary rocks are of a limestone nature (limestones, marlaceous limestones, marls, dolomitic limestones and dolomite), which occur in about 10% of the territory, and of a clastic nature (especially sandstones, conglomerates, sands and clays), which are distributed over about 20% of the portuguese mainland. The metamorphic rocks occur in the other 40% of the territory being of a polimetamorphic and monometamorphic nature, predominantly of hercynian age, and are composed mainly by schists, almost always associated with metagreywackes, quartzites, marbles and rocks of gneissic and amphibolic nature.

The supply of potentially reactive silica forms

The analysis of the Portuguese Geologic Chart, in the scale of 1/500.000, 1/200.000 and 1/50.000 (SGP 1972), (Teixeira 1972), and of specific documents (Neiva et al.

1979), reporting the lithologic and petrographic nature of geologic formations in the portuguese mainland, made it possible to define the potential sources of reactive silica forms and of alkalis grouped in Table 1, and conclude as follows (Silva, H. 1992):

- The silica forms, to which a greater potential reactivity is attributed, are: chert, flint, fhtanite (including jasper quartz), lyddite and palaeozoic formations associated with polimetamorphic, schist-greywacquic and quartzitic complexes; flint especially associated with mesozoic formations and of carbonaceous nature, namely jurassic limestones; several silicifications ("silcretes" included), associated with formations of very different ages; from siliceous palaeozoic schists, predominantly, to jurassic limestones with silicified vegetable fossils and radiolarians and to cenozoic and quaternary arkosic, conglomeratic and sandy deposits.
- Other potentially reactive silica forms can be identified as quartz with a disturbed cristalline network, usually metastable, due to deformation and marked shearing, or finely granulated. Quartz with those characteristics is more often found in the metamorphic palaeozoic series (quartzites, leptynites, gneisses and ordovician and silurian granulites) and particularly in palingenetic granites and of a early hercynian emplacement. Its potential reactivity has been identified with the value of the undulatory extinction angle (Gillot 1975), (Dolar-Mantuani 1981), (Buck 1986). In the descriptive documents of geologic cartography, however, the values of the measured angles are not included, so that consideration of all the rocks in which the undulatory character of the extinction was recognised may therefore be too condemnatory.

Table 1 Potential sources of reactive silica forms and alkalis in Portuguese aggregates.

POTENTIAL SOURCES OF REACTIVE SILICA FORMS	
Minerals:	Opal, Opal CT (Cristobalite, Tridimite), Obsidian, Flint, Chert, Chalcedony, Tectonized Quartz (TQ)
Rocks:	Jasper, Lyddite, Phtanyte, Diatomite, Siliceous Schist, Phyllite, (with TQ, chert or silex), Quartzite (with TQ, chert or silex), Granitoid (with TQ), Volcanite (Rhyolite, Dacite, Andesite, Basalt), Limestone (with silex or chert), Dolomite (with silex or chert)
POTENTIAL SOURCES OF ALKALIS	
Minerals:	Potassium: Sanidine, Orthoclase, Microcline, Leucite, Biotite & Muscovite Sodium: Perthite, Albite, Oligoclase, Nepheline, Sodalite
Rocks:	Granitic, Syenitic & Dioritic families, Feldspathic Hornfels, Leptynite, Arkose, Ectinite (Phyllite & Greywacke)

The supply of alkalis

In the majority of concrete works damaged by ASR, the principal source of alkalis is, in general, the aggregates themselves. The minerals that can be source of alkalis are essentially feldspars, plagioclases, (albite and oligoclase), feldspathoids and micas, that are the major components of igneous and metamorphic rocks, like granites, syenites and ectinites (SGP 1972), largely disseminated in the portuguese mainland and therefore commonly used as aggregates. Some immature alluvial sands, resulting from the alteration of granite, can also contain a high content of feldspars, plagioclases and micas and also be an important source of alkalis. This is a common situation observed in works affected by ASR.

LITHOLOGIC MACROZONING OF ASR SUSCEPTIBILITY

Methodology of evaluation

The zoning of the territory was made by taking into account the availability of outcrop materials for concrete aggregates which may be a source of the above mentioned potentially reactive silica forms and alkalis. The origins of the silica forms were divided into two groups: rocks with amorphous, subcrystalline and cryptocrystalline silica (chert, silex, fhtanite, chalcedony and silicifications); and rocks with crystalline silica (deformed quartz and with cataclasis, recognised by the undulatory character of the extinction). The analysis was made on the squares mesh corresponding to the Portuguese Geological Chart in the scale of 1/50.000. The percentage of area occupied in each square by the lithologic outcrop units was quantified on the basis of the mean area method (Silva, H. 1992). On the basis of this method, three zonings were established, two of them concerning the occurrence of silica forms potentially reactive to alkalis and one related with the occurrence of formations with alkalis. Three groups of percentage of area of squares occupied by rocks with those species were considered: an outcrop area less than 5%, practically meaningless from a global point of view, which can be ignored on a national scale; an outcrop area between 5 and 50%, and an outcrop area higher than 50%.

Potentially reactive silica forms

The map of Fig. 1 represents the network of squares with the distribution of the outcrops of rocks with silica forms of the following types: chert, silex, fhtanite, lyddite, chalcedony and several silicifications. The histogram includes also the number of squares with outcrops that occupy areas less than 5%. Its analysis makes it possible to verify that lyddites were mapped in 59 squares, while chert, fhtanite and other silicifications were mapped in more than 30 squares and silex in about 20. The zones where outcrops occupy more extensive areas are basically located in the Districts of Trás-os-Montes and Baixo Alentejo, Northeast and South of the country respectivelly.

In a similar way, the distribution of the outcrop areas of rocks with well crystallised silica forms (quartz) but with a potential reactivity conferred by the metastable crystalline structure, due to deformation, and by cataclasis, which originates a high specific surface of minerals was also made. In this zoning the same three divisions of outcrop area referred to above were also considered. As can be assumed from the analysis of the distribution, there are 67 squares where a mapping was made of the outcrop areas of igneous and metamorphic rocks higher than 5%, in which quartz presented an undulatory extinction. Only in 21, among the 67 squares, the outcrop area of those rocks was higher than 50%.

Potential sources of alkalis

Considering the rocks which usually contain minerals with a high alkali content (granites, syenites and ectinites), the respective zoning was prepared. The percentage of area occupied in the square by those types of rocks, and, as in the case of zoning of silica forms, three groups were considered: less than 5%, which was disregarded; between 5 and 50%; and higher than 50%. In about 75% of the squares there are areas higher than 5% occupied by formations which contain minerals capable of being an alkali source sufficient to feed ASR. In about 40% of the total squares, the areas occupied are higher than 50%.

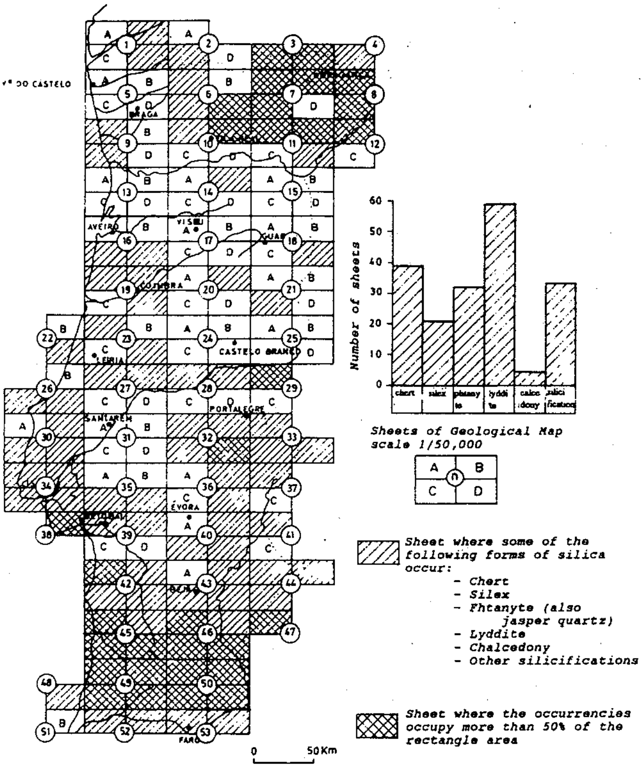

Fig. 1 Distribution of rocks with potential reactive silica forms others than quartz with undulatory extinction.

Regional distribution

Table 2 presents an estimate of the areas of the mainland districts where rocks with potentially reactive silica forms and with alkalis outcrop. Districts are indicated by their capital towns.

Table 2 Percentages of District mainland areas where rocks with potential reactive silica forms outcrop.

DISTRICT	% of outcroping area(*)		
	A	B	C
Aveiro	7.1	18.4	41.4
Beja	48.8	1.6	47.9
Braga	11.4	33.3	64.5
Bragança	52.6	4.4	31.3
Castelo Branco	1.1	13.8	31.9
Coimbra	8.8	11.6	23.6
Évora	12.2	13	35.5
Faro	58.8	0	33.2
Guarda	1.9	45.9	65.7
Leiria	11.6	0.4	1.1
Lisboa	12.7	0	13.5
Portalegre	17.5	19.2	35.3
Porto	5	38.6	62.1
Santarém	7.6	2.1	9.9
Setúbal	28.2	0	32
Viana do Castelo	3.8	54.7	77
Vila Real	21.6	15	72.4
Viseu	17.5	34.3	79.7

(*) Rocks with:
A = chert, silex, phtanite, lyddite, chalcedony, other silicifications;
B = quartz with undulatory extinction; C = minerals with alkalis.

The comparison of the distributions of rocks with potential reactive silica forms and of whose minerals can be source of alkali elements and of the areas by region presented in Table 2, makes it possible to conclude as follows:

- In the zones where silica forms with an ensured potential reactivity (chert, silex, etc.) are more frequent, there are also rocks which constitute a potential alkali source, occurring less frequently in Trás-os-Montes (region of Bragança) and more frequently in Baixo Alentejo (region of Beja) and in Algarve (region of Faro). In the Western and Algarve (South) borders however, the rocks with a high alkali content occur in very reduced areas (syenites and dolerites or basalts).
- The zones where rocks with quartz presenting an undulatory extinction and cataclasis occur more frequently are located in Minho (regions of Braga and Viana do Castelo), Douro Litoral (region of Oporto) and Beira Alta (regions of Guarda and Viseu); in these zones, rocks with high alkali content can also be more frequently found. In most cases, these are rocks presenting the two conditions referred to above, namely rocks of granitic nature.

CONCLUDING REMARKS

The analysis of the lithologic characteristics of geologic formations in the Portuguese mainland shows the existence of materials which, when used as aggregates in an extremely basic environment, may cause alkali-silica reactions. The assumption that has served as a basis for the identification of the potential reactivity of tectonic quartz was the undulatory character of the extinction when observed in a polarising microscope. Normally, the potential reactivity increases with the value of this angle, which was not quantified. There can therefore result from this zoning an aggravated conditioning for certain rocks, namely granitic, which constitute a source of the largest volume of aggregates used in Portugal. The usefulness of this study is due precisely to the global nature of the analysis prepared, but it obviously neither replaces nor invalidates studies and regional or local zonings based on criteria of petrographic and mineralogic analysis and on other appropriate tests. The development of adequate studies for the evaluation of the potential reactivity of this type of rocks in a certain site, can be the most effective way to confirm the situation resulting from the global zoning or, on the contrary, to desaggravate it.

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FIFTY YEARS OF INVESTIGATION AND CONTROL OF AAR IN NEW ZEALAND

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Abstract

The investigation of AAR carried out over the last fifty years is briefly reviewed and assessed in relation to current projects. Important initiatives over the last four years have enabled field surveys, petrographic examinations and chemical investigations to be satisfactorily correlated to explain why some unexpected cases of expansive reaction have occurred.

Keywords: Alkali-aggregate reaction, aggregate-pore solution interaction, silica fume.

Introduction

Volcanic aggregates used in New Zealand concretes have been of investigated for over fifty years. Engineers charged with the construction of major government projects recognised their potential for AAR and as early as 1943 commissioned investigation into aggregates and subsequently limited the alkali content in concretes by specifying the use of low-alkali cements. This approach has been adequate for most concretes and forms the basis of the current New Zealand code of practice for minimising AAR, (Cement & Concrete Association of New Zealand 1991).

In hindsight it is clear that if aggregates had not been investigated and limits on the alkali content of most cements imposed, serious and costly damage to a wide range of both publicly and privately owned structures would have occurred. This was confirmed when a data base on concrete aggregates showed that potentially reactive aggregates were currently being used in some 30% of concrete, (St John 1988). As a result the severity of AAR has been restricted with only minor to moderate effects on structures with less than ten cases of serious damage.

The success of the New Zealand investigations has depended on government agencies being able to conduct research independently of the construction industry over a long period. While initial work was concentrated in the laboratory investigation of affected structures has recently become more important. This paper reviews the research carried out over the last fifty years to control the incidence and severity of AAR in New Zealand.

Early laboratory studies (1945 - 1970)

Early laboratory work concentrated on the identification and testing of aggregates for reactivity and the use of pozzolans to suppress AAR, (Hutton 1945, Kennerley & Clelland 1959, Kennerley & St John 1969, Kennerley *et al* 1975, St John 1988). Extensive testing of aggregates carried out over two decades using ASTM 227 & 289 test methods, supplemented by some concrete tests, provided an excellent base for later work and clearly identified that the more acidic volcanics exhibited well defined pessimum proportions.

Several potential pozzolans were identified and tested. A naturally occurring diatomaceous pumicite was used in many major projects both as a workability aid and to supplement the use of low-alkali cement in suppressing AAR. Generally, the use of this pozzolan has been successful but there is now some indication from field investigation that in the long term this pozzolan may not be effective in completely preventing AAR where alkali is released by interaction from aggregates with the pore solution in concrete.

Inspection of structures (1975 - present)

The first case of AAR in a structure was not unequivocally identified until 1975 (St John 1975), some thirty years after testing started, and now about thirty definite and twenty possible cases have now been confirmed. Most are minor and do not affect the performance of the structure. Many would not have been identified without systematic inspection of structures by experienced concrete technologists supplemented by petrographic examination to confirm the presence of AAR (Freitag 1994). Such surveys require that the inspection team be thoroughly familiar with the appearance of AAR, especially the appearance of minor reaction, as well as other concrete defects.

Petrographic examination methods (1975 - present)

An integral part of the field inspections has been the ability to positively identify AAR in concretes using petrographic examination of thin sections. A decision was made to develop the use of large-area thin sectioning (St John & Abbott 1983) as it was considered that neither the smaller thin sections used by geologists nor large ground surfaces examined by incident light would be adequate for the examination of concrete. Danish (Andersen & Petersen 1961) and English (Jones *et al.* 1966) methods of thin sectioning were developed to the stage where large-area thin sectioning became routine.

With experience, it has been possible to identify minor AAR in structures where as little as 1% of reactive volcanics are present, and to evaluate the relationship of the expansive cracking to other faults present in the concrete. In many New Zealand concretes AAR tends to be localised and large differences can be found between cores taken from adjacent sites. The ability to cut a thin section of the complete axial plane of a 100 mm diameter core to a depth of 150 mm overcomes many of the problems associated with localisation and also gives a more realistic idea of the relationships existing between concrete components.

The effective association of inspection teams, critical sampling and petrographic examination has identified a number of previously unrecognised aspects of AAR. This success of this association requires that the petrographer be consulted on the number and size of samples needed to fully represent the concrete in question and also that samples be cut, transported and stored with care to ensure valuable information is not lost.

Intercomparison of Japanese and New Zealand volcanic rocks

In a co-operative study (Katayama *et al.* 1989) which compared Japanese and New Zealand volcanic rocks, the type of reactivity of the intermediate and acid types of volcanic rocks was shown to be related to the presence of interstitial glass, cristobalite and tridymite. It was found that when the silica in the bulk composition of the rocks falls below approximately 50% the composition of the interstitial glass changes to become non reactive and cristobalite and tridymite are usually absent. This suggested a chemical method of distinguishing reactive and non reactive rocks when the bulk silica content lies between basalt and andesite.

Further investigation of a range of New Zealand basalts was planned to confirm these conclusions but this work has not yet been carried out. In the case of volcanic rocks it has considerable potential to act as a quick screening method to determine whether particular aggregates require further testing for potential reactivity.

Investigation of fluorescence for petrographic examination

Impregnation of concretes with thin epoxy resin containing the yellow fluorescent dye Brilliant Yellow R to measure the porosity of the concrete for the determination of water cement ratio was investigated (St John 1994). For this investigation small thin sections of concrete were prepared. Determination of W/C ratio by fluorescence was found to be unreliable outside the range 0.4 to 0.6. In addition it was found that sections needed to be 25 µm thick or less and unless considerable care was exercised in ensuring full impregnation of the concrete, fluorescence in the thin sections varied and was difficult to interpret.

Investigation (St John 1994) of the uranyl acetate staining method for detecting alkali-silica gel described by Natesaiyer and Hover (1988) confirmed the effectiveness of this technique over other methods. However, it was found that the original zinc uranyl acetate solution described by Feigl (1958) gave better results than the uranyl acetate solution and may define areas of sodium concentration. Concretes that have been allowed to dry out for long periods cannot be used with either of these solutions for detection of alkali-silica gel even when freshly cut surfaces are thoroughly rewetted.

Measurement of residual expansion and its control in structures

In the case of seriously damaged bridges a prediction of residual expansion and cracking is required for planning economic maintenance of these structures. A programme designed to predict the likelihood of future expansion from the accelerated expansion of core samples has identified a qualitative relationship between in-situ and core expansions. A second programme used the accelerated core expansion test to assess the effectiveness of surface treatments on reducing expansion. The results of these programmes are reported elsewhere in these proceedings (Freitag & St John 1996).

Identification of cements and alkali analyses

The control of AAR in New Zealand has been based on the use of low alkali cement and more recently on the total soluble alkali content of concretes (Cement and Concrete Association of New Zealand 1991). As the number of structures identified with AAR has increased the question of validating this method of control becomes paramount. Few construction records are available and in some cases it is suspected that the nominated cement may not have been used. There are problems in analysing a concrete for its alkali content and even if the result is analytically correct, changes to the alkalies present caused by the environment may result in localised alkali contents differing from the original mix.

Chemical techniques have been devised by Goguel & St John (1992) to identify cements so that the original alkali content of a concrete can be reliably estimated. If the cement works can be identified the alkali content of the cement can be obtained from analytical records. When this figure is combined with a determination of cement content, the original alkali content of the concrete can be estimated. There is a another problem in that the usual method of determining alkali contents in concrete by acid dissolution gives erroneously high values. Investigation shows that dissolution of the hardened cement paste by an alkaline complexing solution to determine alkalies provides more meaningful results (Goguel 1995).

The application of these analytical methods to concrete samples obtained from structures has been effective. It has identified structures where the alkali content of the concrete has increased during the life of the structure resulting in AAR in spite of the use of low alkali cement. Some of the initial results using these methods were reported briefly at the last conference (St John & Goguel 1992). The combination of these analytical techniques together with field surveys has enabled reactive combinations of materials and mix designs to be identified (Freitag & St John 1996).

Aggregate pore solution studies and the release of alkalies from aggregates

Van Aardt and Visser (1977) showed that when feldspars are ground finely they react with saturated calcium hydroxide solutions and so release alkalies. Because of the unrealistically fine grain size used in their experiments the relationship of these results to concrete aggregates is in doubt.

In New Zealand structures where apparent increases of alkali have led to AAR, a process of elimination indicates that the only possible source for the alkali is from the aggregates. Investigations reported elsewhere in these proceedings (Goguel 1996) identify basalt as the principal aggregate involved. In addition it was found that the tendency to release alkali decreases from basalt to andesite to rhyolite.

If the results of Van Aardt and Visser (1977) are applied to the cases in the Auckland area it would appear that the feldspars in the finer fraction of the basalt crusher dust used may be releasing alkali. Other possible sources are the glassy matrix of the rock and nepheline but recent investigations (Goguel 1995) have shown that nepheline does not release alkalies to the pore solution and there is some question as to the extent of release of alkalies from the glassy matrix. Further investigation is required to determine the relationship between the mineralogy and grain size of the basalt and its ability to release alkalies to the alkaline pore solution. Our current hypothesis is that poorly crystallised feldspars formed in rapidly chilled basalt lava flows may be the principal source of the alkalies.

Andesites also release alkalies, although at a lesser rate than basalt. A number of structures have been identified where minor AAR has occurred with the use of low alkali cement and andesite aggregate. In these cases the only aggregate present is andesite usually derived from alluvial sources. The amount of extra alkali found in the concretes is much less than occurs with basalt but appears to have been sufficient to initiate the first stages of AAR.

The reactivity of silica fume

On the basis of the many reports in the literature it was assumed that silica fume would be an effective and safe pozzolan for the prevention of AAR. As the following investigations show, which initially were unrelated to our AAR studies, silica fume may cause expansive cracking.

A major investigation into the mixing and fabrication of ultra high strength mortar, known as DSP, using New Zealand cements and aggregates produced expansive cracking due to AAR in test specimens subjected to wetting and drying and outdoor exposure (St John *et al.* 1994). This AAR was found to be caused by undispersed agglomerates of silica fume greater than about 50 µm in diameter and the very high alkali content of the DSP derived from the high cement and superplasticiser contents used. As only small amounts of alkali-silica gel are associated with the reacting silica fume agglomerates it is difficult to identify petrographically. Similar results have been reported by Pettersson (1992) and Shayan et al (1993).

Investigation of a range of silica fumes (St John 1994) confirmed Japanese results (Asakura *et al.* 1993) which showed that dispersibility of silica fume could be measured by its change in particle size distribution with ultrasonic dispersion in water. Some silica fumes continue to remain dispersible irrespective of densification and time of storage while in others dispersibility was reduced. Particle size measurement clearly shows that while the average particle size of the individual spheres of silica fume is about 0.1 µm the effective average particle size can vary from 1 to 50 µm due to agglomeration. More importantly, in some densified silica fumes the larger agglomerates are extremely difficult to disperse. Whether the silica fume is used as a powder or a slurry does not seem to affect its tendency to agglomerate.

Our examination of some of the fumes by transmission electron microscopy shows that they all consist of well-formed spheres which range in size from about 20 nm to 1 µm in diameter. However, close examination of the micrographs reveals that a significant proportion of the spheres are connected to at least one other sphere and also that chains of spheres are present. It is hypothesised that this interconnection of some of the spheres could explain why some agglomerates are difficult to disperse. Where agglomerates are largely held together by Van der Waals forces mixing will break them up, but agglomerates which contain entangled chains will be difficult to disperse even with vigorous mixing. Even where the silica fume is interground with the cement it appears that some agglomerates may persist in the concrete (Sveinsdottir & Gudmundsson 1993).

This lack of dispersion applies to both concrete and mortars. Our recent investigation of a commercial high strength concrete containing 10% by weight of silica fume clearly showed the presence of well formed silica fume agglomerates ranging from 25 to 200 µm in diameter. The alkali content of the concrete was less than 2.5 kg/m^3, giving some protection against AAR. In DSP mortars containing agglomerates of silica fume, the high cement and superplasticiser content combined with the alkali in the silica fume can lead to alkali contents as high as 10 kg/m^3. With this level of alkali the extent of AAR is determined by whether sufficient water can penetrate the DSP to enable the AAR to proceed and the number and size of agglomerates present.

There is a need for further investigation of silica fumes to try and explain their different dispersibilities and find some means to improve the properties of those fumes which are difficult to disperse. Work is also required to determine the amounts and sizes of silica fume agglomerates that remain undispersed in mortars and concrete to allow a better estimate of their potential for AAR. This is necessary because there are some indications that agglomerates larger than about 30 µm persist in concrete and mortars as potentially reactive aggregates and are not consumed by pozzolanic reaction.

Discussion and conclusions

The decision to largely concentrate investigation of AAR to work on field structures has produced excellent results. The success of these investigations have relied on skilled inspection teams and devising methods for the analysis of concrete using chemical and microscopic techniques. Wet chemical techniques have made it possible to unequivocally determine that alkalies are being leached from some New Zealand basalt enabling us to explain the cause of a number of anomalous cases of AAR.

Similarly the expansive reactivity of some silica fumes challenges the concept of this material as a cement dispersant and pozzolan. It underlines the need to thoroughly investigate the expansive reactivity of pozzolans with very fine particles sizes when used in concrete and mortar containing high alkali contents.

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A CASE STUDY OF THE INVESTIGATION OF AAR IN HONG KONG

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ABSTRACT

Until very recently AAR was not considered to be a problem in Hong Kong. A case study, carried out over a three-year period, involving detailed field and laboratory investigations, has confirmed the presence of AAR in the concrete structures of a sewage treatment plant constructed using imported volcanic aggregates.

The study demonstrated the need to review international test methods and investigation procedures for the confirmation of AAR and classification of AAR potential, and to adopt them to suit local conditions.

In order to minimise AAR risk from the use of both locally produced and imported aggregates, specification clauses have been introduced to control the alkali content of concrete to be used in engineering and building works contracts in Hong Kong.

Keywords: chemical analysis, in situ monitoring, petrography, physical testing, specification

INTRODUCTION

With the huge demand for aggregates associated with the rapid growth in construction and infrastructure development, Hong Kong has had to supplement local production of aggregates, predominantly of granitic origin, with steadily increasing imports from the neighbouring parts of China.

The imported material included aggregates of volcanic origin. The use of imported volcanic aggregates together with the planned redevelopment of Hong Kong's largest quarry, which will produce significant quantities of volcanic tuff for possible use as concrete aggregate, is giving concern that significant AAR problems may develop in Hong Kong as some minerals in certain types of volcanic rocks are widely recognised as being potentially reactive.

The appearance of cracks in structures and buildings within the Territory has until recently been assigned to causes other than AAR. This view has been supported by the satisfactory performance of concrete made with local granite aggregates, which are considered to be non-reactive.

In mid-1991, the Public Works Central Laboratory (PWCL) in Hong Kong commenced an investigation into the cause of extensive cracking of the concrete in a sewage treatment plant in the New Territories of Hong Kong. The investigation involved field inspection and monitoring of cracks, core expansion tests, petrographic examination, and chemical analysis and testing of core samples taken from the site.

DESCRIPTION OF THE SEWAGE TREATMENT PLANT

The sewage treatment plant is located in the north-western New Territories of Hong Kong. It was constructed between 1980 and 1983. The plant consists of a number of lightly-reinforced concrete aeration and sedimentation tanks, surrounded by concrete parapet ring walls, and a number of small pump-house structures. The inner faces of the tanks are regularly submerged under sewage. Extensive map cracking has been developed over the internal and external surfaces of the tanks (see Figure 1). In addition, longitudinal cracking is present along the top of a number of the parapet walls. The cracking was first inspected by the PWCL in late 1991. However, it was not possible to determine when the cracking first developed.

Concrete used in the construction of the structures at the site was produced from a dedicated on-site batching plant. The concrete used in the construction of base slabs for the tanks and foundations for associated structures below ground contained granite aggregates supplied from a quarry in Hong Kong. The concrete used in the construction of the walls of the tanks and other above ground structures contained volcanic aggregates supplied from a quarry in Southern China.

Figure 1 Map cracking on parapet wall

FIELD INVESTIGATION

Sampling

In early 1992 thirty core samples, 100 mm in diameter and 300 mm long, were taken at different locations from the walls of the sedimentation and aeration tanks of the sewage treatment plant. The cores were taken both through and adjacent to cracks present in the walls of the tanks. Immediately after coring the cores were immersed in water and then subsequently removed and wrapped in wet cotton towels and placed in UPVC tubes for transport to the laboratory. Upon arrival in the laboratory, the cores were stored individually in a cylindrical container filled with clean tap water.

After about one to two weeks in this condition, considerable growth of a 'white fibrous substance' was observed on the outsides of the core samples.

Crack Survey & Monitoring

In late 1992, a field monitoring programme was commenced to examine the development of cracking in the concrete of the sewage treatment plant. The following field monitoring was carried out over a period of one and a half years:

Stereo-photography and Crack Plotting

High definition stereo photographs were taken over one selected area of the tanks which had developed intensive cracks on the concrete surface. Precision plotting of the crack patterns to an accuracy of 1 mm was also produced. The data were stored in digital form in order that the development of any further cracking at that area could be monitored in future, if required.

Monitoring of Crack Movement

Pairs of Demec points were installed at 90 degrees to selected cracks at six locations. A Demec gauge of 100 mm gauge length was used to monitor crack widening.

Ten hexagonal rosettes of Demec points were installed to enable strains to be derived from the measurements in three directions. A Demec gauge of 200 mm gauge length was used for the monitoring.

The ambient temperature and relative humidity were also recorded at the time of monitoring for reference purposes.

Typical results of the insitu crack monitoring are given in Figures 2 & 3.

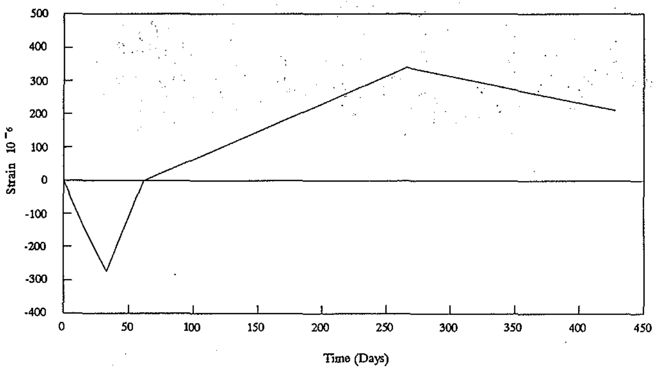

Figure 2 Typical results of insitu measurement of cracks using demec points

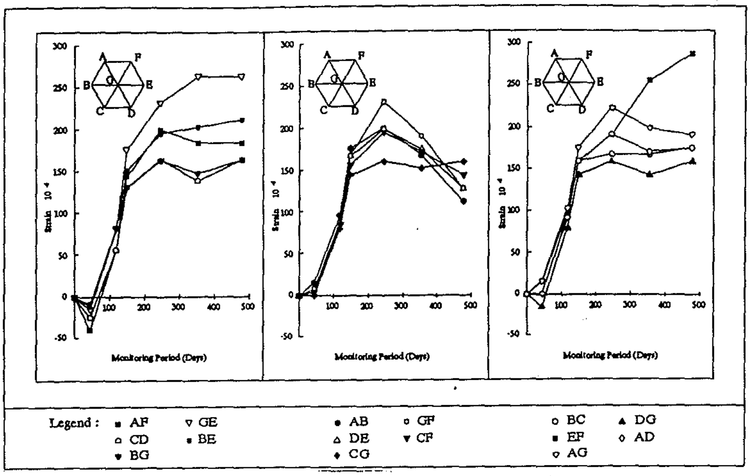

Figure 3 Typical results of insitu measurement of cracks using hexagonal rosettes of demec points

LABORATORY INVESTIGATION

Core Expansion Tests

Seventeen bars with approximate dimensions of 25 mm x 25 mm x 295 mm were prepared from nine 100 mm diameter cores taken at selected locations in the walls of the sedimentation and aeration tanks of the sewage treatment plant.

The test procedure adopted was based on ASTM C227-87 (ASTM, 1987). The bars were individually wrapped in wet towels and placed inside a sealed plastic container. They were then stored in an environmental chamber with the temperature maintained at 38°C and relative humidity above 95%. The expansions of the bars were measured at regular intervals over a period of 365 days.

A typical set of test results is shown in Figure 4. The results indicated that after fourteen days all the specimens showed an expansion of about 200 to 300 microstrains (equivalent to about 0.02 to 0.03% strain) but the subsequent expansion was negligible.

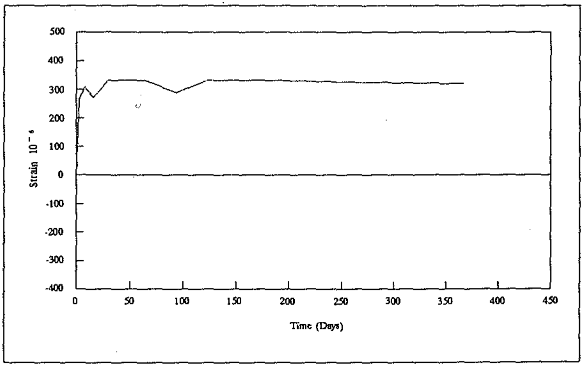

Figure 4 Typical expansion test results

Petrographic Examination

Petrographic examination of thin sections and polished plates was undertaken by the PWCL on five concrete core samples containing significant cracking. The petrographic examination of concrete core samples was carried out generally in accordance with the requirements of ASTM C856-88 (ASTM, 1988). Reference was also made to the guidelines given in BCA (1992), French (1991) and West & Sibbick (1988) for the identification and assessment of AAR in concrete.

The cores examined generally exhibited similar composition consisting of volcanic tuff coarse aggregate, fine aggregate of predominantly granite and volcanic tuff, and Ordinary Portland Cement with no additives.

The coarse and fine volcanic tuff aggregate contains abundant microcrystalline quartz in the rock matrix and strained quartz in the coarse crystal component which are widely recognised as potentially reactive materials (RILEM, 1993).

Evidence of AAR was observed in all of the core samples examined except SW115. The evidence included the presence of reaction sites and residual gel deposits in voids and microcracks. The extent of the reaction was generally quite limited in most of the core samples except SW102 and SW206, which contained numerous reaction sites and abundant gel deposits. The reaction sites where observed were mainly associated with coarse aggregate particles of volcanic tuff. A summary of the results of the petrographic examination is given below in Table 1.

Table 1 Summary of petrographic examination results on concrete cores

Cement	Coarse Aggregate + Fine Aggregate	Cracking	Presence of Reactive Minerals	Presence of Gel
Surface carbonation to 5 mm (10-25 mm in cracks). Residual clinker finely divided and generally <50 μm, occasionally up to 200-300 μm in size. Portlandite crystals where developed are around 50 μm, occasionally 100 μm in size. Small number of voids generally <2 mm in size. Ettringite crystals frequently present in voids.	20 mm nominal size crushed rock. Predominantly fine grained volcanic tuff with occasional free quartz particles. Some flaky and elongate particles. 5 mm nominal size crushed rock. Mixed sources of granite, tuff and some free quartz.	Surface macrocracks penetrate to several cm depth. Some fine cracks and micro-cracks passing through cement and aggregate particles. Ettringite present in some cracks.	Micro-crystalline and strained quartz present in coarse and fine aggregate. Some reaction centres present mainly in coarse aggregate with occasional reaction rims.	Traces of residual and layered gel in some micro-cracks and within reaction rims around aggregate particles and occasionally in small voids.

Chemical Analyses

Chemical analyses were carried out on the 'white fibrous substance' which was observed on the outside of core samples.

The substance was initially oven-dried at 105°C for 4 hours. The dried powder, when examined under an Scanning Electron Microscope (SEM), exhibited a flaky form and occasionally a flower-like pattern. Such crystal forms have been reported by others and are considered to be one of the characteristics of AAR gel products (Shayan, 1989).

Elemental analysis of the dried powder was carried out by Energy Dispersive X-ray (EDX) analyses which indicated that the substance contains mainly calcium, silicon and potassium in proportions similar to those reported by Shayan (1988) as being typical of AAR gel products (see Figure 5).

Infra-red Photospectrometry was also carried out on the dried powder. The results showed that silicon was present which reinforced the findings from the EDX (see Figure 5).

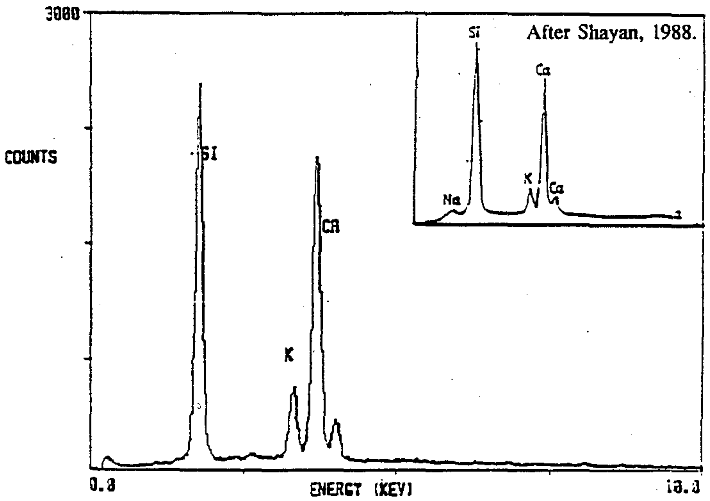

Figure 5 Typical EDX results of white gel from concrete core

DISCUSSION OF RESULTS

The results of the field monitoring indicated that the concrete structures at the treatment plant had no signs of continued expansion. The concrete core expansion test was used to assess the potential for continued expansion. BCA (1992) has suggested that if the total observed expansion is about 700 microstrains or less in the expansion tests, it is unlikely that there will be further expansion of the member from which the cores have been taken. The PWCL expansion test results show that the total expansion was about 200 to 300 microstrains for all of the 17 test specimens. The expansion in the early stage of the test may be attributed to the additional water absorption by the existing AAR gel. It may also be due to thermal expansion associated with raising the temperatures of the cores to 38°C. Negligible expansion was recorded in the later stage of the tests. This lends further support that AAR in the concrete structures has come to an end.

The presence of potentially reactive aggregate particles together with evidence of reaction sites and the occurrence of alkali silica gel identified in the core samples examined is considered to represent confirmation that AAR has occurred in the concrete (BCA, 1992; West & Sibbick, 1998). The gel deposits observed in voids and microcracks were frequently carbonated and recrystallised suggesting that much of the reaction may be historical.

The results of the chemical analyses appear to confirm that the 'white fibrous substance' is AAR gel product.

CONTRACT SPECIFICATIONS

In view of the possible risks of using potentially reactive imported and local aggregates in concrete production, the Hong Kong Government has introduced a 3kg/m^3 limit on the reactive alkali content of concrete used in Government projects. The limit is based on experience in the UK and Canada. A set of specification clauses incorporating this limit have been used in all engineering and building works contracts since late 1994 for the control of AAR in concrete. The PWCL and local universities are presently carrying out research on AAR. It will be necessary to review the present approach towards minimising AAR risk in Hong Kong as local research results become available.

CONCLUSIONS

The investigation carried out confirmed that the cracking in the concrete of a sewage treatment plant in the New Territories of Hong Kong was due to AAR. The main findings of the investigations were as follows:

(a) The insitu monitoring of cracks in the concrete indicated negligible movement throughout the monitoring period and that further cracking of the structures is unlikely to occur.
(b) The core expansion tests indicated that there was unlikely to be any further expansion in the structures from where core samples were taken.
(c) The petrographic examination and chemical analyses confirmed the presence of reactive volcanic aggregates and AAR gel products in the cores examined.

This is the first documented case study on AAR performed by the PWCL in Hong Kong. Further work is being carried out to develop and establish appropriate test methods and procedures for determining the potential reactivity of concrete constituent materials used in Hong Kong.

ACKNOWLEDGEMENTS

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ALKALI-AGGREGATE REACTIVITY IN WESTERN TAIWAN

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ABSTRACT

Aggregates selected from areas of four foothills of gravelly deposits and six main rivers in western Taiwan were investigated for alkali-reactivity by different methods. Test results of chemical method (ASTM c-289) showed that 20 of 44 samples were evaluated to be potentially alkali reactive. Further tests for these 20 reactive aggregates by mortar bar method (ASTM C-227) have been evaluated according to the criteria recommended in ASTM C-33. The evaluated results show that all of them were classed as innocuous except the aggregate from Zuo-Sweh river and the cement containing 1.2% of alkali. However, these results may underestimate the expansiveness of the aggregates, when evaluated according to the Bureau of Reclamation criteria. Furthermore, five samples selected from the 20 reactive samples were all classified as alkali reactive when evaluated by the accelerated mortar bar test method using 1 M NaOH solution at temperature of 80 ℃.

Keywords: alkali-aggregate reactivity, expansion, testmethod

INTRODUCTION

Taiwan is an island country with climate of high humidity and temperature. Besides, the geology and geography condition also make this place possible to have alkali-reactive aggregates[1,3,4]. Since 1980, the government[1-2] and academic institutes[3-5] in Taiwan have noticed the problem of alkali aggregate reaction (AAR) in concrete and have gradually devoted resources to do some survey and research work in this field.

In this article, we try to do a more complete study in alkali reactivity for the aggregates located in western Taiwan. The procedures of AAR tests mainly followed the ASTM Specifications. The Accelerated Test[6] Method was also adopted for some samples to check whether it is appropriate for the aggregates in this area or not.

EXPERIMENTS

Surveying and Sampling of Aggregates

The aggregates were selected from 6 river areas and 4 hill areas distributed in western Taiwan. The river areas from north to south include Lan-Yang, Tow-Chain, Dah-An, Dah-Duu, Zuo-Sweh, and Gaw-Pin rivers and the hill areas include San-Yi, Tai-Chung Basin, Bah-Gua Mt., and Dru-San.

The river-area aggregates were obtained from the aggregate plants near the assigned rivers. The hill-area aggregates were collected by digging them directly from gravelly deposit at the assigned hill areas. A total of 44 samples were collected, including 36 river-area samples and 8 hill-area samples. All the samples were washed, crushed and sieved with water before laboratory testing.

Chemical Test Mehod

The 44 aggregate samples were tested following the specification of ASTM C289[7]. After the amount of dissolved silica (Sc) and the reduction in alkali (Rc) were measured, the alkali reactivity of all the samples were classified.

Mortar Bar Test Method

The mortar bar method was adopted to further test the 20 samples which were classified as alkali reactive in the chemical test method. Some mortar bar specimens were made following the specification of ASTM C227[8] for these tests. In these specimens, the total alkali content (Na_2O + 0.658 K_2O) in cement had two levels of 0.82% and 1.20%. Thus, total of 40 sets of specimens were prepared to measure the expansion at different agges. According to these expansion results and the specification of ASTM C33[9] and the Bureau of Redamation criteria[10] the aggregates can be classified as potentially alkali-reactive or otherwise.

Accelerated Test Method

Among these 20 samples of mortar bar tests, we selected three relatively higher alkali-reactive samples coming from Zuo-Sweh, Lan-Yang, and Dah-An river-areas and two slightly alkali-reactive sample coming from Gaw-Pin river-area and Dru-San hill-area to perform accelerated tests also by measuring the bar expansion at different age referring to Shayan, et al. (1988), using 1 M NaOH solution at temperature of 80 °C. In these tests, the total alkali content in cement were two levels of 0.85% and 1.38%. The expansion was measured until 30 days.

RESULTS AND DISCUSSIONS

Test Results

(1) Chemical Test Method

The results of 44 samples tested by chemical method were listed in Table 1. They showed that 17 of the 44 samples were harmless aggregates; 7 samples were potentially harmful; the rest 20 samples were harmful aggregates which are mainly distributed over Zuo-Sweh, Lan-Yang, and Dah-An river-areas.

(2) Mortar Bar Test Method

We further used mortar bar method to test the 20 harmful samples classified by chemical method tests. The test results of 3-month, 6-month and 1 year expansion are shown in Table 2. Among them the expansion characteristics of four selected mortar bars are sketched in *Fig. 1*. Apparently, the expansion of the samples with 1.20% of total alkali content in cement are greater than those of the samples with 0.82% of total alkali content in cement for any mortar bar ages. By checking the level of expansion, we find that the expansion of the aggregates with 0.82% total alkali content in cement never exceeds the upper limit of the specification of ASTM C33 and the Bureau of Reclamation criteria. However, when the total alkali content in cement is 1.20%, according to the Bureau of Reclamation criteria (> 0.05% @ 6 month and > 0.1% @ 1 year)[11], most of the aggregate samples (13 of 16) coming from Lan-Yang river, Dah-An river and Zuo-Sweh river would be classed as deleterious. These results of evaluations are different from those based on ASTM C33, in which almost all the aggregates were classed as innocuous except one sample coming from Zuo-sweh river-area. So, following the criteria recommended in ASTM C33 for the evaluation of the results from mortar bar expansion test may have some short-comings.

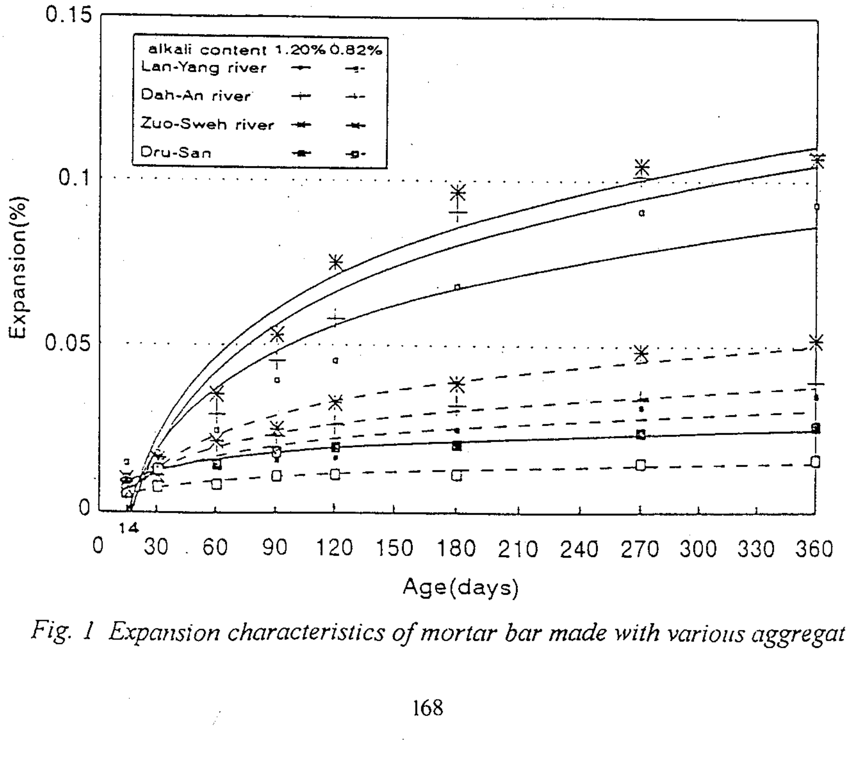

Fig. 1 Expansion characteristics of mortar bar made with various aggregates

Table 1 Potential alkali reactivity test result of aggregates (chemical method)

aggregate			composition	alkali reactivity test		classication*)
				Rc(m·mol/l)	Sc(m·mol/l)	
Lan-Yang river	up-stream	coarse	Quartz, Feldspar	58.80	100.29	×
		fine	Sandstone, Shale	88.36	115.37	△
	mid-stream	coarse	Quartz, Feldspar	73.49	103.33	×
		fine	Shale, Sandstone	86.86	118.33	△
	down-stream	coarse	Quartz, Feldspar	44.27	99.29	×
		fine	Slate, Shale, Sandstone	87.86	115.65	△
Tow-Chain river	up-stream	coarse	Quartz	88.86	69.63	○
		fine	Shale, Sandstone	79.51	61.97	○
	mid-stream	coarse	Quartz, Feldspar	83.52	56.07	○
		fine	Shale, Sandstone, Quartz	79.33	70.14	×
	down-stream	coarse	Quartz, Feldspar	74.33	61.96	○
		fine	Shale, Sandstone	97.05	48.95	○
Dah-An river	up-stream	coarse	Quartz, Sandstone	58.59	78.72	×
		fine	Shale, Sandstone, Quartz	57.76	67.72	×
	mid-stream	coarse	Quartz, Feldspar	53.64	77.85	×
		fine	Shale, Sandstone	56.11	63.62	×
	down-stream	coarse	Quartz, Sandstone	28.06	60.72	×
		fine	Quartz, Sandstone	32.68	64.48	×
Dah-Duu river	up-stream	coarse		51.16	59.35	×
		fine	Shale, Sandstone	60.74	49.59	○
	mid-stream	coarse		38.30	50.35	×
		fine	Sandstone, Shale	56.51	50.12	○
	down-stream	coarse		36.31	46.98	×
		fine	Sandstone, Shale, Quartz	55.68	52.07	○
Zuo-Sweh river	up-stream	coarse	Quartz, Feldspar	88.59	100.91	△
		fine	Sandstone, Shale	108.53	139.95	△
	mid-stream	coarse	Quartz, Feldspar	72.13	82.78	×
		fine	Sandstone, Shale, Quartz	64.49	120.04	×
	down-stream	coarse	Quartz, Feldspar, Mica	55.51	67.01	×
		fine	Sandstone, Shale, Quartz	63.48	132.28	×
Gaw-Pin river	up-stream	coarse	Quartz, Sandstone	92.24	69.34	○
		fine	Shale, Sandstone	102.05	75.74	○
	mid-stream	coarse	Quartz, Feldspar, Calcite	62.49	44.46	○
		fine	Shale, Sandstone	97.06	100.49	△
	down-stream	coarse	Quartz, Calcite, Sandstone	64.98	54.48	○
		fine	Shale, Sandstone	92.24	109.71	△
San-Yi	1	coarse	Quartz, Sandstone	39.59	49.66	×
	2	coarse	Feldspar, Mica	44.27	48.02	×
Tai-Chung	1	coarse	Quartz, Sandstone	41.92	38.79	○
	2	coarse	Feldspar, Mica	38.92	48.50	×
Bah-Gua	1	coarse	Quartz, Sandstone	79.01	37.57	○
	2	coarse	Feldspar, Mica	74.33	35.02	○
Dru-San	1	coarse	Quartz, Feldspar	57.90	46.10	○
	2	coarse	Sandstone	59.30	25.18	○

*) ○ : non-reactive
△ : potentially reactive
× : reactive

Table 2 Mortar bar test result

aggregate			alkali level	expansion (%)			alkali level	expansion (%)		
				at 3 month	at 6 month	1 year		at 3 month	at 6 month	1 year
Lan-Yang river	up-stream	coarse	1.2%	0.02977	0.05228	0.06429	0.82%	0.01600	0.02800	0.03875
		fine		0.02151	0.03277	0.03753		0.00976	0.01876	0.02451
	mid-stream	coarse		0.03925	0.06800	0.09301		0.01500	0.02449	0.03524
		fine		0.02624	0.05098	0.06646		0.01350	0.02299	0.03474
	down-stream	coarse		0.02576	0.05201	0.06226		0.01475	0.01875	0.02275
		fine		0.01927	0.03503	0.04204		0.01200	0.01525	0.02676
Tow-Chain river	up-stream	coarse		0.01276	0.01702	0.03053		0.00875	0.01050	0.02225
		fine		0.01375	0.01976	0.03201		0.01199	0.01399	0.02498
	mid-stream	coarse		0.01501	0.01877	0.03127		0.01075	0.01200	0.02225
		fine		0.01549	0.01949	0.02974		0.01375	0.01725	0.02826
	down-stream	coarse		0.01526	0.02101	0.03252		0.01277	0.01477	0.02578
		fine		0.01800	0.02475	0.03585		0.01475	0.01801	0.02927
Dah-An river	up-stream	coarse		0.04530	0.09060	0.10862		0.02228	0.03203	0.03928
		fine		0.03249	0.06747	0.08421		0.02303	0.02628	0.03554
	mid-stream	coarse		0.03877	0.06480	0.07605		0.01875	0.02299	0.02749
		fine		0.03605	0.05934	0.06986		0.01850	0.02325	0.03024
	down-stream	coarse		0.04783	0.07761	0.08613		0.02128	0.02679	0.03229
		fine		0.03780	0.05733	0.06559		0.01908	0.02159	0.02560
Dah-Duu river	up-stream	coarse		0.02924	0.04823	0.05773		0.01875	0.02250	0.02750
		fine		0.02525	0.03950	0.04799		0.02099	0.02448	0.02548
	mid-stream	coarse		0.02998	0.04747	0.05571		0.01849	0.02274	0.02574
		fine		0.02670	0.04670	0.05495		0.01673	0.02147	0.02297
	down-stream	coarse		0.02402	0.03978	0.04529		0.01874	0.02099	0.02249
		fine		0.02049	0.02174	0.02824		0.01552	0.02128	0.02303
Zuo-Sweh river	up-stream	coarse		0.05306	0.09660	0.10711		0.02454	0.03856	0.05185
		fine		0.05194	0.09314	0.10664		0.02354	0.03632	0.04559
	mid-stream	coarse		0.02526	0.03627	0.04352		0.01727	0.02053	0.02503
		fine		0.02878	0.04730	0.05405		0.01701	0.02202	0.02677
	down-stream	coarse		0.03280	0.04682	0.06560		0.01427	0.01777	0.02353
		fine		0.03578	0.06305	0.08107		0.02279	0.02280	0.03006
Gaw-Pin river	up-stream	coarse		0.02302	0.03153	0.04204		0.01750	0.01875	0.02325
		fine		0.02478	0.02628	0.03629		0.01980	0.02079	0.02681
	mid-stream	coarse		0.02723	0.03272	0.04421		0.01956	0.02031	0.02407
		fine		0.02570	0.03550	0.05024		0.01852	0.01977	0.02553
	down-stream	coarse		0.02277	0.03028	0.04704		0.01276	0.01451	0.02302
		fine		0.02426	0.03476	0.04151		0.01476	0.01677	0.02402
San-Yi	1	coarse		0.01676	0.02652	0.03453		0.01151	0.01451	0.02126
	2	coarse		0.01725	0.02625	0.03651		0.00926	0.01301	0.01926
Tai-Chung	1	coarse		0.01751	0.02126	0.02751		0.01200	0.01279	0.01876
	2	coarse		0.02352	0.02977	0.04428		0.01301	0.01401	0.01901
Bah-Gua Mt.	1	coarse		0.01500	0.01775	0.02175		0.00825	0.00975	0.01525
	2	coarse		0.01476	0.01651	0.02127		0.01000	0.01100	0.01424
Dru-San	1	coarse		0.01750	0.02001	0.02576		0.01025	0.01100	0.01575
	2	coarse		0.01575	0.01750	0.02350		0.01226	0.01301	0.01926

(3) Accelerated Test Method

The test results for the 4 selected aggregate samples are shown in *Fig. 2*. Based on these data, the expansion of all the 10-day-age samples are over 0.1%. Not only three samples which were relatively higher alkali-reactive in mortar bar method tests but also the other two which were slightly alkali-reactive are all classified as reactive. These test results are very different from those of mortar bar tests. Whether this method is suitable for detecting the alkali-reactive of the aggregates in western Taiwan or not is worth a further study.

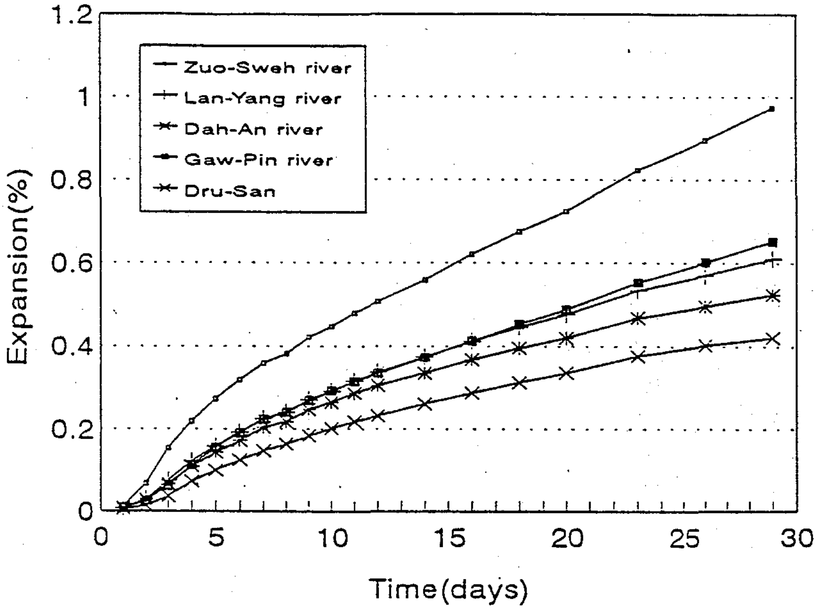

Fig.2 Expansion v. time of mortar bar subjected to the accelerated test

A combined Evaluation for Alkali-Reactivity of Aggregates in Western Taian

According to the test results of chemical method, 20 of 44 samples were tested to be harmful aggregate. When these 20 harmful samples were further tested by mortar bar method, it was found that the evaluation following the criteria recommended in ASTM C33 may underestimate the expansiveness of the aggregates. And when evaluated according to the Bureau of Reclamation criteria, only the aggregates from Lan-Yang river, Dah-An river and Zuo-Sweh river were classed as deleterious. It is therefore evident that the mortar bar method (ASTM C227) is inadequate for exactly identifying the reactivities of slowly reactive aggregates. These aggregates need to be tested according to more advanced methods[6].

The test results of accelerated method show that the 10-day expansion values of all samples are much greater than this method's upper limit, 0.1%. Accordingly, these aggregates will be suggested to be reactive.

Since the traditional mortar bar methods are no longer considered satisfactory for evaluating the potential reactivities of aggregates, there is a need to refine the existing standard tests and to develop new methods. Maybe the accelerated method is a better method, however, it needs further investigations before using this method as the AAR tests for the aggregates distributed in western Taiwan.

CONCLUSIONS

After the chemical method, the mortar bar method, and the accelerated method were investigated for the alkali reactivities of aggregates in western Taiwan, the following conclusions can be made :

(1) Twenty out of 44 samples were classified as alkali reactive aggregates by the mechanical method of ASTM C289.
(2) These 20 alkali reactive samples were further tested according to the mortar bar method of ASTM C227. The test results were evaluated by both the criteria of ASTM C33 and the Bureau Reclamation. The conclusions came to that the former one apparently underestimates the expansiveness of the aggregates. This method therefore needs to be refined.
(3) Five samples were selected from the 20 samples of mortar bar tests and tested by the accelerated method. The results show that all the 10-day expansions are much greater than the suggested upper limit, 0.10%, and should be classified as reactive.
(4) It is suggested that using the accelerated method as the AAR test method for the aggregates distributed in western Taiwan needs further investigations.

ACKNOWLEDGMENTS

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General Topics

on

Alkali-Aggregate Reaction

PHANTOM, OPPORTUNISTIC, HISTORICAL AND REAL AAR
GETTING DIAGNOSIS RIGHT

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Abstract

Investigations into the incidence of AAR around the world have long been bedevilled by frequent examples of misdiagnosis, of misinterpretation and of misguided attempts either to exaggerate or to understate the involvement of AAR in concrete deterioration. Based upon accumulated international experience and the re-assessment of selected examples, it is demonstrated that AAR can occur in concrete in a number of guises, including both benign and malignant manifestations, only some of which represent a significant continuing or future risk of damage. It is argued that accurate and reliable diagnosis of AAR, whilst often not straightforward, is nevertheless both possible and essential if the true significance of these reactions to overall concrete durability is to be fully understood.
Keywords: Alkali-aggregate reactivity, Concrete, Delayed ettringite formation, Diagnosis.

The Challenge

The importance of correct diagnosis when investigating concrete damage and/or deterioration cannot be overstated. Successful remedial solutions depend upon the initial diagnosis being accurate: nobody expects to die of cholera when they have been treated for toothache. Moreover, the effectiveness of preventative measures and the usefulness of indicative pre-construction tests are often established by correlation with deduced 'actual' field performance, so that unreliable diagnosis could undermine the assurance of future avoidance of the problem and perhaps falsely seem to legitimise an unreliable test procedure.

Alkali-aggregate reactivity (AAR) has achieved worldwide a notoriety completely out of proportion to its low frequency of occurrence. This probably results in part from the outmoded notion that the reaction is inevitably and irreversibly fatal to the affected structure, whereas in reality few, if any, structures have had to be demolished primarily because the concrete was affected by damaging AAR. In some places, this apprehension of the consequences of the reaction has caused structure-owning authorities to be unduly defensive and this has hampered many objective attempts to quantify the incidence of AAR.

Overall, it seems likely that the number of concrete structures significantly damaged by some form of AAR has been over-estimated, as a proportion of those investigated, because of the widespread tendency to base the diagnosis on incomplete evidence. Sims (1992a) stressed the need to distinguish between the evidence indicating the occurrence of a reaction and the separate evidence which might confirm that damage had resulted from that reaction. He suggested a five-point consideration scheme, taking into account site evidence, the presence and extent of a reaction within the concrete and, crucially, a characteristic micro-crack pattern demonstrating a causal link between reaction and damage.

A vital component of the five-point scheme was the elimination of other, statistically more likely, causes of damage. In many investigations, either this step is omitted altogether or the significance of any other observations is misinterpreted, so that damage is sometimes attributed to AAR, mainly because it has been identified, even when another damaging or deteriorative mechanism is present and might well be the primary or prevalent factor. The relative ease with which the occurrence of an AAR reaction (as opposed to the causation of damage) can be identified by a petrographer in the laboratory (Palmer 1992) contributes to this problem, as some of the more frequently encountered causes of damage to concrete in practice, such as drying shrinkage, leave only indicative signs for petrographic discovery.

Selected diagnosed case studies are now reviewed[1], which demonstrate that, in addition to examples of 'real' AAR which gave rise to significant concrete damage and the need for repair and management of the structure concerned, AAR can occur in a number of other guises. These include 'historical' AAR wherein physical stability has been achieved and the damage is not continuing, 'opportunistic' AAR which has occurred as a secondary result of other damage or because exposure conditions have changed, and 'phantom' AAR for those examples of concrete damage which, despite allegations to the contrary, are unassociated with AAR. Two currently contentious issues which affect diagnosis are also considered.

Real AAR

The extensive literature on AAR contains many examples of case studies of structures demonstrated by exhaustive investigation to be damaged by AAR. Sims (1992b) reviewed some of the best known UK examples, including a dam in Jersey (the Channel Islands), car park and highway bridge structures in South-West England, and highway bridge and other structures in the English Midlands. It is now apparent that many such AAR-affected structures are nonetheless capable of remaining in long service, suitably monitored and/or repaired or strengthened, and some may become examples of 'historical' AAR in due course.

Historical AAR

It is a practical reality that concrete structures are often investigated only after surface damage has become pronounced and, by that time, in some cases the worst of any detrimental effect of AAR might have been effectively completed, so that the AAR is 'historical'. The aged nature of the AAR will be betrayed by the form of the reaction product, which can become calcified and crystallised with time, and by the nature of the surface cracks and internal microcracks, which may be grime-engrained and infilled with leachate deposits respectively. Expansion testing of suitable samples will confirm the absence of residual expansive potential.

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[1] The investigations described were carried out as commercial projects and, in the interests of client confidentiality, the detailed site locations cannot be revealed.

Concrete parapet walls of the treatment tanks at a sewage treatment works in South-East Asia were found to exhibit historical AAR. The concrete was 8 years old at the time of inspection and the tank walls frequently exhibited map-cracking generally reminiscent of that caused by AAR in unstressed concrete. Microscopical examination of core samples identified coarse aggregate particles, comprising volcanic tuff with a microcrystalline quartz matrix, associated with sites of expansive reaction and low concentrations of gel deposits in voids and microcracks. The gel deposits were frequently crystallised and carbonated (**Figure 1**).

Assessment of the micro-crack pattern indicated a causal relationship between much of the micro-cracking and the coarse aggregate reaction sites. Abundant ettringite was observed in some cracks and voids, including some aggregate boundary spaces, but generally the concrete type and the ettringite mode of occurrence were not considered consistent with 'delayed ettringite formation' (DEF) and the presence of visible ettringite was instead considered to indicate a continuous movement of moisture through the cracked concrete.

Core samples, which had been immediately wrapped after extraction to prevent desiccation, were tested using the BCA method (Palmer 1992) and failed to indicate any significant short-term or longer-term expansion. It was concluded that the previously expansive AAR was now dormant or 'historical', with little or no residual potential for continued expansive behaviour whilst current conditions prevailed.

Opportunistic AAR

It is well established that AAR is able to occur when there is, in combination, a sufficiently high level of alkalies in the concrete, a critical content of potentially reactive material in the aggregate and an adequate supply of moisture; a certain amount of calcium might also be a requirement for expansive AAR (Helmuth et al 1993). Minimisation of risk is usually based on ensuring the absence of at least one of these factors (Hawkins 1995). However, the situation can sometimes be reversed in service, so that a controlling factor which was initially eliminated or suitably reduced, or in any case absent, can be re-introduced by changes which affect the concrete material or localised environmental conditions. In particular, moisture ingress can be facilitated by earlier non-AAR cracking and/or local alkali concentrations can result from certain moisture flow patterns.

A motorway bridge deck in northern mainland Europe had become severely cracked, in a pattern reflecting the main reinforcement positions, because of excessive loading deflections resulting from under-design. Initially, on the basis of some microscopical (including SEM) examination of concrete samples, the cracking had been attributed to AAR, apparently on the strength of the discovery of gel deposits.

A subsequent investigation confirmed the presence of some AAR, involving chert particles contained within the fine aggregate, but found the reaction was often largely confined to concrete proximal to the bridge deck soffit, where there was evidence of continuous water percolation through the deck, leading to concentrations of alkali, visible ettringite and other secondary hydrates.

The evidence of AAR was quantified using the method described by Sims et al (1992), showing greater abundance of reaction in the lower deck (**Table 1**). Chemical analyses showed the concrete alkali (Na_2Oeq.) content to be around 5 kg/m^3 in the lower deck slab, compared with around 3 kg/m^3 in the upper part of the deck. Also the gel deposits lining the micro-cracks in the lower deck were frequently layered and carbonated (**Figure 2**), suggesting periodic, perhaps seasonal, phases of water percolation and gel formation. It was also suggested that the seasonal use of de-icing salt would provide an additional source of alkali. In summary, the deflection cracking had facilitated the development of 'opportunistic' AAR.

Table 1 *Quantified evidence of AAR using the method devised by Sims et al (1992).*
Data simplified from the investigation of a highway bridge structure in Europe.

	Portion of Deck	Deck Flank 1	Deck Flank 2	Deck Axis 1	Deck Axis 2	Overall
Feature		Counts per 100 cm^2 (or % where indicated)				
Total chert particles >1mm & <1mm	Upper	270	311	420	334	1335
	Lower	368	317	419	464	1568
Reacted chert particles >1mm & <1mm	Upper	20	--	33	29	82
	Lower	120	128	97	199	544
% reacted chert particles	Upper	7.4 %	0.0 %	7.9 %	8.7 %	6.1 %
	Lower	32.6 %	40.4 %	23.2 %	42.9 %	34.7 %
Total cracks & microcracks	Upper	166	150	374	277	967
	Lower	500	440	568	632	2140
Gel-filled cracks & microcracks	Upper	42	--	147	108	297
	Lower	264	327	181	238	1010
% gel-filled cracks & microcracks	Upper	25.3 %	0.0 %	39.3 %	39.0 %	30.7 %
	Lower	52.8 %	74.3 %	31.9 %	37.7 %	47.2 %
Gel-filled voids >1mm & <1mm	Upper	5	--	18	5	28
	Lower	6	--	10	18	34

In certain circumstances, surface pop-outs can be induced, often without any other damage, on the painted drying faces of concrete retaining walls. Moisture continuously migrating through the wall from the wet ground behind is able to redistribute alkalies and cause a greatly alkali-enriched zone just behind the painted wall surface, promoting the localised AAR expansion of isolated reactive aggregate particles just beneath. Some UK examples with sporadic cherts have been described (Sims & Sotiropoulos 1989, Sims 1992b) in which the alkali content in the concrete behind the drying surface was enhanced by up to 5 times: one sample representing the surface-most 25mm exhibited an Na_2Oeq. content of 18 kg/m^3.

Similar occurrences have been encountered by the author on a Caribbean island, involving a crushed aggregate of mixed tuffs, some of which were unmetamorphosed and contained variable proportions of volcanic glass and some others were metamorphosed and exhibited microcrystalline quartz in the matrix. The aggregate was in widespread local use, but, on the basis of a visual survey only, concrete damage reminiscent of AAR was mainly limited to some maritime exposures. However, surface pop-outs were identified on certain walls of inland buildings, where broadly similar conditions to those described above for some UK retaining walls were satisfied (**Figure** 3). Salomon et al (1992), reporting apparently similar occurrences, suggested that pop-outs were only the initial manifestation of this type of 'opportunistic' AAR, with cracking and other disorders appearing later.

Phantom AAR

Greatest concern naturally centres on examples of concrete damage or deterioration which are wrongly diagnosed as being affected by AAR, either because AAR has not occurred at all or in cases where evidence of some AAR is present but the real cause is overlooked or not recognised. Such 'phantom' AAR helps to exaggerate the extent of AAR occurrence.

At a tall transmitter mast in South-West England, the large concrete anchor blocks for the wire stays had developed a worsening pattern of map-cracking after 12 years. Although an investigation in 1981 had identified only traces of AAR, the owners decided to build a replacement set of blocks and paid considerable attention to avoiding AAR therein, whilst apparently failing to realise the potential significance of the recorded temperatures of up to 75°C generated during hardening of these nearly 200m^3 unreinforced (apart from near-surface mesh) blocks. A developing pattern of cracking, similar to that experienced with the earlier blocks, was investigated 7 years later (**Figure** 4).

Regular visual inspection of the block surfaces had confirmed that the pattern of cracking was still worsening, including the appearance of new cracks and the rupturing of repaired cracks, even after 7 years. AAR was initially suspected because of this long-term apparent crack initiation, but petrographic examination of concrete from various depths within the block could only find a low incidence of AAR associated with chert particles in the sand. AAR was therefore eliminated as a cause of the cracking observed. Instead, the damage was attributed to a combination of thermal contraction and differential plastic settlement in which cracks would have been initiated deep within the block under and around the embedded complex of anchor elements. It was argued that normal materials movements within the concrete would then have caused these initially-formed plastic settlement cracks to propagate towards the external block surfaces, where they would only become apparent with time (**Figure** 5).

These tentative conclusions remain unchallenged, although reconsideration of the petrographic evidence during the preparation of this paper has revealed the observation of some ettringite deposits surrounding aggregate particles, which might possibly indicate DEF (Lawrence et al 1990). The occurrence of DEF would also be consistent with the high temperatures known to have developed during the hardening of the blocks. It is a case which highlights the difficulties and potential pitfalls of diagnosis in a developing science, but, whatever the precise combination of causes, the original assumption of AAR was illusory.

A second case of 'phantom' AAR can only be described in outline, because it remains *sub judice*. In Europe, some concrete structures made using a high-slag cement had developed patterns of severe surface cracking at an early age. Core samples were taken to a laboratory, where they were examined by a variety of sophisticated techniques including SEM and electron probe microanalysis. Small deposits of alkali-silicate gel were thereby identified and notwithstanding the high-slag character and youth of the concrete, also without reference to the nature of the cracking on site, it became assumed that the damage in dispute was caused by AAR. In actuality, the cracking was largely the result of drying shrinkage, facilitated by inadequate curing and probably exacerbated by an excessive water content of a reactivated concrete; the detected traces of AAR were merely incidental.

Some Controversies

In recent years a number of concrete structures in the UK, made using greywacke aggregates, have been identified as exhibiting AAR damage (Blackwell & Pettifer 1992). Although the evidence of AAR, including its causal link to some damage, is indisputable, many of these structures have a long service history (the old Maentwrog Dam was completed in 1928) and exhibit a variety of other forms of concrete deterioration, such as water leaching and frost attack. The primary cause of damage in some greywacke concrete structures, and particularly the possible role of AAR, have thus been contentious in the UK. However, in the latest AAR guidance (Hawkins 1995), special consideration is recommended for greywacke aggregates.

DEF has recently been recognised as an expansive mechanism in some types of concrete (Lawrence et al 1990). It has been claimed that cases of DEF have previously been mistakenly attributed to other causes (Hobbs 1994). This is, of course, inevitably the case with newly recognised forms of deterioration, but it is more questionable whether DEF has particularly been misdiagnosed as AAR. Many, if not most, reported examples of AAR damage affect types of structural concrete which would not meet the high early temperature conditions which are instrumental in promoting DEF. Also, as elevated temperature and access to moisture would promote AAR as well as DEF, combined occurrences might be anticipated, when the unequivocal attribution of a primary cause of cracking might be difficult.

Conclusions

- Correct AAR diagnosis optimises remedial action and helps to assure future prevention.
- A five-point AAR assessment scheme is commended, using both site and laboratory evidence and taking other possible, statistically more likely, mechanisms into account.
- In addition to undoubted cases of damaging AAR, some examples have been outlined which illustrate the occurrence in diagnoses of 'historical', 'opportunistic' and 'phantom' AAR, wherein damaging AAR is variously dormant, secondary, adventitious or absent.

- UK greywacke aggregates are now recognised as potentially capable of expansive AAR.
- DEF has been misdiagnosed previously, but not necessarily invariably mistaken for AAR, and combined occurrences of DEF and AAR might be anticipated.

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Figure 1 Photomicrograph - crystallised gel in particle boundary space - S E Asia.

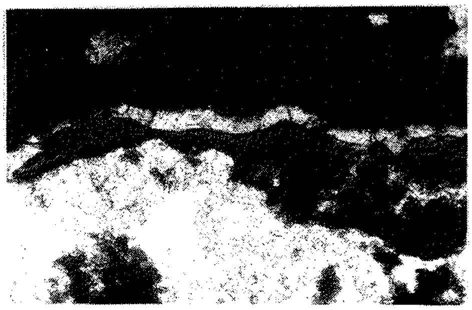

Figure 2 Photomicrograph - layered crystallised gel in crack - bridge deck in Europe.

Figure 3 Pop - outs on a retaining wall in the Caribbean.

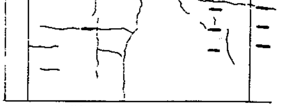

Figure 4 Pattern of cracking on top of a concrete anchor block - S W England.

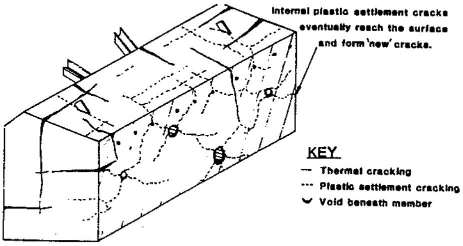

Figure 5 Postulated combination of thermal and plastic settlement cracking (cf Figure 4).

ALKALI AGGREGATE REACTION IN EXISTING STRUCTURES - WHAT CAN IT TELL US?

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ABSTRACT

Concrete in New Zealand structures was used to study three aspects of AAR. Firstly, the incidence of AAR was examined to identify the conditions which lead to the reaction so that they can be avoided in future constructions. Secondly, in situ and accelerated expansions of concrete from five structures affected by AAR were compared to find out whether accelerated core expansions can be used to predict the likelihood of future in situ expansion. Thirdly, the ability of a proprietary surface treatment to reduce further expansion was evaluated by accelerated core expansion tests.

Keywords: alkali aggregate reaction, expansion monitoring, surface treatments

INTRODUCTION

Potentially reactive aggregates are the cheapest and most readily available in parts of New Zealand and there is a need to determine how they can be used safely. In addition, once a structure has been found to be affected by AAR its owners want to know what action, if any, is needed to maintain its performance.

We have used the performance of field concretes to look at three aspects of managing AAR. These are: to identify the reactive combinations of mix designs and materials so that they can be avoided in future constructions; to find out whether AAR expansion of in situ concrete can be predicted from the expansion of core samples; and to see whether a surface treatment could prevent or reduce further expansion of concrete damaged by AAR. This approach has allowed us to study more concretes than would have been possible with laboratory mixes and avoids the difficulty of predicting in-situ performance from laboratory mixed concretes.

PREVENTION OF AAR IN NEW CONCRETES

Selection of "problem" aggregates to study

The incidence and severity of AAR in New Zealand has been limited by the widespread use of low alkali cements. Surveys of several hundred bridges in areas where potentially reactive aggregates are available found that at most 12% of the structures in each area showed signs of AAR (Freitag 1994). Most damage observed was minor. Only five bridges with extensive networks of wide cracks were seen, all in the Tongariro area.

ASTM C289 and C227 tests had earlier identified that andesites and rhyolites from Taranaki, Tongariro and Waikato River deposits were reactive or potentially reactive (Kennerley & St John. 1988). Taranaki andesites and Waikato River deposits are widely used in concrete because alternatives are expensive. Tongariro volcanics are not used as often but have produced severe AAR damage. These three types of aggregate were thus considered most worthy of closer study.

The early investigations of these aggregates included assessing the effect of alkali content on the incidence of reaction, and the current code of practice for minimising AAR in New Zealand (CCANZ 1991) is based on this work, limiting concrete alkali contents to 2.5 kg/m^3 and/or using mineral admixtures or slag. The present investigations aim to specify "safe" alkali limits for the three selected aggregates so that unnecessary costs are not incurred in complying with inappropriate restrictions.

Techniques

Core samples were taken from concretes showing characteristics such as triple point cracking, pattern cracking, open cracks, or wet stained cracks, which were considered the most likely indicators of AAR.

Petrographic techniques were used to identify the aggregates present and whether AAR had actually occurred. Information on the materials and mix designs of concretes believed to be affected by AAR was obtained from construction records wherever possible. To confirm the materials used, cement source and alkali contents were determined by chemical analysis. Cement contents were estimated from point counts in thin section and from determination of water to cement ratio by fluorescence in thin section. These techniques are discussed in an accompanying paper (St John & Freitag. 1996).

Results

For the Taranaki concretes there was no consistent relationship between the incidence of AAR and concrete alkali contents. The reaction occurred in some concretes which contained 1.2-1.7 kg/m^3 alkalis, yet other concretes with the same or higher alkali contents were not cracked and therefore not sampled. Cement alkali contents determined from chemical analyses were within the range indicated by the original records, and chloride ion contents were very low despite the surfaces sampled being exposed to sea spray. These observations suggest that the concrete surfaces were too impermeable for alkalis to be leached by runoff or augmented by external sources. The apparently variable reactivity is therefore probably due to the difference between andesites from different volcanic events. Exposure to moisture could also contribute, since the cracking was most often seen on bridge abutments. The findings suggest that an alkali limit of 1.5 kg/m^3 might be necessary to completely prevent cracking due to AAR where Taranaki andesites are used, compared to earlier laboratory results which suggested that concrete alkali contents less than 3.5 kg/m^3 would be "safe" (Smith 1988).

Among the Tongariro concretes the most severe damage occurred when a high alkali cement was combined with a fine aggregate containing rhyolite at close to its pessimum proportion. Incidences of AAR detected in other concretes from the area were minor and very localised. They may have been caused by alkalis released by andesite particles, particularly since one such concrete contained pozzolan. Overall, a concrete alkali content of 2.0 kg/m^3 or less appears to be required to prevent AAR with the rhyolitic material, but no clear critical alkali content was evident for the andesites.

The Waikato/South Auckland investigation showed that AAR was restricted to structures which contained sand from the Waikato River and were built during a period when a high alkali cement was available in the area. Most also contained basalt coarse aggregate and some contained basalt crusher dust. The incidence of AAR in the samples analysed suggests that even if the cement does not provide enough alkali, the alkali from the basalt could provide the balance required for AAR to proceed, as indicated by laboratory investigations described elsewhere in these proceedings (Goguel 1996). Work to establish critical alkali levels for Waikato River sand is continuing.

PREDICTING FUTURE EXPANSION IN AFFECTED CONCRETES

In New Zealand few cases of AAR have warranted the expense of structural analysis and detailed site monitoring. A more simple method of predicting future expansion is needed so that maintenance decisions can be based on physical evidence rather than speculation.

In 1989 when the investigation was conceived, the most widely used method of predicting the likelihood of further expansion was to measure the accelerated expansion of core samples as described by the British Cement Association (1988). This investigation sought to find out whether such expansion tests would give results which represented qualitatively what was happening to the original structure. The nature of accelerated tests on core samples means it is reasonable to assume that if a core does not expand in the laboratory then the concrete is unlikely to expand in situ, but precludes an accurate quantitative relationship. Should the tests produce significant expansion then in situ monitoring would be recommended.

Selection of test concretes

Six structures were selected for monitoring. Three, in the Tongariro region, had extensively cracked abutments and thus represented the degree of damage most likely to prompt a monitoring programme. The other three were in Taranaki and had minor cracking on the abutments, which were more heavily reinforced than those in Tongariro. Monitoring these concretes would thus test the method on small amounts of in situ expansion.

Laboratory testing

Core samples were taken from the six structures in February 1991. Demec studs were attached along one gauge length representing the interior of the concrete on each side of the core. After being instrumented, the cores were wrapped in polythene bags to prevent further drying and stored for several months to let them equilibrate after being removed from the restraints of the surrounding concrete. They were air dry when put into the test conditions.

The storage conditions specified by the BCA's (1988) method were modified to avoid leaching alkalis from the specimens. Instead, the cores were stored in plastic bags containing about 20 ml of water, with the original outer surface of the core in contact with the water. There was enough space around the core for air (and water vapour) to circulate. Initially each core was wrapped in damp cotton cloth as subsequently recommended by the BCA's modified method (1992) but the cloths soon started to rot in the alkaline conditions and they were discarded after three weeks. This meant that some alkali was removed from the system. The plastic bags containing the cores were stored in waterproof containers immersed in water baths operating at 40 ±2°C throughout the test period.

Gauge lengths were measured approximately every three months. Before each length measurement the cores were cooled to 21 ±2°C overnight. They were also weighed at every measurement. Water was added to the bag if needed. Many of the plastic bags leaked, resulting in further loss of alkali from the test environment.

At 14 months an equipment failure resulted in the water baths overheating to 95°C during a weekend. The cores were measured and weighed after being allowed to cool, then were left at 21 ±2°C for four weeks while the damaged equipment was repaired.

In situ monitoring

Demec studs were mounted over several cracks on abutments of five of the bridges. At each demec site three or four studs straddled two or more intersecting cracks with each gauge length approximately perpendicular to the crack. This arrangement was designed to exaggerate movement, making it easier to detect. Gauge lengths were measured with a vernier calliper fitted with demec points to allow greater flexibility in gauge length. The calliper was fitted with a low precision dial gauge so that it would not detect the small movements caused by moisture movement or thermal effects. Measurements were taken in the same month each year from 1991.

Results

The in situ expansion measurements and core expansions are given in Table 1.

All cores expanded rapidly in the first eight days due to moisture uptake. Records of weight change showed that the specimens were generally taking up moisture consistently throughout the test period, but occasional surface drying on the upper part of the core suggested that moisture distribution in the plastic bag was not even.

None of the monitoring sites on the Taranaki bridges exhibited consistent expansion during the first four years so the expansions of the cores from these bridges can be taken to represent concrete which is not expanding in situ.

Core samples were taken near six of the monitoring sites on the Tongariro bridges. Two of these, T2 and TW2, had expanded more than 1000 microstrain after two years and both corresponded to sites where in situ expansions were detected on more than one gauge length over four years. The two cores from each bridge represent the concrete from the same part of the structure. Therefore it is reasonable to assume that

Table 1 In situ and accelerated core expansions

Bridge		In Situ Crack Widths							Accelerated Net Core Expansions (Inner Gauge Lengths)			
		In Situ Site	Increase in Gauge Length after Four Years (cm*)						Nearest Core Site	Net Expansion (microstrain)**		
			AB	AC	AD	BC	BD	CD		711 Days	1053 Days	1333 Days
TARANAKI	Oakura	1	0	0	0	0	0	0	) Oa 2b	235	228	228
									) Oa 3	162	177	207
	Waiongana	1	0	0		0			)			
		2	0	0		0			) Wg 2	70	84	21
		3	0	0		0			)			
	Waitara	No in situ measurements							Wr 3	182	196	168
TONGARIRO	Tokaanu	1	0	0.020	0	0	0	0	-	-	-	-
		2	0.015	0.025		0.030			T 2	1561	2247	2506
		3	0	0		0			T 4	441	861	1134
		4	0	0		0			-	-	-	-
	Te Whaiau	1	0.010	0	0	0	0	0	-	-	-	-
		2	0.015	0	0	0	0	0	TW 1	706***	842***	888***
		3	0.020	0.010	0	0.010	0	0	TW 2	1372	1939	2268
		4	0	0	0.010	0	0.010	0	-	-	-	-
		5	0.020	0.015	0	0	0	0	-	-	-	-
	Wanganui	1	0	0		0			-	-	-	-
		2	0.020	0.010		0			W 2	658	721	812
		3	0	0		0			W 3	644	882	1071
		4	0	0		0			-	-	-	-

* Smallest division on dial gauge was 0.01 cm. Gauge lengths ranged from 8 to 24 cm, with most between 10 and 20 cm.

** Excludes initial 8 day expansion, which was attributed to moisture uptake.

*** This core rapidly expanded some 2000 microstrain shortly after the equipment failure. The expansion quoted excludes this rapid expansion.

The reactive aggregates are andesite in the Taranaki bridges, and rhyolitic material in the Tongariro region.

the expansions of all the cores from the Tokaanu and Te Whaiau Bridges represent concrete which is expanding in situ. The expansions of the cores from the Wanganui bridge are of similar magnitude to TW1 and T4 so were assumed to indicate in situ expansion even though only one site recorded in situ expansion.

These results suggest that core expansions greater than 700 microstrain at three years could indicate an in situ expansion. The likelihood of the correlation increases with the magnitude of the core expansion: the larger expansions were in cores from bridges where expansion was recorded at more than one location. Thus core expansions greater than 1000 microstrain after two years can probably also be taken to indicate in situ expansion. However, the slow expansion of core Tokaanu 4 suggests that three years is a minimum test period for the conditions used here.

The variation between cores from the same concrete demonstrates how important it is to take more than one core sample from a concrete for accelerated expansion testing.

EFFECTIVENESS OF SURFACE TREATMENTS FOR REDUCING EXPANSION

Selection of test concretes

Prestressed beams were removed from a bridge in 1988 because of cracking caused by AAR. The beams were recovered intact and stored near the site to simulate the original exposure conditions and enable the beams to be used for research.

Initial attempts to examine the effectiveness of coatings to reduce expansion involved treating the beams and monitoring changes in dimension with surveying equipment. The results of this work were inconclusive because neither treated nor control beams expanded significantly (Freitag & Rowe. 1992).

It was then decided to store core samples from the beams in the accelerated conditions described above to determine whether the concrete had any potential for further expansion and, if so, whether the expansion could be reduced by a proprietary surface treatment (Freitag 1995). Core samples were thus taken from two of the beams in 1992.

Test parameters

The surface treatment tested was a proprietary system consisting of a silane-siloxane primer overcoated with an acrylic. This was applied to cores with diameters of 70 mm and 95 mm and lengths of 100-130 mm. Cores were coated in two moisture conditions: air dry and slightly drier than saturated surface dry (SSD). Twenty four specimens were tested, including 16 uncoated specimens used to distinguish the effects of core diameter, parent beam and moisture condition.

Results

Table 2 gives the results of weight change and expansion monitoring to three years.

Weight changes of air dried cores suggest that the treatment may provide a small degree of protection against the ingress of moisture from a humid environment (SSD coated cores gained more weight than the SSD controls because they were drier at the start of the exposure period). All cores expanded, but only five expanded more than 700 microstrain after three years. This supports the lack of overall in situ expansion observed in the earlier work. There was no significant difference between expansions of treated and untreated cores, but no treated cores had net expansions greater than 600 microstrain after three years.

The silane acrylic treatment therefore needs to be tested on an actively expanding concrete to determine whether its effect on deleterious expansion is significant, and whether the moisture condition of the concrete at the time the treatment is applied affects the ability of the treatment to reduce expansion.

Table 2 Weight increase and expansion of treated and untreated cores

Treatment	**Core**	**Initial Moisture Condition***	**Diameter (mm)**	**Weight Increase 966 Days (g)**	**Net Expansion 966 Days** (microstrain)**
Untreated	33	SSD	75	5	240
	34	SSD	75	3	120
	3	SSD	95	5	192
	7	SSD	95	6	302
	18	SSD	75	4	181
	20	SSD	75	2	874
	42	SSD	95	2	439
	43	SSD	95	3	755
	35	Air dry	75	18	407
	36	Air dry	75	20	257
	4	Air dry	95	32	389
	9	Air dry	95	29	229
	21	Air dry	75	17	3198
	22	Air dry	75	15	706
	44	Air dry	95	28	856
	45	Air dry	95	26	460
Silane-Acrylic	13	SSD	95	14	195
	5	SSD	95	16	330
	28	SSD	75	9	350
	29	SSD	75	9	579
	14	Air dry	95	21	381
	17	Air dry	95	25	417
	30	Air dry	75	13	456
	31	Air dry	75	16	371

* SSD = saturated surface dry (slightly drier than SSD in the case of the treated cores).
**Expansion at 966 days less expansion at 7 days, attributed to moisture uptake.

CONCLUSIONS

Examination of concretes from existing structures has identified reactive combinations of materials and mix designs, and use of this information will help to minimise the risk of AAR expansion in future construction. The investigation has also established a qualitative relationship between in situ and accelerated core expansions, which will enable the likelihood of future expansion of a damaged structure to be predicted. Attempts to evaluate the effectiveness of a surface treatment in reducing expansion were not completely successful because the subject concrete was found to be no longer actively expanding, but the results suggest that the material is worth examining further.

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ALKALI-SILICA REACTION;
READY-MIXED CONCRETE PRODUCERS' VIEW POINT

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ABSTRACT

The authors briefly discuss positive and negative effects of alkali in concrete. They draw attention to the fact that although the reliability of methods for estimating the possibility of alkali-silica reaction (ASR) is dubious, they are used for rejection of aggregates even when experience in service does not confirm the laboratory evaluation. This results in an unjustified increase in the cost of the projects. One of the possible resolutions of the problem is the setting up of database on ASR in different areas. The database will be used for making decisions on the potential reactivity of aggregates.
Keywords: alkali-silica reaction, concrete durability, ready-mixed concrete.

INTRODUCTION

The problem of ASR appears to be one of the most attractive in the field of concrete durability or even concrete technology judging from the number of publications and conferences held. But the problem is still far from resolution. "Unanswered questions remain", says the 'Guide to Durable Concrete' by the American Concrete Institute (ACI 201.2R-92). Courtier (1990) notes: "After all, he (an engineer) needs a solution now, problem solved - not as a continuing study".

POSITIVE AND NEGATIVE EFFECTS OF ALKALI

Swamy (1992) notes correctly that although aggregates are grouped for deleterious and innocuous, "it might even more realistic to say that perhaps and probably, there are no aggregates that are completely unreactive although they may remain, and appear to be so for a long, long time". Let us say that all alumina and silica aggregates react with alkalis, the difference is only in the alkali (NaOH or KOH) concentration, moisture and temperature conditions.

From the above paragraph, one cannot take the extreme view that all aggregates are deleterious. For instance, all portland cements with or without supplementary cementing materials will deteriorate with time (earlier or later) in sea water under unfavourable temperature conditions especially if this deterioration is accelerated by increasing the concentration of salts in laboratory testing. However, concrete is being used and will be used in marine structures.

Idorn and Roy (1985) report on the advantageous effect of alkalies on the rheology of concrete containing mineral admixtures. They also point out the effect in densifying the microstructure of the cement paste in hardened concrete. Wang and Gillott (1992) demonstrate the similarity of alkali-silica reaction and reaction of silica fume with calcium hydroxide. This also does not mean that engineers should go from one extreme to another by considering ASR as innocuous. In some cases, it results in cracks formation which leads to an increase in permeability, consequent corrosion of reinforcement and deterioration of structures (Mehta 1994).

DEFICIENCY OF TEST METHODS

ASR is a time-dependent phenomenon which could occur often after several decades of good performance (Swamy 1992). The reaction requires accelerated testing methods, to allow the prediction of concrete behaviour, in terms of ASR, in the future. ASTM C 33 [4] says: "A number of methods for detecting potential reactivity have been proposed. However, they do not provide quantitative information on the degree of reactivity to be expected or tolerated in service. Therefore, evaluation of potential reactivity of an aggregate should be based upon judgment and on the interpretation of test data and examination of concrete structures containing the combination of fine and coarse aggregates and cements for use in the new work."

Limitations of existing test methods on ASR are discussed in many different publications (ASTM C 33-93, Bérubé & Fournier 1992, Hooton 1994, Idorn *et al.* 1992, Diamond & Ong 1994, Mehta 1994, Tordoff 1990, others). In our opinion, among major limitations are the following:

- all criteria, which divide aggregates as innocuous and deleterious, are ad hoc and suitable only for types of aggregates within certain geographical regions (Bérubé & Fournier 1992)
- two main tools of acceleration of the reaction, namely, *(a)* an increase in alkalinity by addition of alkali to the mix water (Diamond & Ong 1994) and *(b)* an increase in temperature during specimens storage (Idorn *et al.* 1992 and Tordoff 1990) could result in incorrect evaluation. Diamond and Ong (1994) have found that the addition of alkali to augment alkali content in cements "does not result in the expected augmentation of the OH^- ion concentration, and much or all of the dissolved alkali hydroxide is immediately converted to alkali sulphate by reaction with gypsum." They conclude "that the ability to simulate a high alkali cement by adding alkali hydroxide to a low alkali cement are illusory." When the testing is carried out at elevated temperatures, Tordoff (1990) notes that "at the higher temperature the viscosity of the gel is lower and results suggest that this increased mobility more than offsets the increased tendency to crack induced by the more rapid rate of reaction."

HOW THE PROBLEM OF ASR IS BEING RESOLVED IN PRACTICE

Unfortunately, ASR confuses many engineers. They worry less about carbonation of concrete containing fly ash and slag or sulphate resistance of piles made from straight cement concrete (although sulphate attack is always deleterious in contrast to ASR) than they do about ASR. To resolve this problem they are satisfied with results of test methods which very often give an unreliable estimate. Sometimes, they are so *diligent* that they specify monthly testing on ASR. One wonders what they would do if in one month aggregate failed to be assessed as innocuous but the concrete had been already placed.

ADDITIONAL EXPENSES HAVE TO BE JUSTIFIED

Gerwick (1994) notes correctly that additional expenses could be needed for enhanced durability of major concrete structures. But this cannot lead to the conclusion that society is willing to spend any amount of money or just more money. Additional expenses have to be justified. Engineers, should evaluate:

- the consequences of the rejection of aggregates only on the basis of laboratory testing (whereas the field experience with this aggregate does not show problems) and increase in the cost of the project due to aggregate cartage from another source
- the consequences of the restrictions on alkali content in cements when supplementary cementing materials are used
- whether it is worth introducing additional frequency of testing for ASR and what to do with these results

etc.

We, engineers, have to be responsible to society for our decisions.

RECOMMENDATIONS

Since the test methods for the evaluation of potential reactivity have limitations, attention should be given to the detection of ASR in structures in service. The authors propose the organisation of a Commission on ASR under the jurisdiction of local concrete institutions (for instance, in Australia, the Concrete Institute of Australia) which will record (through consulting engineers, authorities and the concrete industry) and analyse cases of ASR in different areas of the country. This will help to systematise the cases of ASR, to establish relationships and to find out the most dangerous combinations. It is understood that the definite prediction of ASR in future structures on the basis of evaluation of ingredients and service conditions is subject to different opinions but in the case of recorded deteriorations in structures in service, ASR can and must be determined explicitly. Of course, judgement of new sources can only be made on the basis of laboratory testing (Shayan 1991).

When a decision to reject aggregates (which have been used in concrete for some years) in terms of potential ASR has been made only on the basis of laboratory results, engineers should submit evaluations of other solutions and prove that this is the cheapest one. Generally speaking, this approach should relate to all engineering decisions.

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ALKALI-AGGREGATE REACTIONS IN CHINA

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ABSTRACT

Some of gravel and crushed stones in China were found to be either alkali-silica reactive or alkali-dolomite reactive, or both. They involved igneous, sedimentary and metamorphic rocks. The reactive silica minerals were dominantly microcrystalline to cryptocrystalline quartz and chalcedony. Strained quartz, cristobalite, tridymite and opal also caused some rocks to be reactive. Alkali-aggregate reactions (AAR) have recently diagnosed in numerous buildings, water cooling towers, bridges, railway ties, piles and pavements of airport. To minimize the risk of AAR deterioration in the future structures, standards for evaluation of alkali-reactivity of aggregates and for maximum alkali content in concrete or cement have been proposed.

Keywords: Concrete structure, deterioration, preventing measures, reactive aggregates

INTRODUCTION

The study of alkali-aggregate reactions in China dates back to the early 1960's. Investigations were carried out using ASTM C289 and ASTM C227, and a few of rocks in Yangtse river and some other places were recognized as potentially alkali reactive (Xu et al. 1983 & Chen. 1984). Fly ashes were used to inhibit expansion due to alkali-silica reaction in concrete of dams (Chen. 1984). No case of damage caused by AAR was reported until 1989, when deterioration by AAR was first confirmed in railway ties (Han et al. 1990). In 1991, the first case of deterioration in China due to alkali-dolomite reaction (ADR) was discovered (Deng et al. 1993). Since then, rocks and sands from some quarries have been evaluated in detail. A large amount of rocks in rivers and mountains was found to be alkali reactive. Simultaneously, a number of concrete structures mainly in Northern China were recognized to be damaged by AAR. The reactive aggregates included andesite, siliceous dolostone, metaquartzite and dolomitic limestone etc.

Preventive measures are not generally adopted in construction. With the use of high volume of Portland cement containing high alkali and incorporating chemical admixtures such as antifreezing agent composed of alkali metal salt, some concrete structures in certain regions may be seriously affected by AAR in the near future.

REACTIVE ROCKS

Many types of rocks have proved to be alkali reactive in concrete structures and in laboratory tests. They involved igneous, sedimentary and metamorphic rocks. Table 1 shows alkali-silica reactive rocks found in Beijing, Jixian, Shuizhong, Jingzhou, Changchun, Pingdingshan, Linru, Shijiugang, Hangzhou, Guizhou, Chengdu, Hai river in Hebei, Hongshui river in Guangxi, Yangtse river, Han river and Tuo river (Han et al. 1990, Tamura & Huzikou. 1988, Zhen and Tang. 1982, Xu & Chen. 1983). The dominant reactive minerals in rocks were microcrystalline to cryptocrystalline quartz and chalcedony. The chalcedony made the rocks highly reactive. Cristobalite was a minor component of some andesitic rocks. Tridymite was found in one of perlites. Strained quartz was responsible for the reactivity of meta-granite and meta-quartzite, which may be classified as cataclastic rocks developed by dynamic metamorphism. The typical microstructures of these rocks are demonstrated in Fig. 1.

Table 1 Alkali-silica reactive rock found in China

Types	Reactive rocks	Reactive minerals
Igneous rock	Basalt, diorite porphyrite, quartz diorite-porphyrite, andesite, andesite porphyry, felsite, trachyte, rhyolite, obsidian, perlite, tuff, volcaniclastic rock	Microcrystalline to crypto-crystalline quartz, chalcedony, cristobalite, tridymite
Sedimentary rock	Chert, siliceous dolostone, siliceous limestone, mudstone	Microcrystalline to crypto-crystalline quartz, chalcedony
Metamorphic rock	Argillite, phyllite, meta-granite, meta-granite porphyry, mata-quartzite	Strained quartz, micro-crystalline quartz, chalcedony

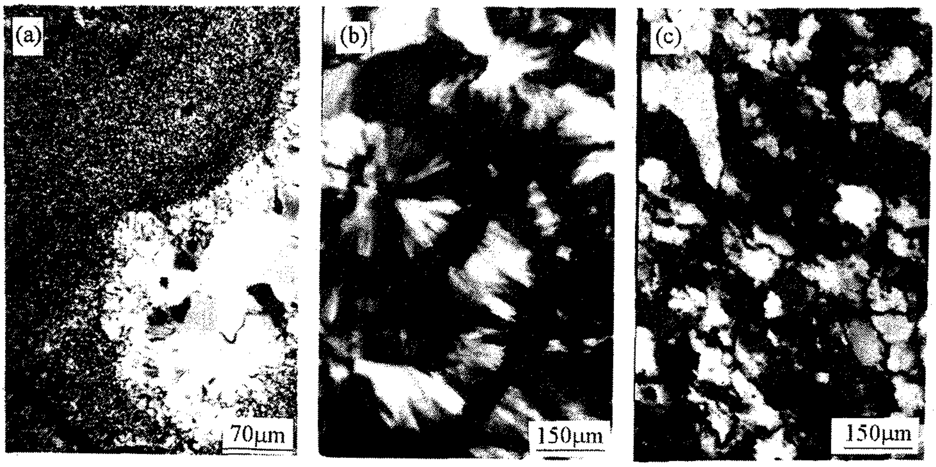
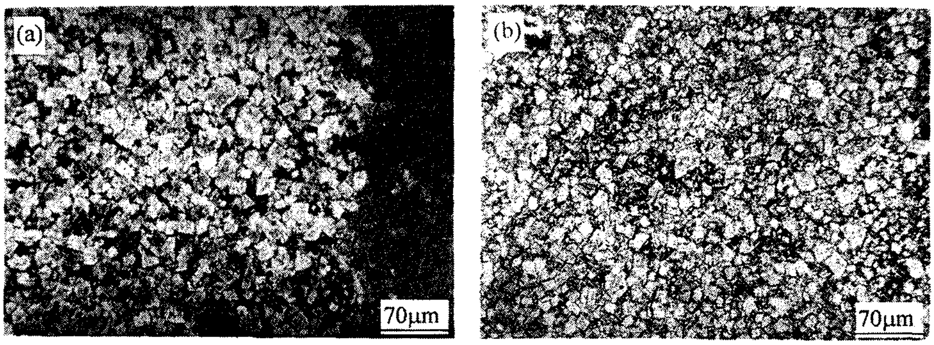


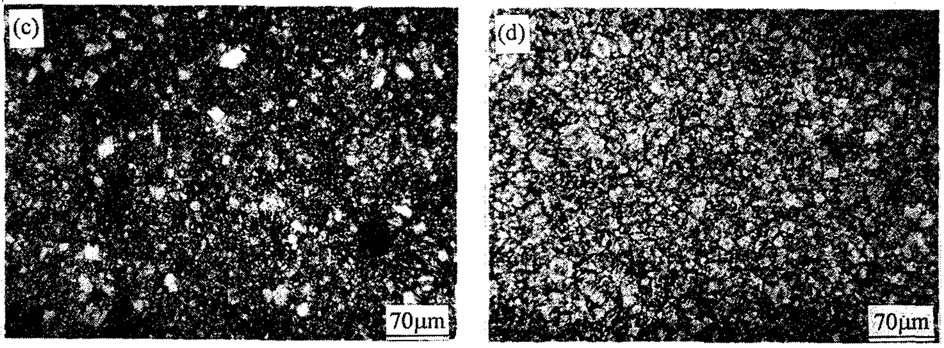
Fig. 1 The microstructures of alkali-silica reactive rocks

The alkali-dolomite reactive rocks included dolomitic limestone, calcitic dolostone, siliceous dolostone and dolostone from Taiyuan, Jixian, Huailou, Huailai, Nankou, Changchun, Pingdingshan, Lintong, Jinan and Weifang. Many of dolomitic limestones showed a texture of fine-grained (10-60 μm) dolomite rhombs scattered in matrix of calcite and a small amount of acid insoluble residue (less than 5%). The residue was composed of crystalline quartz, feldspar and clays. Dolomite in these rocks was distributed either in veins or homogeneously. These rocks showed intermediate expandability. Some dolomitic limestones with more impurities were highly expansive. The calcitic dolostones contained rhombhedral dolomite in 2-20 μm distributed in matrix of calcite, quartz and clays and were very expansive. Fine-grained dolomite and microcrystalline quartz or even chalcedony were involved in the siliceous dolostones. These rocks were founded being both alkali-dolomite reactive and alkali-silica reactive. When silica was homogeneously scattered in the fine dolomite crystals, rocks in concrete might cause a larger expansion. Argillaceous dolostones with fine dolomite were highly reactive. The pure fine-grained dolostones showed a mosaic texture and were expansive although the magnitude of expansion was slightly small. The representative microstructures of these rocks are shown in Fig. 2.



(a) Dolomitic limestone

(b) Calcitic dolostone



(c) Argillaceous dolostone

(d) Fine-grained dolostone

Fig. 2 The textures of alkali-dolomite reactive rocks

CASES OF DETERIORATION

To date it has been confirmed that buildings, water-cooling towers, bridges railway ties, piles and pavements of airport have cracked due to alkali-aggregate reactions. The affected structures by AAR may be more than the discovered cases because many deteriorated structures have not been subjected to investigation.

The west wall of the heat station in Beijing Internal Combustion Engine Plant was constructed with slag Portland cement in 1960. After suffered 40-50℃ and high humidity, the wall showed seriously random cracking. Petrographic examination revealed that the coarse aggregates in 5-30 mm contained microcrystalline quartz and typical chalcedony. Some aggregates were attacked and alkali-silica gel was formed (Han et al. 1990).

The concrete water-cooling towers in Tongliao Power Station, Inner Mongolia and Tiechangou Power Station, Xinjiang cracked after being in service for years. Reactive coarse aggregates with chalcedony and microcrystalline quartz were used. SEM/EDAX observation showed that some coarse aggregates were eroded and there was alkali-silica gel in the interface between aggregates and mortars. Little ettringite was found.

Bridges in Beijing and in railways in Jiangxi, Shandong and Hebei provinces were distressed by alkali-aggregate reaction. The bridges of Beijing were constructed during 1978-1986 and demonstrated map cracking on slabs, beams or piers in 1990. Investigations carried by Zhao et al (1993) and the authors established that cracking of the bridges was mainly caused by alkali-silica reaction between alkali in pore solutions and chalcedony as well as microcrystalline to cryptocrystalline quartz involved in the coarse aggregates basalt, andesite, siliceous dolostone etc. The railway bridges were precast and prestressed. The content of cement was 470-550 kg/m^3 concrete. The bridge beams were showing longitudinal cracking in the belly and bottom flange and random cracking in other parts. Field surveys taken during 1991-1992 revealed that, of 450 spans of bridges 8-15 year old, 405 spans were cracked. The cracked bridges were made with high alkali Portland cements and reactive coarse aggregates contained microcrystalline to cryptocrystalline quartz and/or chalcedony.

The prestressed railway ties manufactured in Beijing and Pingdingshan were showing map cracking beyond the rail seats and longitudinal cracking between the rail seat. The cracked on-line and spare (set beside) ties from Beijing were found in Guiyang, Shanghai, Beijing, Taiyuan and Northeast China. Alkali content of the ties was about 5 kg/m^3. The reactive aggregates were same as that used in the railway bridges from Beijing. Petrographic and SEM/EDAX examinations showed that the concrete was damaged by alkali-silica reaction. The railway ties from Pingdingshan contained reactive dolomitic limestone, calcitic dolostone and metaquartzite. They were made with a local high alkali (1.19% Na_2Oequiv.) Portland cement. The content of alkali was approximately 6.0 kg/m^3. These ties were confirmed to be affected by both alkali-dolomite reaction and alkali-silica reaction brought about by strained quartz.

Prestressed piles made in Taiyuan were longitudinally cracked after 1 year in service. The aggregates were derived from a river gravel source consisting of dolomitic limestone and dolostone. Coarse dolomitic limestone was found to be responsible for the distress. Piles in a concrete product plant and an internal combustion engine plant in Beijing were also showing longitudinal cracking. The piles were precast and prestressed. The materials used were same as that in the railway bridges or the wall mentioned above. Alkali-silica reaction was the main cause for cracking.

Concrete pavements of airport in China were also damaged by AAR. Table 2 lists some details about the pavements. The pavements showed map cracking. Dolomitic limestones were suggested as mainly detrimental component. Dolomite in these rocks was in 10-60 μm and was dispersed in matrix of calcite and acid insoluble residue. Laboratory tests indicated that the dolomitic limestones were alkali-dolomite reactive. Cracks in concrete were partially stretched out of dolomite-rich zones of the aggregates and extenuated to the surrounding mortars. The results of DTA and SEM/EDAX showed that the dolomitic limestone in JN airport was attacked and brucite was formed. The siliceous dolostones in TX airport were both alkali-silica reactive and alkali-dolomite reactive. Microcrystalline quartz and chalcedony involved in these rocks were responsible for their alkali-silica reactivity. Tuff in CC airport contained reactive microcrystalline quartz and caused deleterious expansion.

Table 2 Some details of the deteriorated airport pavements

Airport	Cement (kg/m^3)	Reactive aggregates	Date of construction	Damaged area (%)	Date of field survey
CC	330-350	Dolomitic limestone, tuff	1985-1986	20	1992
TX	310-320	Siliceous dolostone	1986	1.1	1992
LT	290-300	Dolomitic limestone	1975	6.0	1991
JN	310	Dolomitic limestone	1989	21.7	1991
WF	310	Dolomitic limestone	1983-1984	33.3	1991
XJ	300	Dolomitic limestone	1976	18	1989

PREVENTING MEASURES

ASTM C295, C227, C289 and C586 were incorporated to a standard (SD 105. 1982) of the Hydraulic and Power Ministry of China in 1982. They were employed to evaluate alkali-reactivity of aggregates. The standard formulated that low alkali cements (Na_2Oequiv. being not more than 0.60%) should be used or mineral admixtures should be applied to effectively inhibit the deleterious expansion when aggregates were alkali-silica reactive in hydraulic power stations. The alkali-carbonate reactive aggregates should not be selected as aggregates. Fly ashes have been used to prevent dams from being damaged by alkali-silica reaction (Chen. 1984). These measures have effectively made the hydraulic concrete structures not to be affected AAR. These ASTM standards were then transplanted to a standard (JGJ 53. 1993) on sand and rock of the Ministry by Construction in 1992. This standard claimed that only aggregates used by important concrete structures should be submitted to evaluate alkali-reactivity, and preventive measures mentioned above should only be applied to important structures. Practically, the part on AAR of this standard is rarely implemented.

Furthermore, a standard (CECS 48. 1993) was proposed by the authors, which concerned rapid determining of alkali reactivity of sands and rocks. This method was based on the Chinese autoclave method and was only suited to evaluate alkali-silica reactivity. This standard is extensively used by Chinese engineers and researchers.

High alkali Portland cements are now used in some concrete structures. The content of alkali is sometimes as high as 1.20% Na_2Oequiv. Chemical admixtures are universally used in China to improve workability, strength development or other properties of concrete. The commercial admixtures available in China often contain alkali metal salts such as Na_2SO_4 and $NaNO_2$. These salts may carry 0.8-13 kg/m^3 Na_2Oequiv. into concrete. Their combination tends to raise the alkali content of concrete being far higher than 3.0 kg/m^3. To avoid or minimize the risk of deterioration due to AAR, a standard (CECS 53. 1993) for maximum alkali content in concrete was proposed. When alkali-silica reactive aggregates are involved, the limit of alkali content in concrete or other measures shown in Table 3 are proposed. Many factors restrain the application of these measures.

Table 3 The limit of alkali in concrete with alkali-silica reactive aggregates and measures

Conditions of circumstances	Maximum alkali content (kg/m^3) in concrete and measure		
	Ordinary structure	Important structure	Special structure
Dry	no limit	no limit	3.0
Wet	3.5	3.0	2.1
With alkali	3.0*	unreactive aggregate	unreactive aggregate

* *The structures should be effectively painted, otherwise unreactive aggregates should be used.*

Based on the absences of low alkali Portland cements and nonreactive aggregates in North China, we are planing to use sulfoaluminate and ferroaluminate cements to counteract deterioration due to alkali-dolomite reactive aggregates. In addition, we are trying to produce a series of special cements to meet the demands for producing durable concrete with high strength.

CONCLUSION

Reactive aggregates are presented in many regions of China. Some concrete structures have been distressed by either alkali-silica reaction or alkali-dolomite reaction, or both. Many efforts will be made to minimize the risk of alkali-aggregate reactions in concrete. Deterioration due to AAR will be expected to be a continuing problem in China in the next ten years or more.

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THE FRENCH PREVENTIVE APPROACH TO AAR COMPARED TO EXPERIENCE

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ABSTRACT

The approach described in the "Recommendations for the prevention of damage by the alkali-aggregate reaction" and in the "Guide to the Preparation of Quarry Documentation" takes into account a set of data that are:

- related to the environment of a structure;
- specific to concrete;
- and specific to aggregates.

When the use of aggregates rated as non-reactive is recommended for the building of a structure (case of exceptional structures) or when, for a routine structure, knowledge of the materials used is requested, does the qualification procedure developed, which includes a petrographic determination followed by a battery of tests, yield a reliable diagnosis?

We answer this question by comparing the diagnosis and the behaviour of an existing structure for which both a sufficiently long period of observation and adequate documentation are available. It is also essential, for this comparison, to be able to procure exactly the same aggregates, and this was possible here. Each of the standardized tests has already been separately "calibrated" in this way, but it was also judged useful to compare the methodology as a whole to experience.

Our study shows:

- that the qualifying approach is relevant and accurately identifies risks;
- that the preventive approach is effective and suggests realistic solutions.

FOREWORD

One of the motive factors in the alkali-aggregate reaction, which may be defined schematically as the result of reactions reflecting cement-aggregate incompatibility, is the presence of alkalis in quantities exceeding some threshold. The recommendations published by the French Ministry of Equipment and Transportation indicates a threshold value of about 3 kg per cubic metre of concrete.

- In addition, observations both in France and abroad have shown that the reactions are more or less marked according to the type of aggregates, the type of cement, and also the ambient conditions, i.e. the environment of the concrete.
- Our choice, when the aim is to reduce the risk or make it acceptable, will therefore depend on these parameters. It will also depend on the level of risk one is prepared to accept. This will be determined by economic criteria, but also be influenced by psychological and social factors.
- The approach described in the "Recommendations for the prevention of damage by the alkali-aggregate reaction" (1) and in the "Guide to the Preparation of Quarry Documentation" takes this set of criteria into account.
- If, through ignorance, carelessness, or penny-pinching, the method were to be abridged, there would no longer be any assurance of prevention of the risks. For this reason, it is necessary to guard against pseudo-solutions, such as declaring a supply non-reactive on the basis of a test not chosen on grounds of petrographic knowledge as required by the approach recommended by AFNOR standard P 18-542 (2).
- On the other hand, performing the whole battery of tests would not enhance the level of protection against damage. It is, at most, the best way of making a bad decision. Similarly, the petrographic approach must not be taken too far. It must be adapted to the

materials to be used and to such knowledge as is available (or as one is entitled to demand) of the other constituents of the concrete.

BRIEF REVIEW OF THE PREVENTIVE APPROACH

a) Choice of level of prevention

This choice is based on an approach using objective criteria, such as the environment of the planned structure and its characterization, which includes its location, its strategic and economic importance, its size, its purpose, the constraints imposed by maintenance work, etc. This said, the Employer is responsible for the decision as to which category the structure belongs to.

Table 1. Level of prevention versus category of structure and exposure

Environmental class **Category of structure**	1 dry or not very damp	2 damp to wet	3 damp with frost and de-icing salts	4 maritime environment
I Slight risk acceptable	A	A	A	A
II Risk not very acceptable	A	B	B	B
III Risk unacceptable	C	C	C	C

- According to this decision, the document proposes one of three possible levels of prevention, A, B, or C. There are precautions specific to each level. All of this constitutes the preventive approach, described in more detail elsewhere

b) Associated precautions

Level A: No special precautions with respect to the alkali-aggregate reaction are necessary. The only requirements are the usual rules of construction.

Level B: In this case, which is the commonest (it applies to most civil engineering works), there are theoretically six possibilities allowing the use of potentially reactive aggregates. They allow the use of aggregates of all types, and satisfying the conditions of any one of the possibilities eliminates the risk.

Level C: In this case, non-reactive aggregates (NR), or else aggregates characterized as potentially reactive with pessimum effect (PRP), may be used in the concrete.

When there is no other possibility, the "Recommendations" document allows the use of potentially reactive aggregates in this category of structure, provided that the planned formulation is thoroughly studied on an experimental basis specified by contract.

The philosophy underlying this approach can be summed up in three points:

giving responsibility to the participants in construction;
making the best possible use of natural resources; and
building to last at the lowest possible cost.

The question now is, Is this approach reliable?

THE APPROACH COMPARED TO EXPERIENCE

Does the qualification procedure developed, which includes a petrographic determination followed by a battery of tests (flowchart 1), yield a reliable diagnosis?

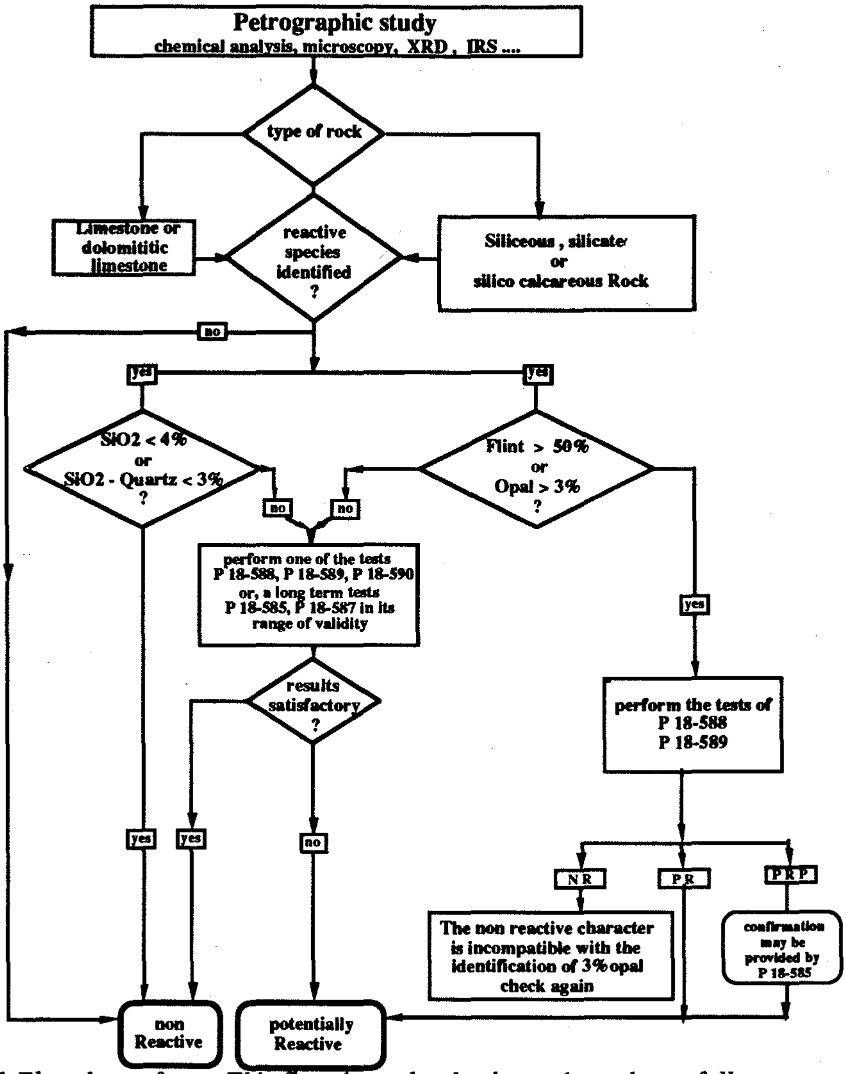

Fig 1 Flowchart of tests This flowchart clearly shows the order to follow to qualify a material with respect to the alkali-aggregate reaction ; first a petrographic study, then possibly a test chosen according to the petrographic diagnosis.

This question can be answered by comparing the diagnosis with performance on existing structures for which both a sufficiently long period of observation and adequate documentation are available. Each of the qualification tests of aggregates has already been "calibrated" in this way. The methodology as a whole, as described in the "Recommendations" document, also had to be compared to experience.
In all cases in which we were able to make the check, we found good agreement between the after-the-fact characterization and the diagnosis that could be established for the structure.
But it may happen that the conclusions are not so simple and obvious and that interpreting the combination of petrographic information and test results calls for reflection and experience, as the example that follows shows.

In a recent study, we had an opportunity to test the approach on a structure built just before 1940. This work had suffered damage sufficient to prompt the operating authority to request a study to find out why. The damage included but was not limited to progressive multidirectional cracking.

A series of cores taken from various parts of the structure in accordance with the methodology developed for this purpose led to a diagnosis of alkali-aggregate reaction, with no room for doubt.

The petrographic slices taken from the cores revealed the mineralogical composition of the aggregate, a rock made up of:

. - quartz, in crystals of various sizes exhibiting more or less corrosion and very often microcracks;
. - sparse alkaline feldspars, badly crazed (Sanidine);
. - plagioclases (oligoclase, andesine);
. - a vitreous phase, more or less abundant depending on the aggregates;
. - ferromagnesians (the commonest being biotite and chlorite).

Depending on the aggregate observed, the structure of the rock ranges from microlithic to microgranular. The phenocrysts (large and well crystalised minerals measuring several millimetres or centimetres) are represented by quartzes and plagioclases.

By its composition and structure, this material is classified in the family of rhyodacites (volcanic rocks intermediate between rhyolites strictly speaking, the igneous counterpart of granites, and dacites, the counterpart of quartzitic diorites). A few aggregates exhibit little or no quartz. They correspond to parts in which the excess silica is found in the vitreous mesostasis.

The documentation on the structure made it possible to identify with certainty the quarry that had provided the aggregates. This was a quarry opened specially for the job and unused since. The samples taken in the rock in place showed the same composition as already described. We completed our knowledge with a chemical analysis and proceeded to crush about a hundred kilograms of rock to perform the qualification tests, a performance test on the actual formula in conformity with the procedure described in the "Recommendations" document, and tests designed to ascertain the limit beyond which the formulation becomes reactive.

Chemical composition of the rhyodacite

SiO2	Al2O3	Fe2O3	CaO	MgO	SO3	Na2O	K2O	TiO2	MnO	P2O5	Pf	Total
69.67	13.80	2.47	2.9	0.73	0.06	2.15	4.65	0.11	0.12	0.11	3.47	99.93

The silica content of 70% is between a control rhyolite at 73-75% and a dacite at 63-68%. To go from a rhyolite to a rhyodacite, one must also find a smaller proportion of potassium feldspar, and so a smaller proportion of potassium (control rhyolite 5.30%; material of our study, 4.65%).

In short, the chemical analysis clearly shows that the material is rich in silica, and also that the alkali content is average for this type of rock and that there is roughly twice as much potassium as sodium.The sodium content as Na2O equivalent is:

$$0.658(K2O) + Na2O, = 5.20\%.$$

The test of release of active alkalis (4), performed on rocks having an identical structure and composition, showed that the quantity of active alkalis likely to go into solution in the concrete could be estimated at 300 grams per tonne of aggregates, or 600 g/m3 of concrete.

Petrography classifies the material in the rocks as probably reactive, with the reactivity resulting primarily from the more or less abundant vitreous phase and the corroded and microcracked quartzes. It would be tempting to say that the rock possesses a slow reactivity.

Given this diagnosis, the possible qualification tests, among the rapid tests, are:

- test P 18-588, the Microbar test;
- test P 18-589, the kinetic test;
- test P 18-590, the autoclaving test.

The Microbar test rated the material as potentially reactive, with a value of 0.12%, just above the critical threshold of 0.11%.

In an actual case, given the good agreement between the petrography and the test, we would have stopped there and qualified the materials as

- Potentially reactive.

Since our aim was to validate the methodology, we attempted to confirm this diagnosis. Test P 18-590 also placed the material in the category of aggregates that are

- Potentially reactive.

Test P 18-589, for its part, placed the aggregates in the zone of materials that are

- Non-reactive.

This contrast between the kinetic test and the other two tests is not surprising. We have already seen that the material is heterogeneous in terms of the constituents qualified by petrography as reactive. And the values given by the tests based on dimensional variations are just above the critical thresholds. Finally, the kinetic test, otherwise often good, is not ideal for materials having a slow kinetics.

Summary table of results of characterization of the aggregates

Test	Results
Petrographic examination	rock: rhyodacite reactive species: yes - glass more or less in stage of devitrification ·································>PR -corroded quartzes, with or without undulatory extinctions
Chemical analysis	SiO2 approx 70%
Test P 18 589 (chemical kinetics)	Result NR; disagrees with petrographic examination
Test P 18 588 (rapid test of dimensional variation)	Result PR; agrees with petrographic examination; test on which decision is based
Test P 18 590 (autoclaving - rapid test of dimensional variation)	Result PR; confirms test above and petrographic examination
Examination of concrete of structure	multidirectional cracking and alkali-aggregate reaction products

In any case, if there is disagreement between the petrography and the diagnostic test chosen, the approach calls for performing a second test belonging to a different group. We see that in this case the diagnosis would have been: - Potentially reactive -
Given the character of the structure (it would be classified in category C), the approach would lead to rejection of the aggregates or, if no other source could be found, to the requirement of a mix design study providing guarantees regarded as adequate and accepted by contract. The procedure currently considered most dependable consists of a series of dimensional variation tests at increasing alkali contents, for example

2, 2.5, 3, 3.5, 4, 4.5, 5, 5.5, 6, ... kg/m3

The duration and threshold might be 8 months, as in test P 18 587, at a temperature of 38°C, or 5 months, as in the performance test, at a temperature of 60°C, or even some longer period.

This approach may determine the alkali content above which the mix design becomes sensitive. Using a mix design in which the level is lower, with an adequate but realistic margin, should forestall the risk. This way of proceeding is long and constraining, but for exceptional works, for which in principle the necessary time can be taken, it can be applied profitably.

- To take the approach to its conclusion and arrive at a realistic evaluation, we carried out a concrete performance test. This test was performed using an alkali content of close to 1% in the cement. In conformity with the rules of the performance test, as laid down in the "Recommendations" document, given the nature of the aggregates (a material having a slow kinetics), five months was the duration fixed for the test. Under the experimental conditions, the theoretical total quantity of alkalis was the quantity contributed by the cement, 4 kg/m^3.

Under these conditions, the dimensional variation of the mix design was less than the 0.02% threshold at 5 months, as shown by figure 2.

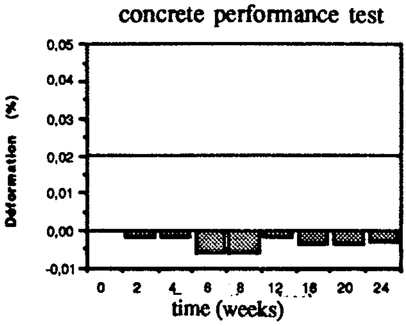

Fig 2. Dimensional variations at an alkali content of 4 kg/m3

The alkalis from the aggregates, with a theoretical quantity of not more than 0.6 kg/m3, are characterized by very slow release under the curing conditions of the test, which explains why they cannot contribute to the total content even though they in fact exist in the concrete of the structure.

- The level that must be reached to damage the concrete is 6 kg of alkalis per cubic meter of concrete, a level that may in fact be reached locally in the structure. In the case of a structure still to be built, prudence would call for limiting strictly the alkali content to the values proposed for category B structures in the "Recommendations on prevention of the risks arising from the alkali-aggregate reaction".

In the case of a cement content is 400 kg/m3, this limitation imposes a cement containing 0.6% alkali equivalent ((4 x 0.6) + 0.6) = 3 kg/m3.

- What trust can be placed in the preventive method?

We are already in a position to state that the proposed tests and methodology in fact qualify materials with respect to the alkali-aggregate reaction. Our approach still requires improvement when economic conditions make it necessary to use potentially reactive aggregates having a slow kinetics in works that are to be classified in category C. Currently, one approach that is elegant, dependable, but slow, consists of performing tests at 38°C or 60°C with different alkali contents to be able to propose a limit above which the reaction will appear. These tests, used to judge the future of the structure, must be interpreted prudently, especially as regards the quantity of alkalis that may be released, and therefore the quantity of alkalis acceptable in the formulation.

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Diagnosis of Alkali-Aggregate Reaction in Concrete

DIAGNOSIS OF THE CAUSE OF CRACKING IN FOUR STRUCTURES IN WHICH ASR IS OCCURRING

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ABSTRACT

Four examples are considered of the diagnosis of the primary cause of visual cracking in concrete within which alkali-silica reaction has occurred: a dam, a reinforced beam, a prestressed beam and a wing wall. In the first, it is shown that frost attack is the likely primary cause of the visual cracking, in the second, alkali-silica reaction, in the third, thermal shock and in the fourth, 'delayed ettringite formation'.

Keywords: Alkali-silica reaction, delayed ettringite formation, frost attack, thermal shock.

INTRODUCTION

Often when alkali-silica reaction occurs, concrete has sufficient strength to resist the deleterious effects of the reaction, either because the available 'alkali' content is too low or because the reactive silica content is low or well above the pessimum. Gel-filled fine cracks, and aggregate particles which have cracked as a result of ASR can therefore be found in sound concretes, in concretes which exhibit visual cracking due to ASR, and in concretes which exhibit cracking due to other causes. As a consequence, the identification of ASR in a petrographic examination of a single representative thin section of a suspect concrete does not enable a judgement to be made that ASR is the primary cause of any visual cracking (Diagnosis Working party, 1992). To establish that ASR is likely to be the cause of the visual cracking it is necessary firstly, to rule out other causes of such cracking, secondly, to establish that expansion has occurred, thirdly, to establish by thin section examination that there is considerable evidence of alkali-silica reactivity within the concrete and fourthly, to establish that the internal crack distribution is characteristic of that induced by ASR (Diagnosis Working Party, 1992). Fig. 1 shows the characteristic internal crack pattern induced by the reaction. This crack pattern has been observed both in laboratory concretes and field concretes adversely affected by ASR. When abnormal expansion is induced by ASR, a network of fine cracks connecting cracked aggregate particles is formed in the heart of the concrete. The cracks are frequently filled or partially filled with gel, and in some cases, also partially filled with ettringite. In the exposed surface region of the concrete the reaction is of lower intensity and consequently, the exposed surface layers of the concrete restrain the expansion of the heart concrete resulting in a tensile stress in the surface layers and, if the intensity of the reaction is sufficient, macro-cracks are induced at right angles to the exposed surface of the concrete member. If some of the cracks in a section of a suspect concrete are filled with gel and the internal crack distribution is typical of that induced by ASR, then ASR is likely to be a major cause of the deterioration (Diagnosis Working Party, 1992).

Four examples are considered of the diagnosis of the primary cause of visual cracking in concretes in which ASR has occurred: a dam, a reinforced beam, a prestressed beam and a wing wall. In the first, it is shown that frost attack is the likely primary cause of the visual cracking, in the second, alkali-silica reaction, in the third, thermal shock and in the fourth, 'delayed ettringite formation'.

A DAM

In 1989 BCA was asked to investigate the cause of cracking in a 40- to 50-year old dam in the South of Scotland. In 1976 it had been concluded that the cracking in most of the exposed parts of the dam may have resulted from an alkali-aggregate reaction. The aggregate employed was a greywacke. The dam has a concrete core wall with rock fill either side. The surface of the exposed part of the concrete core wall exhibited cracking broadly of two types: firstly cracks up to 10 mm in width, probably associated with a number of the construction joints and sometimes intermediate between visible construction joints, and secondly, random cracking of varying levels of intensity orientated primarily in the horizontal direction. Tapping the concrete in areas of random cracking indicated that internal cracks were present parallel and close to the surface of the concrete. Such cracking is characteristic of that induced by freeze-thaw attack.

Test programme

If both ASR and frost had contributed to the visual cracking in the exposed parts of the core wall then characteristic cracking due to ASR would be expected in concrete below the rock fill level. As a consequence, pits were dug to a depth of 2 m either side of the core wall to enable examinations to be made of parts of the wall unaffected by freeze-thaw attack. No visual ASR-type cracking was observed, the concrete exposed in the pits being uncracked apart from one fine crack associated with a construction joint.

Fig. 1 Internal crack pattern which can be caused by ASR: reactive silica content in the coarser end of the sand fraction.

Table 1: Summary of observations made on dam and test results

Observation or test	Comment	Is the observation of test result compatible with : ASR cracking?	Freeze-thaw cracking?
Cracking above rock fill level	Some spalling, cracks parallel to exposed face of wall.	no	yes
Cracking below rock fill level	Concrete in good condition, very limited cracking	no	yes
Condition of cores above rock fill level.	Cracks parallel to surface, decreasing in intensity as the distance from surface increases.	no	yes
Condition of core below rock fill level	No visible cracking.	no	yes
ASR gel on surface of cores	Few air pockets filled or partially filled with gel.	unlikely	-
Crack distribution under UV light (section from above rock fill level). Core depth ≤ 360 mm	Inter-connected system of random cracks in 'heart' of concrete. Also cracks roughly parallel to exposed surface. Expansion has occurred.	no	yes
Crack distribution under UV light (section from below rock fill level)	Cracking of significantly lower intensity than in concrete above rock fill level. Fine cracks not interconnected. Expansion has not occurred.	no	yes
Thin section examination (concrete from above rock fill level).	Concentration of ASR gel low compared to concretes which are known to have cracked due to ASR. A number of air pockets and cracks lined with gel (approximately 1/5th)	unlikely	yes
Thin section examination (concrete from below rock fill level).	Concentration of ASR gel low. A number of air pockets and cracks lined with gel (approximately 1/5th).	unlikely	yes
Expansion of core sections maintained moist.	Small expansion, 0.01-0.06%	possibly	-
Expansion of core sections in a caustic solution.	Further ASR gel formation, small contraction.	no	-
Shrinkage of core sections, 20°C, 65% RH	High shrinkage for sections taken from above and below rock fill level, 0.07 to 0.10 %.	possibly	possibly
Freeze-thaw resistance (section from above rock fill level), prism size 95x95x311 mm. ASTM C666 (Procedure A).	Low freeze-thaw resistance.	possibly	yes

Six cores were taken from the upstream face of the dam, including one taken 1.5 to 2 m below the top of the rock fill level. The following tests were carried out on sections of the cores: Internal crack distribution on sections impregnated with a resin containing a fluorescent dye and ground to achieve a flat surface. Examination of thin sections using a petrological microscope under plane and polarised illumination. Expansion of core sections maintained moist at 20 °C and immersed in a 1-M sodium hydroxide solution. Shrinkage tests. Freeze-thaw tests.

Summary of main observations and test results

The field and test observations are summarized in Table 1. Photographs of the internal crack distribution in a core taken from above the rock fill and in a section of core taken from below the rock fill level, are shown in Figs. 2 and 3 respectively. In the final two columns of Table 1, the question is considered as to whether or not the observations or test results are compatible with alkali-silica reaction or freeze-thaw attack being the mechanism which has led to the general deterioration of the concrete core wall above the rock fill level. It is concluded that the deterioration of the concrete in the exposed part of the core wall is due primarily to freeze-thaw attack and that ASR is unlikely to be a secondary cause of deterioration.

A REINFORCED BEAM

In the late 1970's it was concluded that a number of concrete structures in the South West of the UK had cracked as a consequence of ASR. The fine aggregate employed was a washed sea-dredged sand and the coarse aggregate a low-porosity limestone or granite. The cement used had an alkali content, at the time the concretes were cast, of between 1.05 and 1.4% by mass. In the 70's the judgement was made that ASR was the cause of deterioration because gel and cracked aggregate particles were observed in thin section. It is now known that such observations are not sufficient conditions for concluding that ASR is the cause of the visual cracking (Diagnosis Working Party, 1988 and 1992).

Table 2 summarizes the field and test observations made on more recent sections taken from one of the beams, where in the 70's the judgement had been made that ASR was the cause of the visual cracking. In the final column, a judgement is made as to whether or not each observation is compatible with ASR being the cause of the visual cracking. It is concluded that ASR is probably the cause of the visual cracking.

A PRESTRESSED BEAM

In 1974 four T-shaped prestressed beams manufactured in the North of England were rejected by a UK County Council on the grounds that the steel cover was inadequate. These beams had been cast in March 1974, subject to a steam cure for 16 hours prior to stressing and then at an age of about 24 hours stored in the Precaster's yard. The materials used in the beams were a RHPC with an alkali content of about 0.9% by mass, a siliceous limestone coarse aggregate and a natural sand. The cement content was in the range 450 to 500 kg/m^3.

An inspection of beams in the mid 1980's showed fine longitudinal cracking in the web and fine cracking in the lower flange. In 1989 tests were carried out by BCA to establish the possible cause of the visual cracking in the beams. Four 74 mm diameter cores were taken through the full width of the web of one of the T-shaped prestressed beams. Cores 1, 3 and 4 were 148 mm in length and core 2, which was taken through the shear region of the beam, was 245 mm in length. A visual examination of the cores indicated that they were of high quality and apart from core 2, no gel or white deposit was visible in the air pockets on the core surfaces. Two of the exposed air pockets on core 2 were partially filled with gel. From these observations it may be concluded that the intensity of the ASR reaction was low.

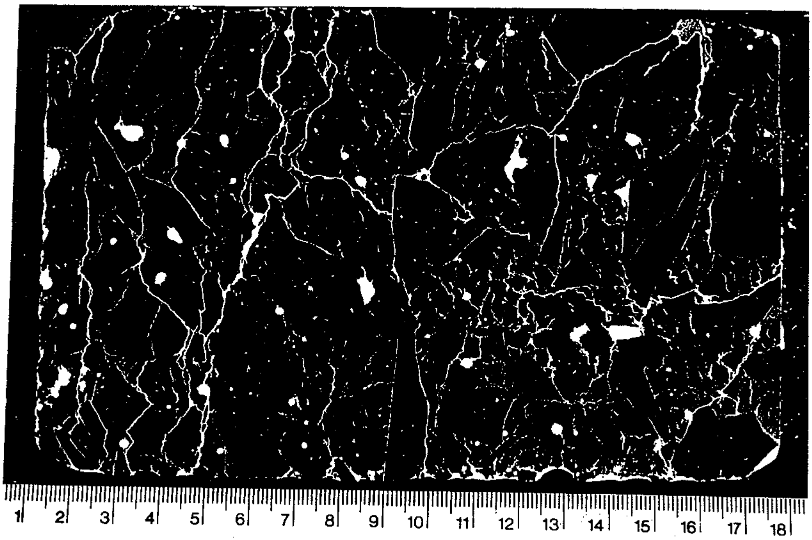

Fig. 2 Crack distribution in section taken from above rock fill level.

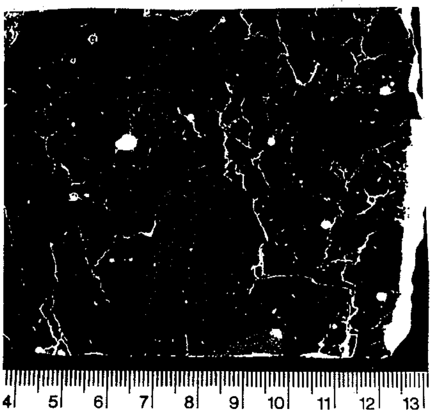

Fig. 3 Crack distribution in section taken from below rock fill level.

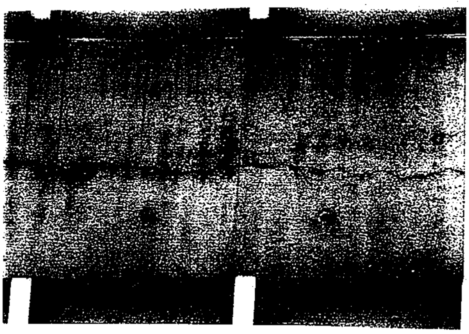

Fig. 4 Section of the exposed beam.

Table 2: Summary of observations. Reinforced beam

Observation	Comment	Is the observation compatible with ASR being the possible cause of the cracking?
Visual cracking (see Fig. 4)	Cracking characteristic of that induced by excessive shrinkage or expansion.	Yes
Age to cracking	< 5 to 6 years	Yes (Hobbs, 1994 and 1996)
Core examination	Macro-crack depth 15 to 45 mm, some gel filled air pockets, some cracked coarse sand particles.	Yes
Internal crack distribution (Fig. 5)	System of fine cracks interconnecting cracked aggregate particles in the coarser end of the sand fraction. Macro-cracks present but fine cracks absent in surface region (Fig. 6)	Yes (Diagnosis Working Party, 1992)
Thin section examination for gel	Isotropic gel, gel filled fine cracks, gel saturated paste.	Yes (Diagnosis Working Party, 1992)
Cement content	~ 470 kg/m^3	Yes (Hobbs, 1996)
Estimated original alkali content	4.9-6.6 kg/m^3	Yes (Hobbs, 1996)

Fig. 5 Section of affected core. Depth 40-57 mm (mm scale).

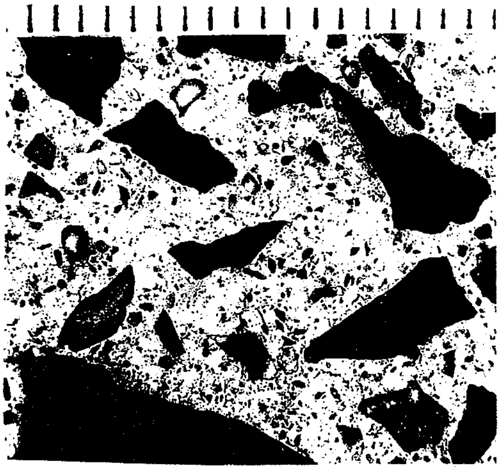

Fig. 6 Section of affected core. Depth 22-40 mm (mm scale).

Thin section examination

The petrographic examination of two thin sections, approximately 74 x 65 mm, prepared from a resin-impregnated section of core 2 showed the following: 1. Extensive fine-cracking with crack widths up to 40 μm but generally in the range 10-30 μm. 2. Fine cracks in the coarse aggregate and the cement matrix. These cracks skirted round the fine aggregate. 3. A proportion of fine cracks partially filled with ASR gel: the gel was present both within some limestone particles and the cement matrix. 4. Several voids, up to 400 μm in size, filled with ASR gel and several larger voids up to 3 mm in size rimmed with ASR gel. 5. Gel-saturated cement matrix close to some of the fine cracks which were partially filled with ASR gel. 6. An absence of reaction sites associated with the fine aggregate. 7. Small amounts of silica within some of the limestone particles in the form of small coarse-grained quartz crystals and lens-shaped inclusions of micro-crystalline quartz. 8. No evidence of reaction or cracking appeared to be associated with the visible silica present within the limestone particles. 9. No ettringite banding around aggregate particles. 10. Some secondary infilling of fine cracks by ettringite as single or groups of needle and tabular crystals.

From the thin section examination it was concluded, firstly, that the limestone coarse aggregate was the most probable source of reactive silica and secondly, assuming the sections examined were representative of the concrete, that ASR had probably not been of sufficient intensity to induce abnormal expansion and visual cracking.

Fine crack distribution

The fine cracking distribution visible under ultra-violet light in a full-width web section taken from core 2 and impregnated with a resin containing an ultra-violet sensitive dye is shown in Fig. 7. An examination of this figure shows that the fine cracks are uniformly distributed over each section. Little evidence of fine cracking is apparent within the limestone particles. Microscopic measurements showed that the width of the fine cracks was generally in the range 10 - 30 μm and that the 'macro-crack' crack visible on the surface of the web was approximately 50 μm in width.

A comparison of the fine crack widths in regions where it was suspected that ASR had occurred, with regions in which it was suspected that ASR had not occurred, showed no material difference. A comparison of Fig. 7 with Fig. 1 shows that the fine cracking in the web is untypical of that induced by ASR. The fine cracking has the appearance of being induced by a physical process, the limestone particles either expanding by more than the mortar matrix or alternatively contracting less than the mortar matrix. Such differential movement would induce tangential tensile stresses around the limestone particles and could, if the differential movement was of sufficient magnitude, induce the fine crack pattern shown in Fig. 7.

Limestone aggregate particles generally have a lower coefficient of thermal expansion than either cement paste or siliceous sand (Browne, 1972) and since the prestressed beam was steam cured shortly after casting, it is concluded that the fine cracks were probably induced by differential thermal contraction. Because of the uniformity of the fine crack distribution, it is also concluded that ASR had no material effect upon fine crack width or their distribution.

Conclusions

The following conclusions relate to the web of the particular steam-cured prestressed concrete beam from which cores were taken for examination:

1. ASR is neither a primary nor secondary cause of the fine cracking within the web or of the fine longitudinal cracking which is visible on the surface of the web.
2. The most probable cause of the longitudinal cracking on the exposed surface of the web is early age differential thermal contraction.

A WING WALL

In 1991 BCA carried out tests to investigate the cause of cracking in a section of a reinforced wing wall of a bridge in the South West of the UK . The cracked section was cast in the summer upon an uncracked section, probably cast 48 hours earlier.The cracks were first noted in 1985 when the bridge was about 11 years of age. The coarse aggregate was a limestone and the fine aggregate a natural sand. The cement content was about 480 kg/m^3 and since the wing wall was about 500 mm thick, it is probable that the early peak temperature within the concrete exceeded 80°C.

Three 94 mm diameter cores were taken from the wing wall to enable thin section and internal crack distribution examinations to be made. No evidence of ASR was observed on the surface of the cores, however, a white deposit could be seen lining some of the air voids.

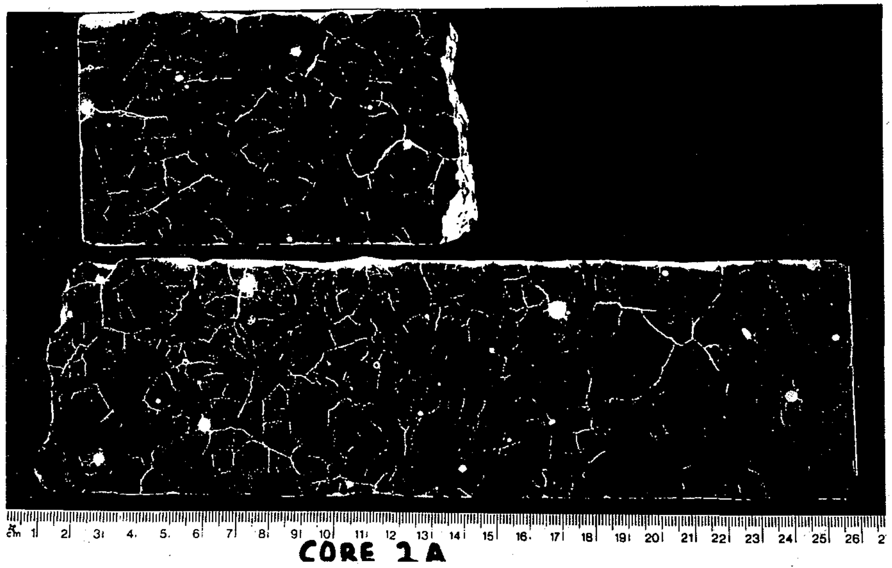

Fig. 7 Fine crack distribution in full width web section.

Thin section examinations

The petrographic examination of thin sections taken at a depth of about 400 mm from each of the cores showed the following:

- A high ferrite-phase Portland cement
- Fine cracking around the periphery of coarse aggregate particles and fine aggregate particles of up to 50 µm and 15 µm in width respectively (Fig. 8). These fine cracks were infilled, or partially infilled, with densely packed ettringite. Approximately 90 % of the coarse aggregate particles had periphery ettringite bands. Less than 50 % of the sand particles had periphery ettringite bands.
- The voids were rimmed with ettringite.
- Fine cracks were present running through the cement paste and skirting the sand particles.

In one thin section (Core 3), the following was also observed:

- A major macro-crack system, up to 100 µm in width, extending through the section and skirting the aggregate particles. It was judged that these cracks were induced by restraint to plastic settlement.
- One particle of cracked chert with associated gel (Fig. 9).

Fine crack distribution

An examination of the internal crack distribution in sections impregnated with a UV-sensitive dye showed the following:

Fig. 8 Ettringite band around limestone particle. Core 3. Magnification 280.

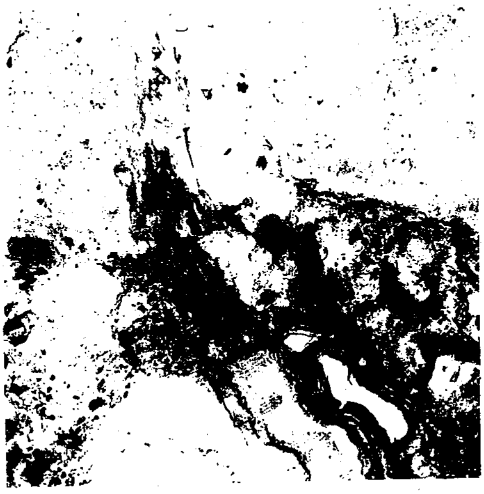

Fig. 9 Reacting chert particle showing cracking and ASR gel. Core 3. Magnification 210.

Cores 1 and 2: Discontinuous fine cracks together with some cracked coarse aggregate particles.

Core 3: A macro-crack at right angles to exposed surface of depth 73 mm, together with macro- and fine cracking inclined to the vertical axis. It is considered that this cracking was induced by restraint to plastic settlement.

Conclusions

In the following conclusions, it has been assumed that the sections examined were representative of the concrete in the wing wall:

1. The intensity of alkali-silica reaction is very low and would not have had an adverse effect upon the concrete in the wing wall.
2. The concrete in the wing wall has expanded as a consequence of 'delayed ettringite formation'.
3. The visual cracking in the wing wall is due to non-uniform expansion induced by 'delayed ettringite formation' (Hobbs, 1994).
4. It is estimated that the expansion induced by 'delayed ettringite formation' may be as high as 0.8 %.

ACKNOWLEDGEMENTS

The author is grateful to Mr R Tapp, previously of the British Cement Association, for carrying out the thin section examinations and for preparing ground sections for examination of the internal crack distribution within the concretes tested.

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CONCRETE DAMAGE CAUSED BY GREYWACKE REACTIONS - AN AAR?

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ABSTRACT

This paper discusses a damage mechanism in concrete produced with greywacke as an aggregate. The macroscopical phenomenon is characterized by network-like rifts on the surface of the concrete analogous to a well-known ASR. However, the special feature is that the greywacke grains damaged do not swell (e.g. like an opal containing sandstone), but they cleave. On each cleaving plane of the greywacke grain and also in pores near damaged greywacke grains a characteristic white substance is formed. This substance can be described as a strongly disordered layer silicate analogous to stilpnomelan (a transition mineral between hydrobiotite and montmorillonite). It can be argued that this silicate is probably a final product of a "weathering" process of plagioclases and micas inside the greywacke matrix. Most important is that greywacke itself can obviously provide all constituents of the damage reaction, what can be shown by specific concrete analysing methods. If this mechanism takes place in such a way it is different from other processes described in the literature as AAR until now.
Keywords:aggregate reaction, cleaving, greywacke, layer silicate formation, weathering

INTRODUCTION

In the sixties it was reported for the first time in Germany on damage at concrete buildings in which alkali ions of the cement used and aggregates with a high content of amorphous silica were involved. In a short time these processes were identified as a so-called Alkali-Silica-Reaction (ASR). In the meantime the reactions between cement components (alkali) and aggregates which form a concrete damaging alkali silicate, the conditions for the course of these reactions and the macroscopical phenomena of the damage are well-known (Dahms. 1994).

FEATURES OF THE CURRENT DAMAGE

In the area of Lower Lusatia in the eastern part of Germany in the last few years some cases of damage at concrete buildings (dams, roads) have been observed which do not match any classical alkali silica reaction described before and which have been well-known in Germany until now. According to this "classical" ASR the macroscopical phenomenon of the damaged concrete building is characterized by a network-like rift structure (see Fig. 1). But substantial differences to a "classical" ASR are:

- The concrete aggregates damaged are greywacke grains. Greywacke is sedimentary rock with crystalline mineral components.
- The age of this rock is about 600 million years (the amorphous substances formed a long time ago have recrystallized).
- Only some parts of the concrete building are damaged although the "same" greywacke aggregates were used in the whole building.

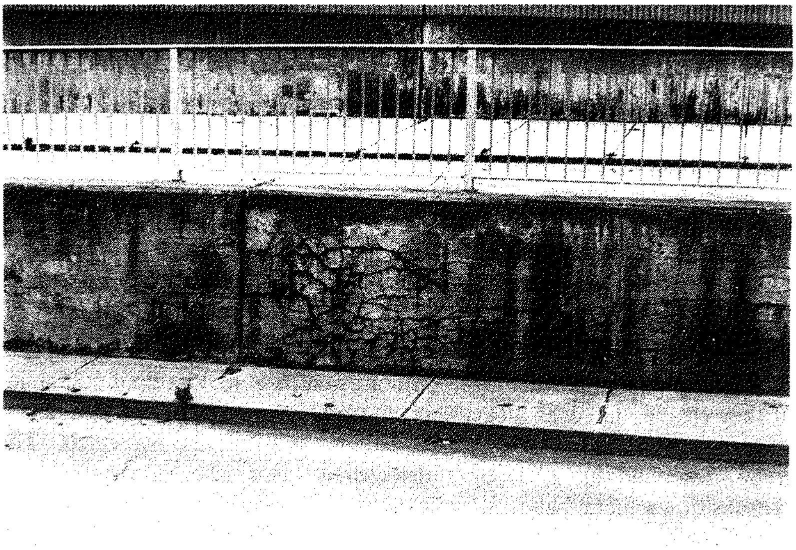


Fig. 1: Typical damage of the concrete

- Damage was also observed at low concentrations of alkali ions in the cement of the concrete.
- The aggregates damaged cleave. A swelling process on the surface of the aggregate grains was not observed.
- On **each** cleaving plane of the greywacke grains a characteristic white substance, formed as a seam, can be seen (see Fig.2).

Fig. 2: Typical structure on the cleaving plane of a greywacke grain

- Pores in the near of the damaged greywacke grains are also filled with a white substance. But this substance cannot be identified as Ettringite.
- The riftzone on the surface of the concrete is characterized by dirty brown crusts.
- Inside the greywacke aggregates rifts can be seen which do not have any continuation in the surrounding hardened cement.

Drilling sample investigations show that the destruction of the concrete is primarily a result of the damage of the greywacke grains. The damage mechanism can only be explained by investigation of the changes inside the greywacke grains and also by determination of the chemical composition and the structure of the white substance on the cleaving planes of the greywacke grains.

INVESTIGATIONS

Chemical analysis of the concrete

Several buildings were investigated (concrete wall, concrete road, dam) and from every building some drilling samples were gathered with the aim to determine the chemical composition of the concrete. The investigations were carried out according to two schemes:

a) analysis of the whole drilling sample or the whole concrete respectively, and
b) analysis of the concrete after sorting out the greywacke grains damaged.

Table1: Results of the chemical concrete analysis

	a) The whole concrete			b) Concrete without greywacke	
Constituents (wt %)	Crushed sample	Cement and soluble constituents of greywacke		Crushed sample	Oxide content of the pure cement
Loss of mass after heating	4.79			7.19	
Insoluble	71.60			65.83	
Soluble constituents		(values in parenthesis see text)			
SiO_2	5.05	21.39	(29.17)	5.82	21.57
Fe_2O_3	3.15	13.34	(18.19)	1.65	6.12
Al_2O_3	2.65	11.22	(15.30)	1.99	7.38
CaO	10.41	44.09	(60.23)	16.25	60.23
MgO	0.87	3.68	(5.02)	0.65	2.41
SO_3	0.58	2.46	(3.35)	0.81	3.00
Chloride	0.025	0.106	(0.15)	0.013	0.048
K_2O	0.49	2.08	(2.84)	0.15	0.56
Na_2O	0.07	0.30	(0.41)	0.03	0.11
Na_2O-eq.	0.39	1.65	(2.25)	0.13	0.48

This approach was necessary because it was not possible to get sufficient and useful information from the concrete analysis made according to scheme **a)** about the kind of the cement used, the alkali content and the content of the other oxides. A typical result of the chemical concrete analysis which represents the drilling sample investigated is shown in Table 1.

Analogous differences (more or less for the individual constituents) between both kinds of investigation shown in the table can be found out also by investigations of the other samples. Normal concrete does not show such differences. In order to discuss the results of Table 1 it can be seen that the whole concrete (scheme **a**) has a high alkali content, a high content of iron and aluminium oxide, but a very low content of calcium oxide. In contrast to this the concrete without greywacke grains (scheme **b**) has the same composition as a typical Portland cement with a low alkali content. This can not be because it is the same drilling sample.

These differences are important. The one and only conclusion from this is:

Certain constituents of greywacke were dissolved during the life of the building caused by the permanent influence of the largely basic environment of the concrete.

Where can these ions or oxides come from?

Greywacke used here as an aggregate consists of quartz, feldspars (plagioclases), micas (muscovite, biotite, sericite) and pyrite. Caused by the longterm permanent influence of the largely basic environment of the concrete it can be assumed that the ions come from micas and feldspars (SiO_2, Fe_2O_3, Al_2O_3, MgO, alkali ions) as well as pyrite (Fe_2O_3). Investigations of basaltic rock (Goguel. 1995) to determine the alkali solubility confirm this assumption.

Since the ions or oxides cannot be removed inside the closed system of the concrete and calcium oxide exists as supplementary oxide caused by the surrounding porewater of the hydrated cement, a new formation of minerals takes place inside or in the near of greywacke grains damaged. The results of these new formations of minerals are obviously the white substances which were found on the cleaving planes of the greywacke grains and also in the pores of the hardened cement stone.

Using a Leica scanning electron microscope (SEM) coupled with a Link energy dispersive X-Ray spectrometer (EDX) the chemical composition of the white substance located on the cleaving planes was measured. In the second column of Table 2 the results calculated as oxides are listed. These values are compared with the oxide composition dissolved from the greywacke (third column of Table 2). The calculation of these values based on the assumption that CaO does not exist as a soluble constituent of greywacke minerals. In the absence of limestone this proved to be a tenable assumption.

Considering the relations of the CaO contents determined by scheme a) and b) factor 1.36 results (the equation for this is: 60.23/44.09=1.36, see values in Table 1). The oxide concentration values of the whole concrete analysis were multiplied by this factor (except the CaO value) and the results are already shown in Table 1 as values in parenthesis. If you subtract the concentration values of the pure cement from the values in parenthesis you can get the concentrations of the constituents of greywacke which are available for the new formation reactions of minerals.

Tab. 2: Comparison of the EDX-results of the white substance with the soluble constituents of greywacke

Constituents	Oxide composition of the white substance inside greywacke grains (using EDX-detection) (wt %)	Oxide contents which are available from greywacke (for calculation see in the text) **
SiO_2	50.1	7.8
Fe_2O_3	7.8	12.1
Al_2O_3	5.4	7.8
CaO	7.6	0.0
K_2O	12.7	2.2
Na_2O	4.8	0.3
TiO_2	1.4	not investigated

** These values are the soluble content of 100g greywacke grains.

In Table 2 only the concentration ratios of the oxides can be compared, but not the absolute values. Very interesting is that the potassium oxide/silicium oxide-ratio is the same (about 0.3) measured according to both analysing schemes. The pore solution of the surrounding hardened cement paste consisting of $Ca(OH)_2$ yields the missing content of calcium oxide. The oversupply of iron oxide can obviously be found in the dirty brown crust on the surface of the concrete. This crust consists of iron oxide or iron hydroxide. The oversupply of aluminium oxide determined by the chemical investigation of the concrete cannot be explained at this time because in such high concentrations aluminium could not be found in the white substance on the cleaving planes of the greywacke grains and also not in the pores of the cement stone by using the EDX method. But also ettringite, the compound which can bind a high amount of aluminium, could not be proved in the pores of the hardened cement or could only be proved in traces.

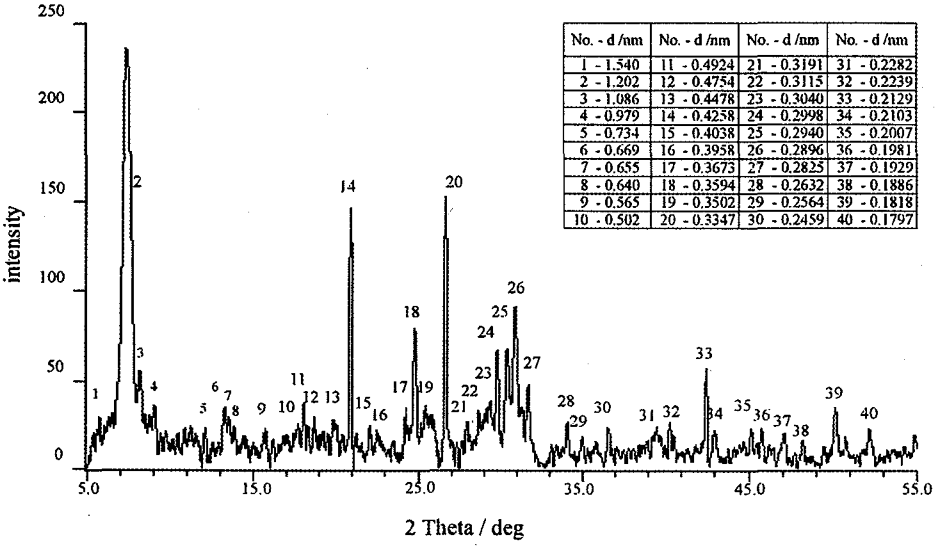
Structure investigations

X-ray analytical investigations show that both the substance of the cleaving planes and the substance inside the pores of the cement stone are the same. Fig. 3 shows a typical X-ray diffractometer pattern of a sample of the white substance. Obviously, this is at least a partially crystalline material which differs to a high degree from the amorphous alkali silicates found in Germany until now.

At the moment an identification of the white substances based only on the X-ray analytical or the EDX-investigation results is not possible.

Therefore we tried to convert the chemical composition results measured by using the EDX detection into a structure formula. These results are listed in Table 3.

The result of this calculation can be interpreted as an anion complex with the formula $[Al_{0.4}Si_{3.6}O_{10}]^{-4.4}$. This is a typical structure of a three-layer silicate. The numbers of oxygen atoms (13.5) resulting from the calculation (see Table 3) can spread over the tetrahedral layer of this silicate (here 10), the octahedral layer as hydroxide and the interstratified layer as water molecules. The cations Ca^{2+}, K^{+}, Na^{+}, $Fe^{2+/3+}$ and Ti^{4+} compensate the charges in the octahedral layer and also in the interstratified layer of this layer silicate.



No. - d /nm	No. - d /nm	No. - d /nm	No. - d /nm
1 - 1.540	11 - 0.4924	21 - 0.3191	31 - 0.2282
2 - 1.202	12 - 0.4754	22 - 0.3115	32 - 0.2239
3 - 1.086	13 - 0.4478	23 - 0.3040	33 - 0.2129
4 - 0.979	14 - 0.4258	24 - 0.2998	34 - 0.2103
5 - 0.734	15 - 0.4038	25 - 0.2940	35 - 0.2007
6 - 0.669	16 - 0.3958	26 - 0.2896	36 - 0.1981
7 - 0.655	17 - 0.3673	27 - 0.2825	37 - 0.1929
8 - 0.640	18 - 0.3594	28 - 0.2632	38 - 0.1886
9 - 0.565	19 - 0.3502	29 - 0.2564	39 - 0.1818
10 - 0.502	20 - 0.3347	30 - 0.2459	40 - 0.1797

Fig.3.: X-ray diffractometer pattern and d-values of the white substance of the cleaving planes transmission geometry - capillary technique (radiation: Cu K_α)

Tab.3: Calculation of a possible silicate structure based on the EDX results

Constituents	(Wt %)	(Mol)	Calculated to 12 oxygen atoms	Calculated to 13.5 oxygen atoms
Calcium	5.4	0.14	0.54	0.6
Potassium	9.7	0.25	0.97	1.1
Silicium	**23.4**	**0.84**	**3.25**	**3.6**
Aluminium	**2.5**	**0.1**	**0.39**	**0.4**
Sodium	3.9	0.17	0.65	0.8
Iron	4.8	0.09	0.35	0.5
Sulphur	0.0	0.0	0.0	0.0
Titanium	0.8	0.02	0.08	0.1
Oxygen	49.5	3.1	12	13.5 (10+3.5)

But Differential Thermal Analysis (DTA) and Thermogravimetric Analysis (TG) investigations show that the distribution of the oxygen atoms given below is not quite correct (Hünger et al. 1994). There is only one mol hydroxide in the octahedral layer, i.e. there is obviously a deficit in this layer. The rest of the oxygen (2.5 mol) can be assigned to the water of the interstratified layer.

EVALUATION OF THE RESULTS

- The white substances can be described as predominantly crystalline silicates with a layer structure. The basal spacing of the layers measured by the X-ray diffraction method is about 1.20 nm.
- The distance of the element layers is variable (between 1.17 to 1.24 nm) and depends on the alkali content (especially potassium), the alkali/silicium-ratio, the calcium content and water.
- The following formula summarises all information on this silicate obtained until now:

$$\mathbf{(Na_{0.7},K_{1.0},Ca_{0.6},2.5H_2O)[Si_{3.6}Al_{0.4}O_{10}/Fe^{2+/3+}{}_{0.6}Ti_{0.1}(OH)].}$$

This formula shows that the white substance can be a three layer silicate, but with a high deficit in the octahedral layer. The alkali and calcium ions (calcium is also bound as carbonate) are concentrated in the interstratified layer. The composition and the stability of the octahedral layer is not clear yet. Eventually the oversupply of aluminium determined by the chemical analysis of the concrete could compensate the deficit in the octahedral layer.

Other identifications of silicates given in the literature, e.g. as zeolite (Cole et al. 1981), as okenite (Cole & Lancucki. 1983) or as tobermorite gel (Davies & Oberholster. 1988), are more difficult because of the measured chemical relations. The Ca/Si ratio (=1/6) and the Al/Si relation (=1/9) measured here do not match the chemical composition of tobermorite or zeolithe. It is also important that the swelling process of the mineral observed here requires a specially crystalline silicate structure.

What can be the origin of this silicate? An analogous mineral could not be found in greywacke grains.

POSSIBLE REACTION MECHANISM - "WEATHERING" OF FELDSPARS AND MICAS

Based on the structure of the new mineral formation mentioned above the so-called greywacke reaction can be a "weathering" process. The origin can be the plagioclases containing a high amount of alkali ions and the mica minerals biotite and sericite. The final product would be the three-layer silicate found as a white substance on the cleaving planes of the greywacke grains and also the white substance inside the pores of the cement stone. Important for this mechanism are the other solubility conditions in the basic environment, the grain sizes of the constituents of the greywacke matrix consisting of feldspars and micas and certain petrographical features of this sedimentary rock.

This "weathering" hypothesis is supported by the accumulation of alkali ions, aluminium, silicium, iron and titanium inside the white substances. These ions or constituents can only come from the greywacke minerals themselves. A direct proof for this are the differences measured between the chemical composition of the whole concrete and the chemical composition of the concrete without greywacke. These differences are obviously caused by the solubility process of certain greywacke minerals.

The iron ions Fe^{2+} (there is enough in the Lusatian greywacke minerals biotite and pyrite) in the octahedral layer but also the aluminium ions control the silicate formation process in such a way that the formation of layer silicates especially of three-layer silicates is possible. This layer silicate is disordered (it seems to be a crystal with a high deficit in the octahedral layer). In the interstratified layer of this mineral alkali and earth alkali ions and water can be intercalated.

Based on this "weathering" mechanism a characterization of the white substance as a disordered mineral analogous to stilpnomelane offers good correspondence with the measured pattern (see Powder Diffraction File of the International Centre for Diffraction Data, PDF# 17-505 or 25-174). Stilpnomelane is a strongly destructed transition mineral between hydrobiotite and montmorillonite and it is a layer silicate.

CONCLUSIONS

The damaging process of concretes produced with greywacke aggregates must be a very complex mechanism because of the many crystalline minerals inside the greywacke grains.

The permanent pore solution containing $Ca(OH)_2$ can react with several mineral components of greywacke. After 15 to 20 years this leads to a release of certain ions from the greywacke grains. Controlled by iron (Fe^{2+}) and aluminium ions an at least partially crystalline layer silicate with a predominantly smectite analogous structure is formed. The mineral identification is difficult, but it might be a mineral analogous to stilpnomelane which is strongly disordered in the octahedral layer. But only a three-layer silicate structure can explain the swelling process observed. Under certain petrographical conditions the swelling process of this silicate leads to a cleaving of greywacke grains and thus to damage of the concrete.

Concrete damage caused by greywacke reactions - an AAR?
Our investigations have shown that **greywacke itself** used as an aggregate in concretes **can provide all constituents** which are necessary **for the damaging process**. These are the reactive silicate and aluminate components, but also the alkali ions. The cement provides only the CaO caused by the permanent $Ca(OH)_2$ solution. The one and only reactant that has to come from outside is the permanent humidity.

Therefore it would be only an aggregate reaction (maybe a specific greywacke reaction).

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INVESTIGATION OF AAR BY PHYSICO-CHEMICALS ANALYSIS IN THE CASE OF DIFFERENT CONSTRUCTIONS FROM ROMANIA

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ABSTRACT

The problem of AAR diagnosis began to be considered in the last 3-4 year by different construction's administrators from Romania, beside other possible factors of damage or destruction. The general procedures involve laboratory investigations, like psysico-mechanical, chemical and physico-chemical analysis. This paper partially describes the analytical algorithm developed for the diagnosis and investigation of ASR realized by optical microscopy, x-ray diffraction, infrared spectroscopy, scanning electron microscopy, electron microanalysis and differential thermal analysis. This analytical algorithm, completed with a special chemical sequence, was tested on real and laboratory concretes and some results are presented as two case study for ASR and ACR in Romanian constructions.

Key words:AAR diagnosis for real concretes. physico-chemical procedure. ASR and ACR case study in Romania.

INTRODUCTION

The continuing flow of conferences and papers on AAR wich demonstrates that the knowledge is still accumulating in this field of theoretical and practical research and development. The tests required for a detailed diagnosis of AAR need the expertise and the dedication of several investigations and laboratory staff, if reliable results must be obtained. The presence of AAR in concretes must be checked together with other factors as freezing and thawing, salt action, differential thermal strains. unsound aggregate, sulphate compound in soil and water, leaching, chemical attack, wear and abrasion, foundation movements, shrinkage and flexural action, rusted reinforced steel. It is very important for the laboratory staff to know and to speak the same language as the constructors, being necessary a close collaboration between engineers and laboratory staff in all stages of an ASR investigation. The general stages of AAR investigation are: site inspection and testing, sampling, laboratory investigations, assessment of site and laboratory findings, evaluation of the risk of future reaction.

When damage caused by AAR does occur, its consequences for the affected structure may appear to be serious, but, it is the general expert the deciding factor for further investigations, in situ or in laboratory. Some of the ASR macrostructural symptoms could be found at the microstructural level, or during the analytical process. Such symptoms are presented below:

1)Map cracking or pattern cracking, defined as openings on concrete surfaces in the form of a pattern of several three-armed star shapes that join up.

This can results from a decrease, increase, or both, in volume of the material near or below the surface.

2)Exudation, which means that a liquid or viscous gel-like materials can discharge through a pore, crack, or opening in the surface.

3)Efflorescence, given by salt deposits , usually white, which appear at the concrete surface, the substance having emerged in solution from within the concrete or masonry and deposited by evaporation.

4)Popout, defined as breaking away of small portions of a concrete surface due to localized internal pressure that leaves a shallow, typically conical, depression.

5)Discoloration, wich represents a change in colour from that which is normal or desired, for cement paste, reaction rims around the aggregates, filing material of the fissures and pores.

GENERAL PHYSICO-CHEMICAL LABORATORY PROCEDURE

The laboratory physico-chemical procedure developed for ASR diagnosis uses the optical microscopy, in reflexion and transmission, x-ray diffraction, infrared spectroscopy, differential thermal analysis, scanning electron microscopy and microanalysis. *Optical microscopy*, OM, the most widely used in AAR diagnosis, involves two types of examinations, in reflexion on finely ground sections, and petrografical investigations on thin sections, usually in transmission, with a quality polarizing microscope. The general flow chart for optical investigation used for ASR concrete investigation is indicated in figure 1. *X-ray diffraction, XRD,* completes the chemical and OM ASR investigation in our procedure, following a dedicated analytical flow chart which it is not related in this paper . The analysed samples are representative for the whole concrete, cement paste and aggregate, separated by crushing and sieving, or by chemical attack. It is not possible to obtain a complete separation, but this operation can reduce the influences of the matrix. There are limited data in the literature relating to ASR products, but it is possible to suppose "reaction products" as rose like crystal, "CKSH", products with structures as Okenite, CSH, Tobermorite, Gyrolite, "N"CSH-C"N"SH, $NaSi_{17}O_{13}(OH)_3.3H_2O$, $Na(K_2Na_2Ca)_{16}Si_{32}O_{80}.2H_2O$, Feldspar, Rhodesite, etc. Calcite, different alkaline carbonates, brucite and monocarboaluminate are implied as ACR reaction products. *Infrared spectroscopy*, IR, can confirm the results obtained with other analytical techniques and uses the same sample as X-ray diffraction analysis. The essential molecular IR vibrations bands are those relating to SiO_4^{2-}, SO_4^{2-}, CO_3^{2-}, OH^-, in correlation with the same reaction products indicated for the x-ray diffraction. *Scanning electron microscopy,* SEM, is an excellent tool for ASR diagnosis. The only ASR product accepted classification was made by SEM, based on the morphostructure criterion at magnifications of the order of 2000-5000. The SEM procedure was developped as algoritm for secondary and backscattered electrons with topologic, material and crystalline contrast. *Electron microanalysis* completes the general physico-chemical procedure, and in fig.2 it is presented the characteristic flow chart used for ASR diagnosis. It is very import for this analytical technique to ensure both the result representativeness and correlation with the analytical techniques.

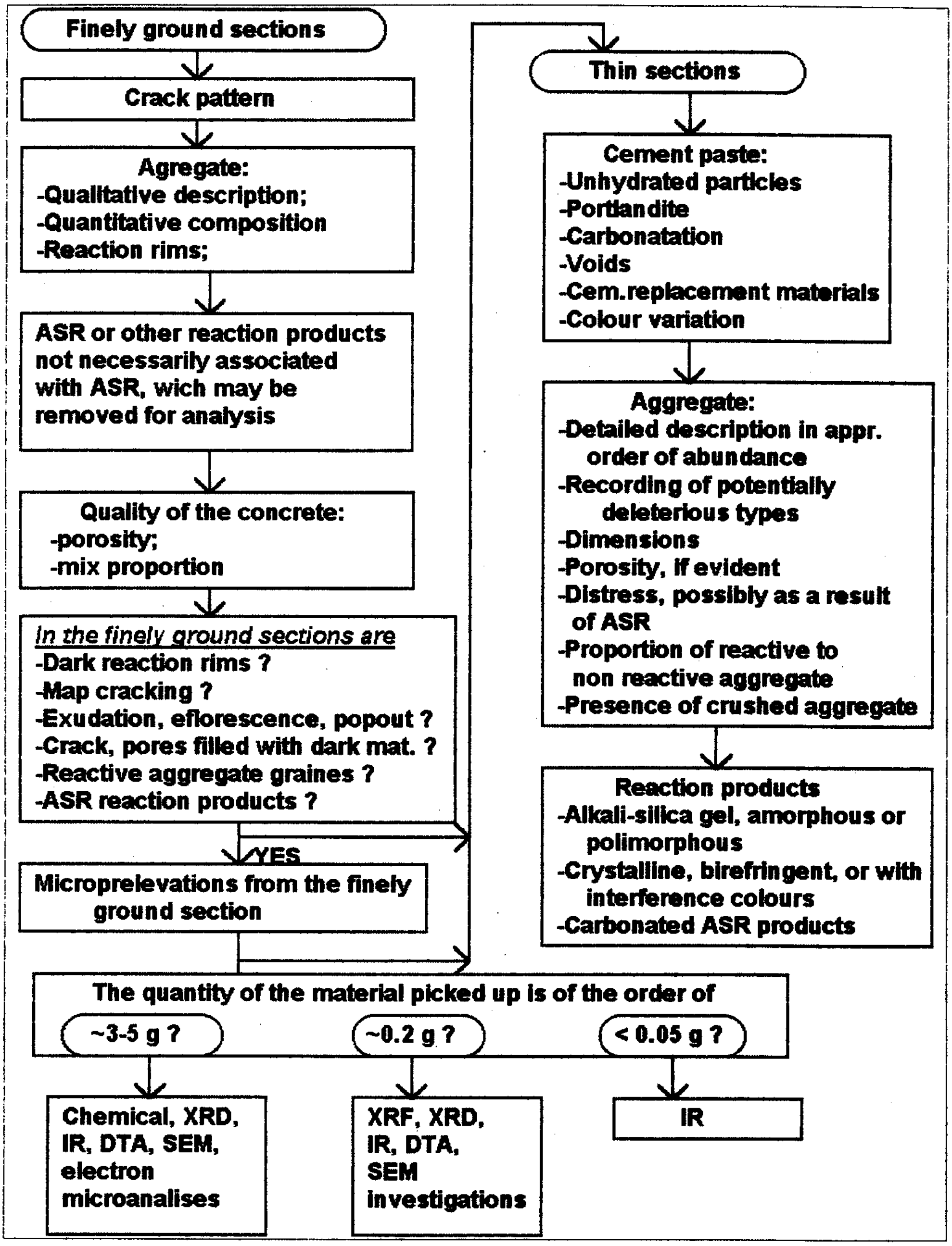

Figure 1. Flow chart for the concrete diagnosis and/or investigation of ASR by optical microscopy and connections with other analysis.

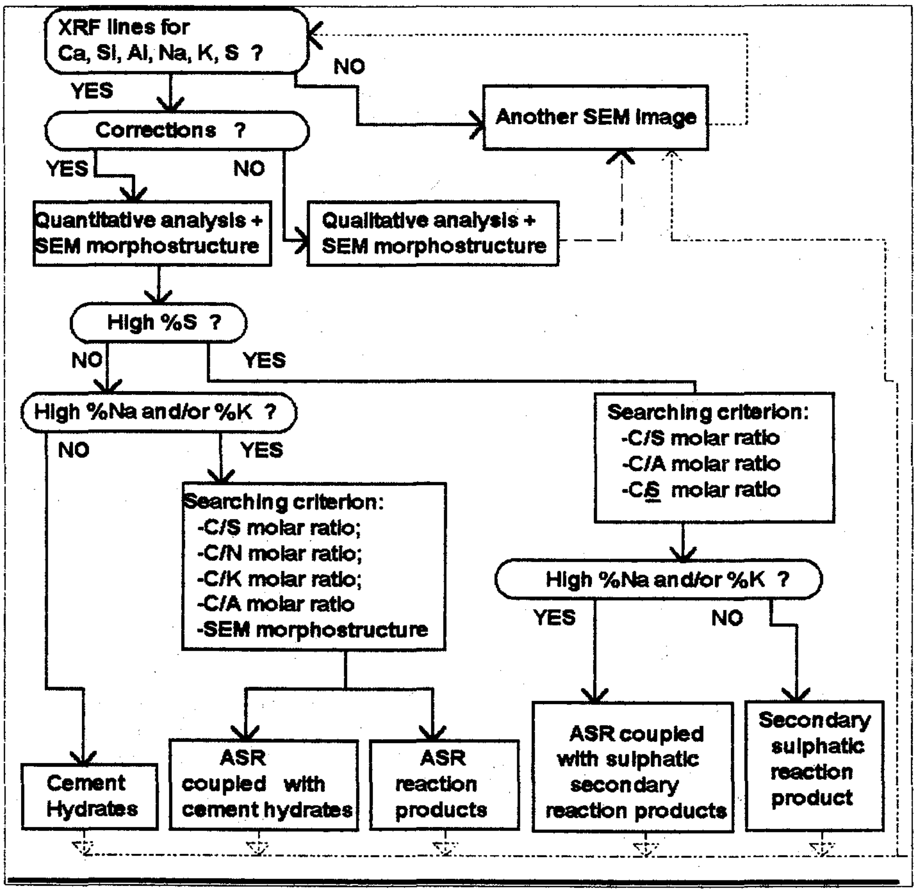

Figure 2. Flow chart for the ASR diagnosis by electron microanalysis

CASE STUDY FOR ASR-CHANNEL LINING VINATORI

In the region of Bicaz there are in service many dams and several channels. The main administrator observed very serious deteriorations of the channel lining at Pîngăraţi. There are map cracking, movements and concrete disintegration. Cores were extracted from the variable level of the water, below and above, both from affected and reference area too. The sample arrived in laboratory with very limited information, only codes and indications about the environmental humidity. The customer's assumption was that "ASR damaged the concrete structure". The physico-chemical procedure, completed with chemical analysis, were applied for every core. The equivalent alkalis content for concretes was between 1.19% and 1.56%, coming specially from the aggregate. The mineralogical composition of the investigated concrete was: (40-45)% Quartz, (12-20)% Calcite, (10-15)% Feldspar, (10-15)% Illite-micas, ~5% Clorite, ~2% Gypsum, and amorphous compounds, of the order of 10-15%.

The main component of the aggregate is siliceous, containing quartz as discrete grains, quartzite micro and crypto-crystalline, gneiss, schist, flint as chalcedonic silica. Reaction rims are frequently associated with $Ca(OH)_2$, cement hydrates, granular carbonate and cracks, approximately coloured as ferric oxides' impregnations. Acicular reaction products were identified near reactive aggregate grains. The hardened cement paste is completely hydrated, with carbonated areas, and low distribution of the fissures. The fissure pattern observed by OM seems to be generated by freezing and thawning, leaching, ASR and secondary sulfatic reactions, these being the damaging factors. SEM and electron microanalysis indicated ASR reaction product like amorphous and polymorphous silica gel, crystallized products like rods, lamella, in associations with CSH I, II and sulphate compound, fig.3. ASR reaction products are frequently carbonated, specially at the cement paste level. Silica gel and sulphate crystallized reaction products are very unstable in the SEM being altered by the electronic beam. The chemical composition for ASR reaction products, observed at SEM with morphostructure as compact like gel, it was obtained by the electron microanalysis, with the following compositional ranges as bellow :

$(48\text{-}92)\%SiO_2$ $(5\text{-}42)\%CaO$ $(3\text{-}22)\%Al_2O_3$ $(0\text{-}1)\%K_2O$ $(0\text{-}5)\%Na_2O$

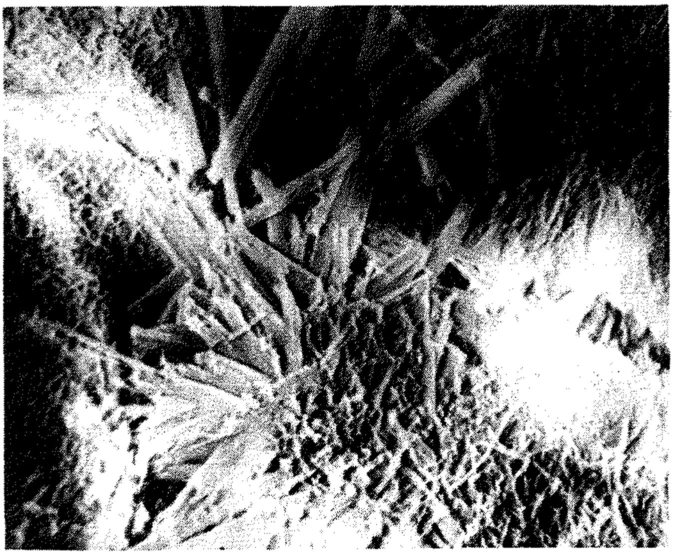

Figure 3. SEM image for the cement paste morphostructure damaged by ASR products associated with secondary sulphate reaction products.

The final conclusion of the laboratory repport was that ASR and secondary sulphate reaction are implied in the concrete deterioration, but the assement of the contractor was that the ASR contribution, as damaging factor, is of the order of 20-25%.

CASE STUDY FOR ACR - ADDUCTION CHANNELS FROM DRAGAN-REMETI

The Dragan-Iad hydrotechnical construction, located in the region of Transilvania, is in service from 1980. The initial concrete mix was made with a special cement for dams, coarse dolomite aggregate, 3-40 mm, and river sand mixed with very fine dolomite powder obtained during the rock grinding. The ACR reactivity of the aggregate was tested by STAS 5440/70, (ASTM C227 mortar test). The external concrete damage symptoms are as massive and contiunous calcite efflorescence, "popouts" with irregular depths and forms, cracking and cement paste loosing.

The actual concrete investigations were conducted in close cooperation between contractor and Ceprocim laboratory. For the final results assembling and conclusions the Polytechnic University of Bucharest it was implied. The laboratory investigations, (chemical, XRD, IR, DTA and optical microscopy were done on cores, (transversal and longitudinal cut sections) and crushed samples from damaged areas. Eight different cores, with the chemical and mineralogical compositions indicated in tables 2 and 3, were used for this paper.

TABLE 2. *Concrete mean chemical analysis*

Compound	1	2	3	4	5	6	7	8
SiO_2	22.70	24.70	33.77	29.21	26.73	21.51	31.56	20.05
CaO	24.53	24.04	22.08	22.89	22.03	24.70	21.58	25.23
Al_2O_3	3.33	2.44	4.09	2.95	2.84	2.81	4.94	3.18
Fe_2O_3	1.20	1.41	1.00	1.61	1.62	1.40	1.78	1.79
MgO	13.00	13.03	9.09	11.73	13.35	13.96	9.33	13.69
SO_3	0.89	0.67	0.59	0.65	0.47	0.51	0.41	0.71
Na_2O	0.44	0.58	0.66	0.59	0.53	0.42	0.58	0.67
K_2O	0.44	0.53	0.69	0.63	0.56	0.42	0.59	0.59
TiO_2	0.59	0.58	0.55	0.59	0.52	0.62	0.58	0.60
Ins.	14.95	27.99	36.31	32.60	29.22	23.63	36.18	24.43
LOI 580 °C	5.96	7.48	5.32	5.73	5.69	6.52	6.55	5.71
LOI 1050 °C	33.20	31.96	26.80	29.10	31.03	34.05	28.48	33.75

TABLE 3. *Concrete mean mineralogical composition.*

Compound	1	2	3	4	5	6	7	8
Dolomite	50	45	45	45	45	45	40	50-55
Calcite	5	5	5	5	5	5	<5	5-7
Quartz	20	15-20	20-25	20	20-25	15-20	25-30	15
Feldspar	5	5	5	<5	5-10	5	5	5
Illite	5	5	5-10	5	-	-	5	5
$Ca(OH)_2$	<5	1-2	<5	2-5	1-2	1-2	2-3	2-3
$Mg(OH)_2$	-	x	?	?	x	?	?	?

The actual concrete mix compositions was obtained using special corrections for chemical, XRD and IR analysis for the mean concrete sample, solubilizated fraction, insoluble and hardened cement paste, (for MgO from dolomite, CaO from dolomite and calcite too, for Al_2O_3, SiO_2, Na_2O and K_2O from soluble illite).

It was possible to approximate the actual mix proportions, table 4, and the actual chemical compositions for the hardened cement paste, table 5.

The coarse aggregate contains 90-95% dolomite, with 3% insoluble, (as 80-85% quartz and 15-20% feldspar), and it has a granular texture. The dolomite grains bigger than 16 mm are frequently fissured with a propagation in the cement paste. The fissuration seems to be in more extent for concrete coming in contact with the water. The reaction zone between dolomite and the hardened cement paste is as "positive" type and the dark colour can be associated with very fine reactive siliceous grains. The dolomite grains have a gradient colour from inside to the reaction zone in correlation with presence of the granular calcite. The calcite is identified among the coarse grains fissures. The cracks' aspects, without filling materials, at the level of the cement paste or near the aggregate, can indicate a leaching phenomenon.

TABLE 4. *Calculated concrete mix compositions.*

Compound	1	2	3	4	5	6	7	8
Total aggr.	85%	80%	90%	80%	85%	75%	85%	85%
Carbonate	63%	63%	56%	63%	59%	67%	53%	71%
Sand	31%	37%	44%	37%	41%	33%	47%	29%
S/C	0.43	0.59	0.79	0.59	0.69	0.49	0.89	0.41
"C"/Aggr.	0.15	0.20	0.10	0.20	0.25	0.25	0.15	0.15

(Total aggr.=The aggregate percentage in concrete; Carbonate, Sand= Aggregate component's percentage normalized for %Total aggr.; S/C= Sand/Carbonate percentage ratio , "C"/Aggr.="Hardened cement paste" / Total aggregate percentage ratio.)

The sand/carbonate ratio may indicate an increasing of the siliceous component, from the initial ratio of 0.33, caused by the calcite exudation on the concrete's surface as allkali-carbonate reaction product produced obtained during the dedolomitization reaction. The brucite content is at the limits of XRD and IR analytical techniques. The MgO content for the hardened cement paste seems to be bigger than the initial limits imposed for the used cement.

TABLE 5. *Chemical compositions calculated for the hardened cement pastes.*

Compound	1	2	3	4	5	6	7	8
SiO_2	18.04	16.12	18.59	18.79	18.24	14.45	16.36	18.54
CaO	55.62	59.18	57.36	57.71	55.23	53.08	66.68	48.64
Al_2O_3	6.67	7.12	9.02	6.79	7.71	5.00	5.27	9.61
Fe_2O_3	3.14	3.00	3.74	5.14	3.64	3.15	3.10	5.83
MgO	9.54	9.53	6.13	7.14	12.61	19.93	5.43	8.16
SO_3	5.82	3.76	3.62	4.36	2.74	3.15	2.17	6.89
Na_2O	0.72	0.65	0.68	0.71	0.72	0.68	0.49	1.07
K_2O	0.59	0.71	0.74	0.79	0.78	0.55	0.38	0.97
Na_2O_{eq}	1.10	1.11	1.15	1.22	1.23	1.04	0.74	1.69

The fine aggregate, < 3mm, contains quartzite, illite, feldspar, and there were identified tipical aspect for ASR, specialy for quartzite.

The study of the old repports, (10-15 years ago), indicated that the problem of AAR was either unknown and/or neglected. Based on the actual state in the AAR field, it is possible to afirm that the ACR and ASR deterioration began after two years of service. The contractor assessed laboratory findings with other physico and mechanical tests and concluded that reparations are needed, being also proposed the closing of the exploiting carrier for future hydrotechnical constructions in the region of Dragan-Iad.

CONCLUSIONS

The ASR diagnosis involves the use of a complex and elaborated analytical step-by-step procedure. For that it was created a specially physico-chemical algorithm wich involve Optical Microscopy, Chemical X-ray Fluorescence Analysis combined with wet techniques, X-ray Diffraction, Infrared Spectroscopy, Differential Thermal Analysis, Scanning Electron Microscopy with secondary and backscattered electrons and electron microanalysis. It is not possible to indicate a versatile technique, and that is why we tried all possible information and analytical criterion, including site observations and testing, a close cooperation with the constructor, a transfer of actual level of the knowledges of ASR. There are signs that some of Romanian constructions may prescrit ASR and ACR damages, but it is not only the laboratory who decides the final diagnostic. The procedure was tested for dams, roads, industrial constructions and monuments.

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ALKALI - AGGREGATE REACTION IN A CONCRETE WATER STORAGE TANK

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ABSTRACT:

A large reinforced concrete water storage tank, built in 1982, has shown signs of distress in the form of map-cracking in the walls, and vertical cracking in many of the 42 internal columns supporting the roof of the tank. The cracking was noted 4 years after the tank was constructed and has progressively worsened.

The cracking was most severe in the cylindrical columns, but was also evident in the walls of the structure. A second tank of similar age is also exhibiting the same symptoms.

Cores were extracted from various parts of the structure for diagnostic purposes using petrographic methods, scanning electron microscopy, and evaluation of residual expansion of the concrete.

The investigations have shown that the schistose hornfels aggregate used in the concrete has undergone a severe alkali-aggregate reaction, causing the observed cracking. The residual expansion of the cores appeared low and indicated that the reaction has probably levelled off, and that remedial action can be contemplated. Preliminary steps taken towards repairing the structure will also be discussed.

Keywords: Alkali-Aggregate reaction, expansion, Cracking, Concrete Water Tank.

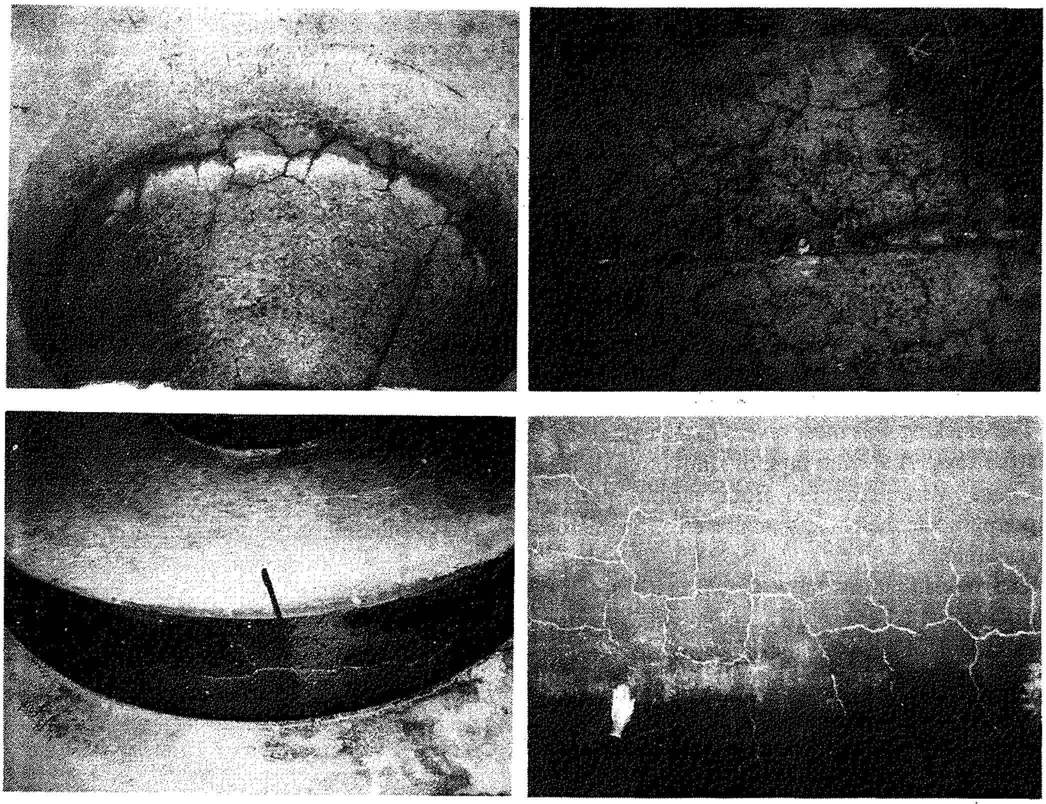
INTRODUCTION:

A large reinforced concrete tank approximately 41 metres in diameter and 5 metres high is a major component in potable water storage for the City of Mildura. The tank has a wall thickness of 300 mm and contains 42 columns, of 500 mm diameter, concentrically arranged in three circles on which the 300 mm thick roof of the tank has been cast. The roof has been cast so that it rests on the columns and the wall of the tank but is not integral with the walls. The columns are sitting on reinforced ring beams (platforms) of 2400 mm diameter. The floor of the tank is 150mm thick reinforced concrete. The construction of the tank was completed in 1982 and cracking of the columns and walls was noticed in 1986. The cracking progressively became more evident.

The columns have developed more severe cracking than the walls. Some cracking features of the water tank are illustrated in Figure 1. Another tank of the same age is also showing the same symptoms. Alkali- aggregate reaction (AAR) was suspected as a possible cause of cracking.

Records kept of the construction project revealed the concrete mix to contain 380 kg/m^3 of a general purpose Portland cement. This cement is known to be of high alkali content, generally above 0.8%Na_2O equivalent, although the alkali content of the cement used has not been recorded. The coarse aggregate was an imported schistose hornfels and the fine aggregate a local fine river sand. The water/cement ratio was 0.45 and the average concrete slump 55 mm. The concrete also contained a water reducing chemical additive (Embecon ALF30) at a dosage rate of 400 ml/100 kg cement. The chemical composition of this admixture is not available.

The purpose of this study was to identify whether AAR was the cause of the observed cracking and accordingly propose some remedial action to prevent further deterioration of the structure.



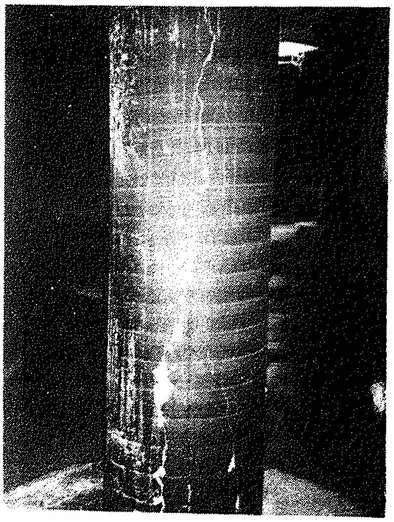


Figure 1 - Some cracking features of the various parts of the water tank. Clockwise from left: cracked column, column ring beam, top of column joining the ceiling, the ceiling and wall. Cracks in columns are wider than other cracks.

EXPERIMENTAL WORK

Samples of the coarse and fine aggregates were obtained from the same sources as the original materials, and tested independently for alkali- reactivity. Core samples 100mm in diameter and varying in length, depending on their location, were taken from various parts of the tank. Table 1. shows the location of the cores and the condition of the concrete zones sampled. The drilled cores were used for petrographic examination and measurement of their residual expansion. The petrographic examination included preparation of thin sections from slices of concrete sawn from the cores and impregnated with a yellow-stained, fluorescent epoxy resin to show up the micro cracking. Scanning electron microscope (SEM) examination of the concrete specimens was done on the concrete slices, that included the fracture surface of the cores, where reaction rims and gel products were evident.

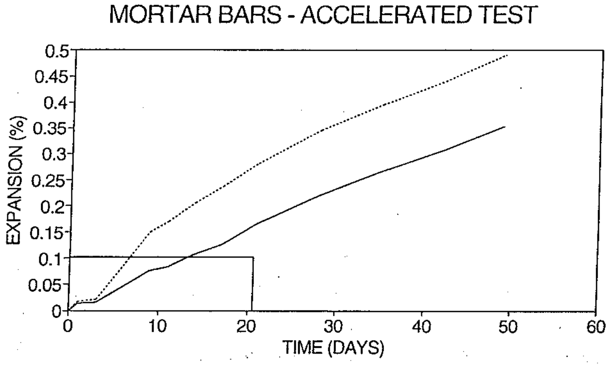


Figure 2. Accelerated mortar bar test results for the coarse (dotted line) and fine (solid line) aggregates showing them to be potentially reactive. Time refers to Nº of days in 1M NaOH solution at 80°C.

Figure 3. View of a fracture surface of a core showing reacted aggregate particles with dark outer rims and white inner rims. The matrix is impregnated with AAR gel. A void on the LHS is filled with white AAR product.

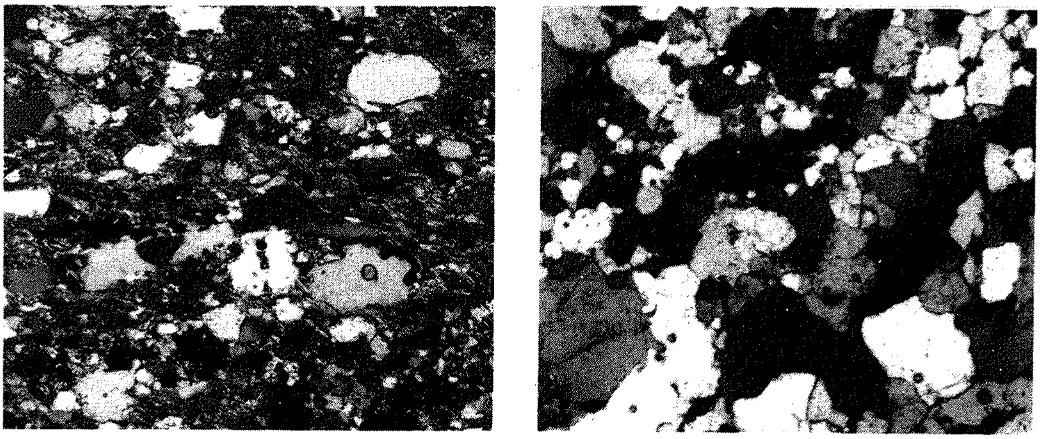


Figure 4. Some petrographic features of the reactive hornfels in cross polarized light showing schistosity and varying grain size (left) in a matrix of chlorite and mica, and bands with quartzitic features (right). Cordierite was sometimes completely sericitised in a fine quartz, mica, chlorite matrix (magnification 37 x).

Table 1. - Location of core samples.

Core No.	Location	Condition
1	Drilled from the roof into the top of column 17	Minor crazing on roof top
la	Drilled 300 mm away from core 1 into column 17	Minor crazing on roof top
2	Drilled externally from wall near column 17	Crazed
3	Platform at base of Column	Cracked
3a	Near Core 3	Cracked
4	Shoulder height on Column 2	Cracked
5	Drilled from floor into ring beam under Column 2	Widely spaced cracks
6	Base of Column 12	No visible cracking

For the measurement of the residual expansion potential of concrete, cores of suitable length were trimmed to a length of 150 mm, and two sets of demec disks were glued to each core (on opposite sites) with a gauge length of 100mm. A moisture and alkali-resistant epoxy resin was used for gluing the demec disks and allowed to harden for one week. The cores were then immersed in a solution of 0.5 M NaOH solution for 24 hours at 23°C before taking the initial length reading. After this treatment the cores were wrapped in a wet cloth and then in plastic sheeting and stored over water in sealed containers at 40°C. Two segments of 150 mm length were cut from Core 6, one was soaked in 1 M NaOH and the other wrapped in a wet cloth and plastic sheeting (both at 23°C), with the initial length reading taken 24 hours later. These were then stored at 40°C for monitoring their expansion. The core specimens were allowed to recover overnight at 23°C before each expansion measurement. The reason for soaking the specimens in 0.5 M NaOH was to allow the specimens to absorb moisture so that this is not counted towards AAR-related expansion, and at the same time not to unduly dilute the pore solution of concrete, which usually contains a high concentration of alkali hydroxide. This depends on the alkali content of the cement used. A cement of 0.8% Na_2O equivalent could result in a pore solution concentration of about 0.6 M NaOH in a mortar with sand/cement ratio of 2 and water/cement ratio of 0.5 under sealed conditions.The soaking treatment was thought appropriate for the concrete of the water tank which is constantly in contact with water and remains wet.

RESULTS AND DISCUSSION

Aggregate Specimens: The coarse and fine aggregates originally in the concrete were allegedly required to comply with the then Australia Standard test methods AS 1465 (outdated) but no problem with alkali reactivity of the aggregates had been established. It must be noted that the alkali-reactivity tests included even in the current Australia Standard methods AS 1141-38 (mortar bar test) and AS 1141-39 (quick chemical test) have been shown to be inappropriate for determining the alkali-reactivity of many Australian aggregates (Shayan,1992). The newly acquired aggregate samples were tested according to the more suitable test procedure and criteria developed for Australian aggregates (Shayan et al. 1988) consisting of storage of mortar bars in 1M NaOH solution at 80°C for 21 days. Expansions of 0.1% or greater at 10 days of storage in the NaOH solution indicate reactive aggregate and the same level of expansion at 21 days of storage in the NaOH solution indicates slowly reactive aggregate. Expansions less than 0.1% in 21 days indicate non-reactive aggregate.

Figure 2 shows that the schistose hornfels is classed as reactive and the river sand as slowly reactive. Therefore these aggregates would not have been accepted for use in concrete, without adequate precaution, had the new test method and criteria been in effect at the time of construction. As a matter of interest the more reactive coarse aggregate was subjected to the chemical test (AS 1141-39) and was erroneously classed as innocuous (Sc=33 and Rc=31 m moles/l)

Concrete Specimens: Most cores showed fracture surfaces that contained aggregate particles with distinct reaction rims and much gel on the mortar phase (Fig. 3.). Some of the aggregate particles on the sides of the cores were internally cracked which sometimes extended into the matrix.

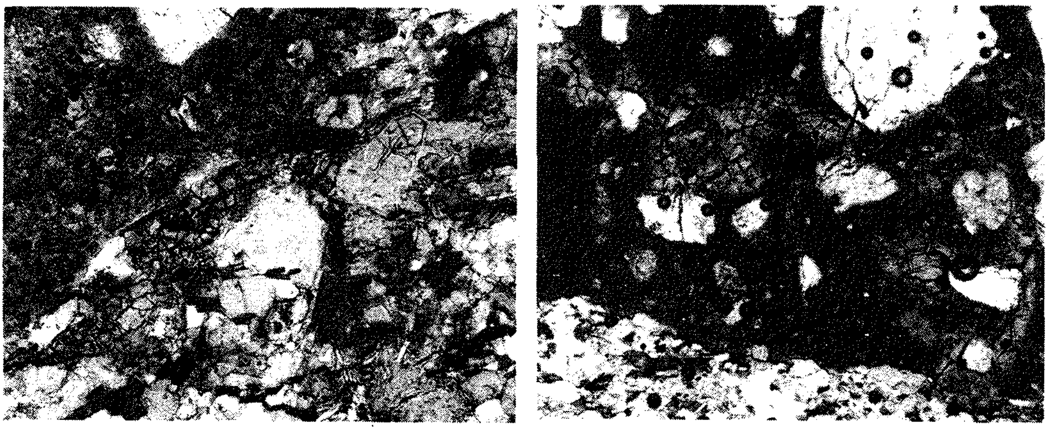
a) **Microscopic Examination**

Thin sections made from the various cores have shown the coarse aggregate to be a fine-to medium-grained schistose rock containing quartz rich bands with foliated mica (biotite and muscovite) and other bands containing foliated biotite, muscovite, chlorite quartz and cordierite. Some cordierite grains were seen to be completely sericitised. Some quartz grains showed a small degree of undulatory extinction. Figure 4 shows some of the petrographic features of the reactive hornfels.

The metamorphosed nature of the rock and the microcrystalline nature of the quartz may have made the aggregate reactive to alkali in the concrete.The presence of large amounts of mica in the aggregate implies that some potassium may have been contributed to the concrete from the aggregate, increasing the concentration of alkali-hydroxide in the pore solution of the concrete, and providing more favourable conditions for AAR to occur.

Figure 5 shows the presence of AAR gel in thin sections of the cores, occurring both at the aggregate interfacial zone (Fig 5A) and in the matrix (Fig 5B) probably filling micro-cracks. The sand grains also appear to have reacted although the observed AAR gel may have originated from the nearby coarse aggregate (Fig 5B,bottom). The AAR gel has also filled air bubbles in the vicinity of the reacted particles (Fig 5C).

Figure 5. *Some Petrographic features of the AAR-affected concrete in plane polarised light (magnified 40x).*



(5A) *gel at aggregate/matrix interface*

(5B) *gel in the matrix near sand grains and above coarse aggregate (bottom)*

(5C) *pore filled with AAR gel from the coarse aggregate on the LHS (next page)*

(5C)

Scanning electron microscopy of specimens taken from the cores revealed various morphological features of the reaction products. Figure 6 shows both amorphous AAR gel and crystalline products, the latter often forming in the white inner rim observed around reacted coarse aggregate. Energy dispersive X-ray analysis of the AAR products showed that the gel in contact with the mortar phase was richer in Ca than K, whereas the crystalline products are very rich in K. All the cores contained AAR products similar to those in Fig 6 although the amount of gel in the visually uncracked column was much less, and no crystalline product was detected. The latter forms in the more advanced stages of AAR, such as in the cracked columns and the wall of the tank. Occasionally, some mats of ettringite were found at the aggregate interfacial zones and in air bubbles, but this has no connection with the cracking of the concrete. These AAR features are the same as those observed in many AAR-affected concretes in Australia and around the world.

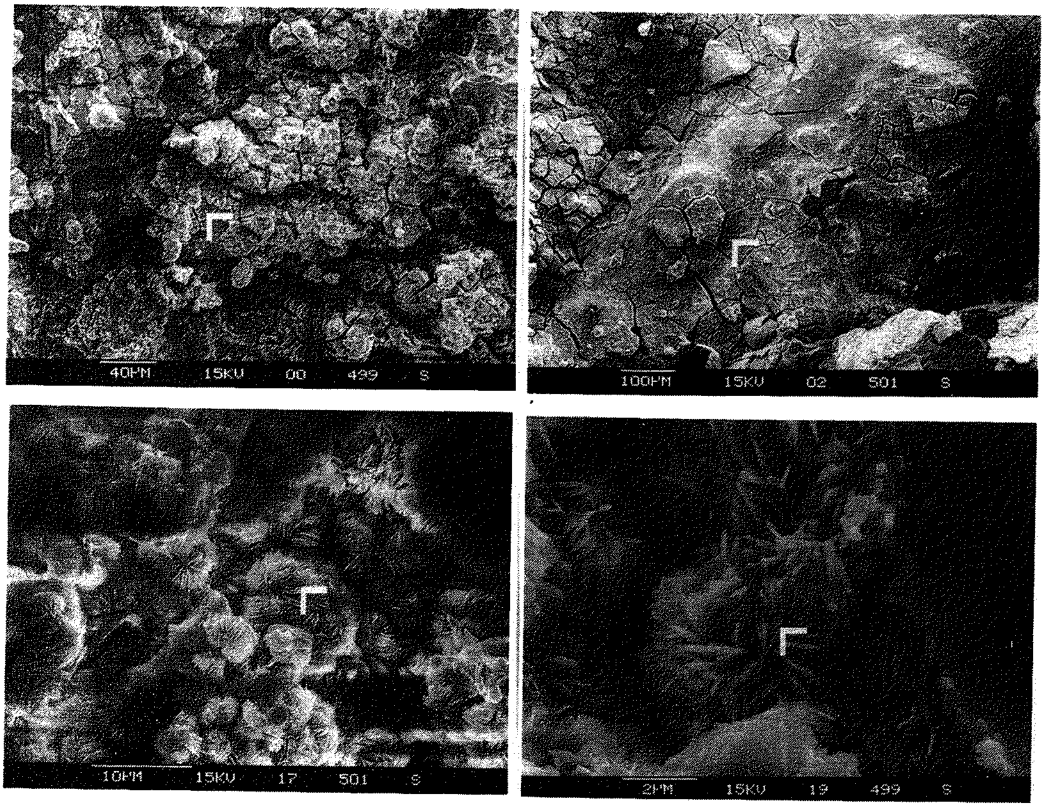
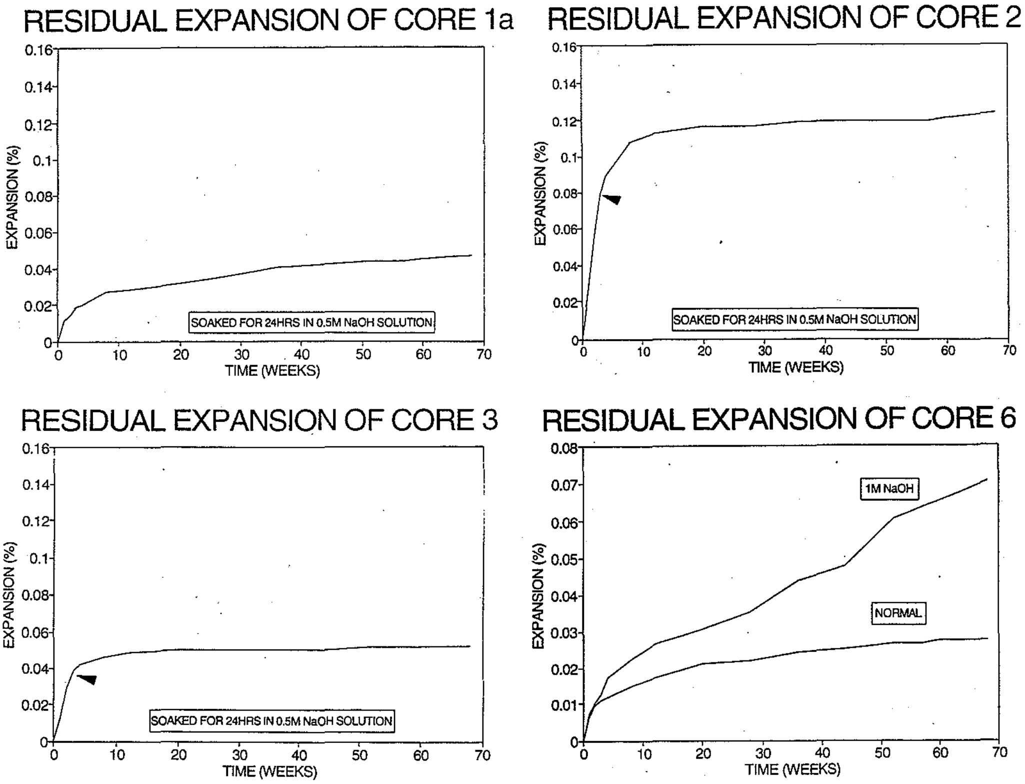


Figure 6. Representative morpholgical features of AAR gel (top) and crystalline products (bottom). The gel contains high Ca due to contact with the matrix, whereas the crystalline products are high in K.

RESIDUAL EXPANSION OF THE CORES

Results of expansion measurements on cores up to 68 weeks are represented in Figure 7.

Cores 1 and 1a behaved similarly, with a trend of continuing gradual expansion, although Core 1 which had a piece of reinforcement bar expanded less than Core 1a without any reinforcement piece. The initial faster rate of expansion may have been due to water absorption by both the AAR gel and the cementilious phase, but the gradual expansion is likely due to continuing AAR. Core 2 from the exterior of the wall showed a high initial rate of expansion which levelled off over a few weeks. This probably means that despite the presence of reacted aggregate, the moisture supply was inadequate in the structure, but when exposed to more water rapid expansion took place. However it is not possible to translate this behaviour to that of the heavily reinforced wall of the tank. The presence of reinforcement has a considerable effect on restraining the AAR expansion (Hobbs 1988). Cores 3, 3a and 5, all from reacted zones inside the tank, showed a rapid initial expansion over the first four weeks, probably due to moisture uptake, which levelled off and did not continue, indicating little residual expansion potential from such zones. For Core 6, the piece continually soaked in 1 M NaOH solution is continuing to expand, indicating the presence of reactive components, and the piece that was only kept moist without soaking continues to expand at a very slow rate, but this may not result in cracking in the column itself. The shape of the expansion curves for the cores together with the fact that the structure is reinforced indicates that future expansion of the concrete in the structure would probably be low, and that remedial measures could be taken to prevent further deterioration of the cracked concrete, without much risk of re-cracking.


RESIDUAL EXPANSION OF CORE 1a

RESIDUAL EXPANSION OF CORE 2

RESIDUAL EXPANSION OF CORE 3

RESIDUAL EXPANSION OF CORE 6

Figure 7. Residual expansion of representative cores initially treated as indicated, then wrapped in wet cloth and plastic sheeting and stored over water at 40°C in sealed conditions.

REPAIR OF THE WATER TANK

The element of the tank which has exhibited the greatest degree of cracking has been the cylindrical columns with pattern cracking towards the top and multiple single vertical cracks typically 1.0 mm wide along the length of column. The roof, walls and floor have a varying degree of cracking over their surface area and are best described as hair line in width. So far only the columns have had a repair/protective treatment.
This treatment has been applied for two reasons :-

- To restrict the water supply to columns and prevent further AAR expansion
- To prevent or reduce corrosion of the steel reinforcement

As the AAR reaction is water dependent it was decided to encase each partially dried column in a waterproof membrane. This approach will probably meet the two criteria above.
The technique used was :-

- drain the tank
- clean the column surface with a fine grit blast
- dehydrate the structure internally by allowing it to dry under forced ventilation and heat for a period of 28 days
- apply a fibreglass coating

The effects of this treatment is currently being monitored.

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ALKALI-AGGREGATE REACTION COMBINED WITH FREEZE/THAW IN NEWFOUNDLAND, CANADA -- PETROGRAPHY USING EPMA

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ABSTRACT

Detailed petrographic examinations coupled with EPMA measurements were made of unhydrated cement minerals, hydrated C-S-H gels and ASR gels in seven deteriorated 50-70 year old concrete structures in Newfoundland, Canada, to relate their chemical compositions with their origin and petrographic characteristics. It was found that the compositions of the ASR gels and C-S-H gels are strikingly continuous, particularly in the weathered portions of these concretes. Under weathering conditions, both alite and belite hydrate to leach calcium down to a Ca/Si ratio 1.2, while ASR gels during migration absorb such liberated calcium and transform into pozzolanic C-S-H gels having a Ca/Si ratio of 1.0-1.2. This suggests that an apparent chemical equilibrium is attained between the C-S-H and ASR gels within concretes that are undergoing combined deterioration from ASR and freeze/thaw weathering.

Keywords: alkali-silica reaction, ASR gels, C-S-H gels, deterioration, EPMA

INTRODUCTION

In Newfoundland, Canada, some concrete structures more than 50 years old are undergoing severe deterioration from alkali-silica reaction (ASR) combined with freeze/thaw weathering (Bragg & Katayama 1993, Bragg 1995) (***Table1, Fig.1***). Various types of reaction products have been reported from these concretes (Katayama & Bragg 1995).

To clarify the relationship between the type and the process of chemical reactions in these old deteriorated concrete structures, as well as to interpret the origin of the reaction products, detailed petrographic examinations coupled with a total of 600 EPMA measurements were made of unhydrated cement minerals, their hydrates and alkali-silica gels found in concretes. As a result, a continuous series of calcium silicate gel products predicted by Urhan (1987), ranging from typical ASR gels through intermediate pozzolanic C-S-H gels to typical C-S-H gels, were confirmed from each concrete structure.

This paper discusses the process of a long-term alteration of cement hydrates and ASR gels in these old concretes, with emphasis on the usefulness of petrography and EPMA analysis of both cement minerals and gel products in diagnosing chemical reactions in the deteriorated concrete structures.

Table 1 Deteriorated concrete structures near St.John's

Location	Structure	Year	F	ASR
Red Cliff	Radar base	1941	I	W
Water Street	Retaining wall and gate	1923	W	I
Fort Amherst	Gun bunkers	1941	I	I
Cape Spear	Gun bunkers	1941	I	I
Petty Harbor	Reservor dam	1923		W
Tors Cove	Bridge (old)	1935	I	I
Cape Broyle	Bridge (old)	1935		I

F: freeze/thaw, I: intense, W: weak

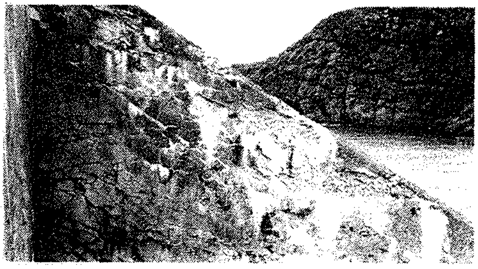


Fig.1 Concrete structure undergoing ASR and freeze/thaw

PETROGRAPHY OF REACTED AGGREGATES

Petrographic examination of the deteriorated concrete revealed that late Precambrian siltstone and sandstone coarse aggregates containing crypto- to microcrystalline quartz (Katayama & Bragg 1995), have reacted deleteriously in these concretes, due to late-expansive alkali-silica reaction which was formerly called the alkali-silicate reaction in Eastern Canada (e.g. Lewczuk et al. 1990). Rhyolites in the sand, originally glassy but now devitrified to a cryptocrystalline state, have also reacted. However, metarhyolite did not react because of its coarsely recrystallized texture (*Table 2*). Sea shells and wood fragments were occasionally found in these concretes.

Table 2 Lithology of aggregates in the deteriorated NF concretes

Rock type	Red Cliff pillar		Red Cliff wall		Water Street	Fort Amherst		Cape Spear	Petty Harbor	Tors Cove	Cape Broyle old		Cape Broyle new
	b	c	b	c	b	b	c	b	c	b	b	c	c
Sandstone	S		S		C*	G	C*	G*	C*	G	G		G
Siltstone/sil.sh	G*	G*	G*	G	C*	G*	G*	G*	S	G*	G*	G*	G*
Slate/phyllite	S		S			S		S		S	S		S
Quartzite		S	S		S	S							
Gneiss	S		S	S	S								
Basalt(alt/meta)	G	G	G	G	S			S				S	S
Rhyolite	S	S	S*		S*	S*		S*	S	S		S*	G*
Metarhyolite	S	G	S	S			S				S	S	S
Porphyrite		S	S		S	S		S			S		S
Granite	G	G	G	G							S	S	G

*: reacted, b: block sample, c: core, S: sand, G: gravel, C: crushed stone

ASR GELS IN CONCRETE

Five types of ASR gels were recognized in the deteriorated concretes, which represent different origins and stages of formation. They are 1) crack-filling in aggregate, 2)crack-filling in cement paste, 3)air void-filling/lining in cement paste, 4)gels interreplacing C-S-H gels, 5)gels replacing reaction rims of aggregate. The entire processes of chemical alterations of these gels in the concretes were characterized by polarizing microscopy of polished thin sections, coupled with about 600 quantitative measurements of compositions of these products using a WDS-type EPMA (Katayama & Bragg 1995).

Crack-filling ASR gels in aggregate

These gels filling cracks in the reacted aggregates are relatively fresh and contain a large amount of alkali and lower calcium (*Table 3*). In one example from the Petty Harbor dam, the Ca/Si ratio of a "gel vein" ranged from 0.2 to 0.4. The gel vein sometimes consists of several alternating layers of different crystallinity (*Fig.2*), and their chemical compositions may differ layer by layer. Generally, inner layers of the gel vein represent a later formation, and have higer Ca/Si ratios than outer layers (gel-walls) of older generation.

Fig.2 Crack-filling ASR gels

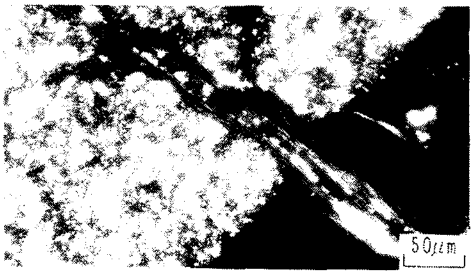
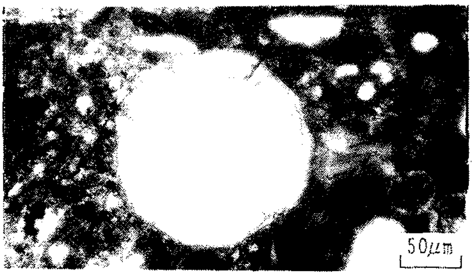


Fig.3 Air void-filling ASR gels



Crack-filling ASR gels in cement paste

Where a large crack extends from the reacted aggregate into cement paste, ASR gels often fill this crack as a continuous gel vein (***Fig.2***). During migration from the reacted aggregate, these gels lose alkalies and gain calcium from the cement paste in a short distance (***Fig.4***), sometimes losing transparency. In one case (Water Street), the Ca/Si ratio of such crack-filling gels increased from 0.7 at an aggregate/cement interface toward 1.2 in the cement paste, only within a distance of 0.5mm (***Table 4***). The crack-filling gels appear to gain crystallinity with an increasing Ca/Si ratio, particularly over 0.7.

ASR gels replacing reaction rims of aggregate

This type of gel is often found in the periphery of reacted sand grains of rhyolite and rhyolitic tuff, but can be rarely seen in the reacted coarse aggregate of a sedimentary siliceous shale. In thin section, the reacted rhyolite has colorless and transparent reaction rims, which are replaced by ASR gels with a Ca/Si ratio between 0.7 and 1.1 (Water Street & Cape Spear). Calcium content is relatively high, because these gels have long been in contact with the cement paste and have probably altered their compositions.

Air void-filling ASR gels in cement paste

Along the crack in the cement paste, ASR gels migrate from the reacted aggregate to fill the air-void. This type of gel has a higher Ca/Si ratio than the crack-filling gel because it has long been in contact with cement paste (***Table 5, Figs.3, 4***). Large air-voids are lined with transparent ASR gels with a cyclic zoning from amorphous to crystalline, but their compositions are not different from each other. Gels filling smaller air voids have higher Ca/Si ratios than those in the larger air-voids, and sometimes present a brownish tint in thin sections.

Table 3 Crack-filling ASR gels in aggregates

	Petty Harbor In sandstone			Fort Amherst In sandstone			Cape Broyle In siltstone			Cape Spear In sandstone		
An.no:	Pc422	Pc415	Pc420	Fc31	Fc27	Fc28	Cb136	Cb138	Cb137	Cb105	Cb104	Cb102
SiO_2	62.19	60.36	56.46	59.20	55.14	51.11	50.55	52.72	50.71	52.34	49.96	44.77
Al_2O_3	0.07	0.08	0.09		1.24	0.08	0.03	0.12	0.01	0.19	0.10	0.03
Fe_2O_3	0.03		0.01		0.01				0.03	0.09	0.26	0.22
MgO		0.01	0.04			0.02		0.01	0.01	0.06	0.04	0.04
CaO	11.11	17.37	23.38	22.35	30.61	34.84	21.15	23.38	25.18	27.56	31.78	32.98
Na_2O	3.47	2.70	1.19	1.42	0.50	0.06	1.23	0.20	0.18	0.16	0.18	0.01
K_2O	4.82	4.09	3.02	3.68	2.12	0.32	6.64	2.28	1.43	0.39	0.22	0.13
SO_3	0.02						0.03			0.09	0.06	
Total	81.73	84.61	84.20	86.65	89.60	86.43	79.64	78.70	77.55	80.07	82.55	78.18
Ca/Si	0.19	0.31	0.44	0.40	0.59	0.73	0.45	0.48	0.53	0.56	0.68	0.79

Table 4 Crack-filling ASR gels in cement paste

	Cape Spear From sandstone		Water Street From sandstone				Fort Amherst	Cape Broyle			
An.no:	Cb109	Cb108	Wb43	Wb51	Wb50	Wb52	Fb92	Cc124	Cc126	Cc127	Cc129
SiO_2	52.47	37.94	48.58	38.90	39.25	30.64	35.90	38.59	38.43	36.82	31.99
Al_2O_3	0.05	2.27	0.03	0.11	0.22	1.92	1.65		0.08	1.01	0.23
Fe_2O_3	0.23		0.24	0.23	0.26	0.26	0.19			0.03	
MgO	0.10		0.05		0.03	0.04	0.01	0.04	0.01	0.04	
CaO	27.99	32.82	30.24	28.20	33.14	35.03	39.45	42.05	42.76	42.13	37.68
Na_2O	0.07	0.03	0.10	0.05	0.03						0.25
K_2O	0.17		0.34	0.02	0.03		0.01		0.03	0.02	0.50
SO_3	0.03	0.19		0.06			0.04	0.07		0.03	0.05
Total	81.02	73.34	72.61	67.57	72.95	67.89	77.26	80.74	81.32	80.09	70.71
Ca/Si	0.57	0.93	0.67	0.78	0.90	1.23	1.18	1.25	1.28	1.31	1.35

ASR gels interreplacing C-S-H gels

Large gel veins occasionally replace C-S-H gels in the cement paste, leaving pseudomorphs of cement minerals surrounded by ASR gels. At this stage, both ASR gels and C-S-H gels have the same compositions with a Ca/Si ratio around 1.0-1.2 (***Table6***), indicating that an interreplacement occurs between the two gels (***Fig.5***). At this Ca/Si ratio, an apparent chemical equilibrium is attained between the C-S-H and ASR gels, the former leaching calcium while the latter absorbing the liberated calcium (***Tables 9,10, Fig.11***).

Fig.4 Alteration of ASR gels during migration in concrete

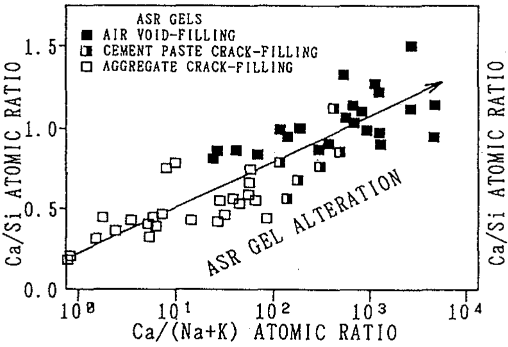


Fig.5 Interreplacement between ASR and C-S-H gels

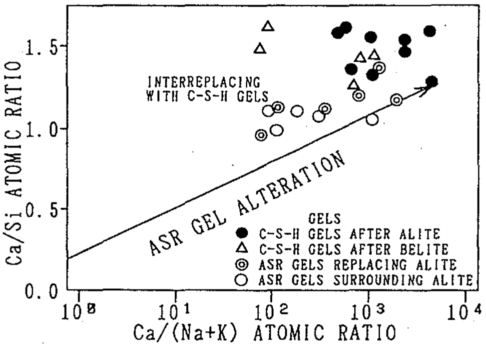


Table 5 Air void-filling ASR gels in cement paste

	Tors Cove Large pore-lining			Water Street Pore-filling			Fort Amherst Pore-filling			Fort Amherst Pore-filling		
An.no:	Tb5	Tb9	Tb10	Wb73	Wb72	Wb74	Fb90	Fb91	Fb88	Fc53	Fc54	Fc55
SiO_2	39.08	38.14	34.58	33.16	31.14	30.04	36.28	34.39	35.87	36.06	35.78	37.26
Al_2O_3	0.98	0.90	0.78	2.37	2.32	2.60	1.92	2.09	1.96	1.37	0.18	0.22
Fe_2O_3	0.13	0.09	0.23						0.40			0.04
MgO	0.01	0.05	0.02	0.03	0.04	0.03	0.05	0.03	0.05			
CaO	31.30	33.15	32.53	30.22	31.31	32.43	36.35	36.73	44.09	36.06	38.94	43.04
Na_2O	0.22		0.09		0.06		0.01		0.03			
K_2O	0.09	0.05	0.02	0.03	0.03	0.05	0.01	0.01	0.04	0.04	0.01	0.01
SO_3	0.33	0.29	0.26	0.23	0.13	0.03	0.07		0.11	0.06	0.02	
Total	72.14	72.66	68.51	66.04	65.02	65.18	74.69	73.25	82.54	73.59	74.92	80.56
Ca/Si	0.86	0.93	1.01	0.98	1.08	1.16	1.07	1.14	1.32	1.15	1.25	1.33

Table 6 ASR gels interreplacing C-S-H gels in cement paste

	Fort Amherst			Tors Cove			Cape Broyle			Water Street		
	ASR*	"alite"		ASR*	"alite"		ASR*	"alite"		ASR*	"alite"	
An.no:	Fc49	Fc41	Fc47	Tb110	Tb151	Tb152	Cc133	Cc134	Cc132	Wb253	Wb249	Wb247
SiO_2	38.80	32.63	31.34	33.94	34.58	30.82	37.45	38.06	35.35	30.35	25.31	24.07
TiO_2		0.04	0.24	0.02			0.05				0.29	0.20
Al_2O_3	0.16	2.22	2.85	1.08	0.65	0.64	0.08	0.28	0.58	2.44	2.44	1.87
Fe_2O_3		0.67	1.33					0.02	0.02	0.09	0.88	0.70
MgO		0.39	0.24	0.07	0.09	0.17	0.01	0.01			3.28	1.92
CaO	36.62	30.53	37.07	35.27	31.08	32.27	38.74	41.25	39.99	33.52	28.02	30.53
Na_2O				0.06	0.15	0.08						0.02
K_2O	0.08	0.18	0.14	0.09	0.12	0.10	0.13	0.11	0.05	0.01	0.01	
SO_3		0.12	0.12	0.03			0.02			0.02	0.03	0.02
P_2O_5		0.05	0.08	0.06	0.01	0.05	0.13	0.26	0.24	0.01	0.15	0.05
Total	75.67	66.82	73.26	70.61	66.67	64.13	76.62	79.99	76.22	66.44	60.40	59.39
Ca/Si	1.01	1.00	1.27	1.11	0.96	1.12	1.11	1.16	1.21	1.18	1.19	1.36

*: ASR gel surrounding replaced alite, "alite": C-S-H gel replaced by ASR gel

CEMENT MINERALS IN CONCRETE

Polarizing microscopy of polished thin sections of deteriorated concrete from Newfoundland, revealed the common presence of unhydrated cement minerals along with hydration products. This method has a greater advantage to our SEM observations, because it is easy to identify mineral species and confirm the textural relationships between original clinker minerals and their hydrates. The EPMA has an optical microscope, convenient for work using thin sections.

Clinker minerals in hardened concretes

It is remarkable that the concrete field structures examined, even 50-70 years old, contain unhydrated alite, belite, aluminate and ferrite, except for water-soluble alkali-sulfates. Calcium silicates alite and belite are rimmed with hydration products in a cluster of cement minerals (*Fig.6*). Aluminate occurs as optically anisotropic elongated crystals in all structures, and was identified as alkali-aluminate because of its optical properties (*Fig.7*).

EPMA analysis indicated that the Ca/Si atomic ratio of unhydrated alite and belite generally exceeds 3.0 and 2.0, respectively, but this is due to solid solutions of these calcium silicates with other elements. Aluminate was confirmed to be the alkali-aluminate variety, containing 3.5% of Na_2O_{eq}. Alkali contents in the original cement (water-insoluble part) were estimated to be about 0.7%, based on the EPMA data (*Table 7*). But the cement alkali was probably higher, because water-soluble alkali-sulfates were also likely contained. Recent investigation showed that a Portland cement from this province contained about 1.0% of Na_2O_{eq} (Rogers 1989).

Fig.6 Partly hydrated alite rimmed with C-S-H gels

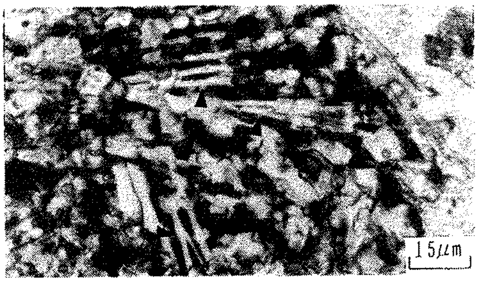


Fig.7 Elongated crystals of alkali-aluminate

Table 7 Unhydrated cement minerals in concrete

	Fort Amherst				Cape Broyle			Water Street		
	Alite	Belite	Alum.	Ferr.	Alite	Belite	Alum.	Alite	Belite	Alum.
An. no:	Fc9	Fb178	Fc13	Fb214	Cc102	Cc113	Cc142	Wb265	Wb259	Wb279
SiO_2	24.15	31.09	3.34	1.52	25.01	30.92	4.85	23.07	28.94	3.82
TiO_2	0.19	0.13	0.28	1.67	0.16	0.20	0.03	0.31	0.36	0.11
Al_2O_3	0.97	1.64	29.23	21.72	0.88	1.50	29.27	1.38	2.14	29.90
Fe_2O_3	0.61	0.94	5.77	22.91	0.52	0.97	4.40	0.65	1.22	4.68
Mn_2O_3				0.51		0.01	0.03	0.05	0.08	0.04
MgO	0.88	0.17	0.59	2.37	1.48	0.52	0.92	1.64	0.33	0.52
CaO	70.82	63.81	53.73	46.77	69.77	60.99	53.74	67.04	58.43	50.70
Na_2O	0.14	0.45	2.54	0.07	0.12	0.71	2.53	0.18	0.60	3.02
K_2O	0.03	0.61	1.25	0.05	0.06	0.70	1.39	0.05	0.50	0.90
SO_3	0.05	0.37		0.02		0.06	0.05	0.04	0.30	
P_2O_5	0.26	0.33	0.08		0.08	0.41	0.14	0.35	0.61	0.02
Total	98.03	99.84	96.81	97.61	98.04	97.02	97.35	94.70	93.52	93.71
Ca/Si	3.14	2.20			2.99	2.11		3.11	2.16	
Na_2O_{eq}*	0.10	0.17	0.34	0.01	0.10	0.23	0.34	0.13	0.19	0.36
	(Total Na_2O_{eq}=0.62%)				(Total Na_2O_{eq}=0.67%)			(Total Na_2O_{eq}=0.68%)		

* Assumed content in cement: alite 60%, belite 20%, aluminate 10%, ferrite 10%

C-S-H gels in hardened concretes

Both alite and belite hydrate to similar, optically amorphous, transparent C-S-H gels, leaving pseudomorphs of their original outlines (***Figs.8,9***). ***Table 8*** shows the results of EPMA analysis of the C-S-H gels, with a distinctive compositional changes during hydration of alite and belite in the deteriorated concretes. These calcium silicates hydrate to a Ca/Si ratio down to 1.2, irrespective of whether they were originally alite or belite, and Ca, Na and K are leached from these silicates while Al and Mg are concentrated.

In cement chemistry, both alite and belite are known to hydrate to a Ca/Si ratio, down to 1.6-1.5 under normal calcium-saturated conditions (Brunauer & Kantro 1964). Thus, the C-S-H gels with a lower Ca/Si ratio ranging 1.5-1.2 in these concretes are suggestive of products under calcium-depleted conditions, such as leaching and weathering. Petrographic examinations revealed that these low Ca/Si gels were actually confined to weathered portions of concrete, severely exposed to freeze/thaw and rain-water percolation around cracks.

Fig.8 Alite pseudomorph of C-S-H gel

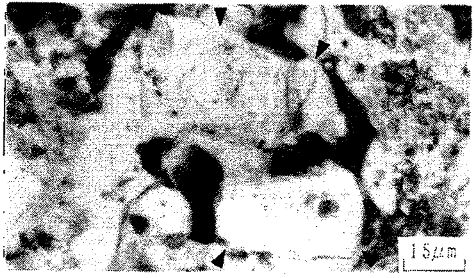


Fig.9 Partly hydrated belite rimmed with C-S-H gels

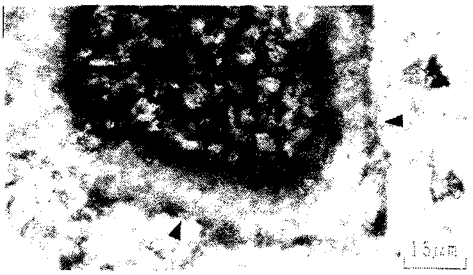


Fig.10 Hydration of alite and belite into C-S-H gels

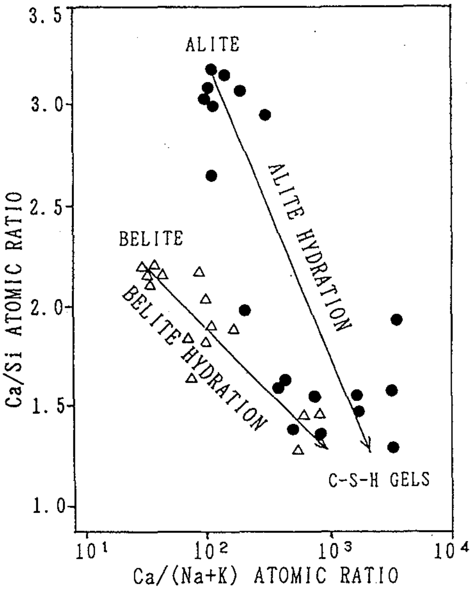


Table 8 C-S-H gels forming pseudomorphs of alite and belite

	Fort Amherst			Water Street			Cape Broyle			Tors Cove		
	Alite pseudomph.			Alite pseudomph.			Belite pseudomph.			Belite pseudomph.		
An. no:	Fc36	Fc37	Fc51*	Wb233	Wb224	Wb261	Cc121	Cc115	Cc119	Tb146	Tb94	Tb89
SiO_2	25.78	30.62	28.54	23.28	24.80	25.56	26.84	26.52	25.97	26.79	28.01	28.76
TiO_2	0.22	0.30	0.29	0.50	0.40	0.22	0.39	0.15	0.50	0.19		0.31
Al_2O_3	1.44	1.83	2.10	2.70	3.04	3.88	3.67	3.09	4.23	2.53	3.10	5.84
Fe_2O_3	0.88	0.80	1.09	0.92	0.81	0.57	1.18	0.62	1.28	0.97	0.56	0.92
MgO	1.07	1.15	2.03	3.44	3.15	5.16	0.70	1.00	2.95	1.99	2.87	3.52
CaO	48.39	45.86	36.21	36.03	34.02	29.63	46.80	42.48	40.36	41.29	38.28	32.23
Na_2O	0.07			0.03			0.09			0.16		
K_2O	0.08			0.08			0.09			0.15	0.04	
SO_3	0.14	0.13	0.16	0.69	0.20	0.21	0.22	0.08	0.43	0.06	0.32	0.32
P_2O_5	0.33	0.30	0.14	0.10	0.29	0.23	0.30	0.15	0.27	0.19	0.27	0.14
Total	78.38	81.01	70.54	67.76	66.70	65.38	80.26	74.08	75.99	74.31	73.46	72.02
Ca/Si	2.01	1.60	1.36	1.66	1.50	1.24	1.87	1.71	1.67	1.65	1.46	1.20

*: Structure surface of core, Wb & Tb: weathered concrete blocks

DISCUSSION

Alteration of ASR gels

A general process of alteration of ASR gels in the deteriorated Newfoundland concretes is shown in ***Table 9.*** ASR gels lose alkalies and gain calcium during migration from reacted aggregate into cement paste: the Ca/Si ratio of the ASR gels increases from 0.2 in the reacted aggregate to 0.8 in the cement paste, and further reaches 1.0 or more after a long contact with cement paste. At a Ca/Si ratio around 1.0-1.2, ASR gels even replace the C-S-H gels which correspond to products of a pozzolanic reaction. Such a transformation of ASR gels into C-S-H gels has been found in a laboratory experiment (Dent Glasser & Kataoka 1982), but little has been documented from field concretes.

Hydration of calcium silicates

In the normal hydration process(***Table 10***), both alite and belite hydrate to C-S-H gels, down to a Ca/Si ratio around 1.5, which are equivalent to the final hydration products of these calcium silicates under normal calcium-saturated conditions (Brunauer & Kantro 1964). However, where a weathering process dominates near the concrete surface, calcium is continuously leached from the C-S-H gels down to a Ca/Si ratio 1.2, and such liberated calcium is absorbed by ASR gels in concretes. Excess leaching of calcium may form stalactites and secondary coatings, composed of calcium carbonate on the structure surface.

Apparent equilibrium between C-S-H and ASR gels

Fig.11 shows the entire processes of alteration of ASR gels and hydration of alite and belite in the deteriorated Newfoundland concretes. Final products of these alteration fall in the narrow range of the Ca/Si ratio around 1.0-1.2, intermediate between typical ASR and cement hydration. They correspond to products of a pozzolanic reaction predicted by Urhan (1987) to occur in concretes undergoing ASR. Probably, these pozzolanic products are stable under calcium-depleted conditions, such as weathering due to rain-fall and freeze/thaw. The Ca/Si ratio 1.0-1.2 represents an apparent chemical equilibrium between the leaching process of calcium from the C-S-H gels and the absorbing process of this liberated calcium by the ASR gels in concretes.

Table 9 Alteration of ASR gels during migration

Migration → Weathering

ASR gels → Pozzolanic C-S-H gels

	Aggregate	Cement paste		
	Crack-filling	Crack-filling	Air void-filling	C-S-H replac.
Ca/Si	0.2 →	0.8 →	1.0 →	1.1

Ca: leached from C-S-H gels & absorbed by ASR gels

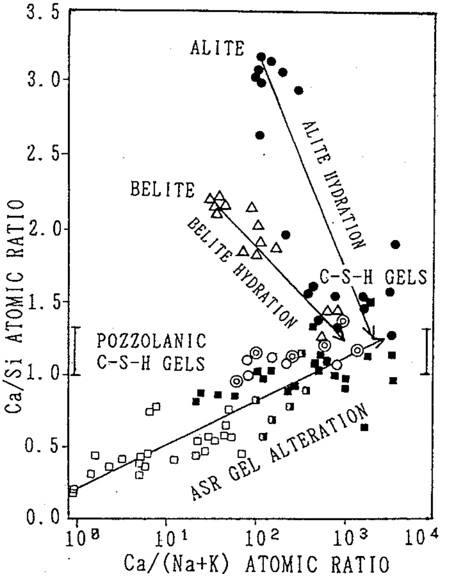
Na, K: leached out from concrete

Table 10 Alteration of cement hydrates during weathering

	Cement	Hydration → C-S-H gels Ordinary	Weathering → C-S-H gels Pozzolanic
Alite Ca/Si	3.1 →	1.5 →	1.2
Belite Ca/Si	2.1 →	1.6 →	1.2

Leached Ca → absorbed by ASR gels
→ form stalactite & coating

Fig.11 Chemical changes of gels in deteriorated concretes



CONCLUDING REMARKS

EPMA analysis of old deteriorated concrete structures in Newfoundland, undergoing ASR and freeze/thaw weathering, presented the following chemical and petrographical basis for diagnosing deterioration processes in concretes.

- Gel products in the deteriorated concretes have a continuous range of composiitons from typical cement hydrates to typical ASR gels, particularly in the weathered portions exposed to freeze/thaw and rain-water percolation.
- Intermediate gel products, having a Ca/Si ratio of 1.0-1.2 and corresponding to products of a pozzolanic reaction, were formed at an apparent equilibrium between the C-S-H and ASR gels, due to long-term weathering of concretes.
- In this alteration process, calcium was leached from the C-S-H gels and entered ASR gels, transforming the latter gels chemically into pozzolanic C-S-H gels. Possibly, these pozzolanic C-S-H gels are stable and harmless to concrete under weathering conditions.
- Long-term weathering by freeze/thaw, coupled with rain-fall, likely enhances water-circulation, leaching and migration of soluble elements in concretes.
- Petrographic examinations coupled with EPMA analysis made it possible to estimate the cement properties in old hardened concretes, such as the water-insoluble alkali content of the cement used even more than 50 years ago.

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ALKALI-AGGREGATE REACTIONS IN THE CEMENT CONCRETE PAVEMENTS OF AIRPORT

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ABSTRACT

The concrete of many airport pavements in north China has been deteriorated since 1970's. The causes of deterioration were investigated. Results show that the concrete was mainly damaged by alkali-aggregate reactions. Measures to prevent newly constructed concrete from being destroyed were evaluated.

***Key words*:** Airport, alkali-aggregate reaction, concrete, preventing measures

INTRODUCTION

The pavements of airport in China were constructed almost with cement concrete. The designed life span of them was 20-30 years or more. This life was based on the strength of concrete structures rather than the service life of concrete itself, which may practically be more than 40 years under the conditions of safe loads and use in scheduled circumstances. However, some pavements constructed during the past 25 years showed deterioration of concrete in 3-5 years. Field surveys show that pavements of 16 airport on the north of Yangtze river were damaged by physical or chemical factors. In some airport, 30% pavements were cracked, scaled or even loosened. While the pavements in some other airport were slightly affected.

To evaluate the causes of deterioration, concrete cores in 10-15 cm diameter were drilled from the pavements of 6 airport. Carbonation proceeded only 1.0-1.5 cm from surface to interior, and was not considered as a major cause of the deterioration. The aggregates did not contain gypsum and ferro sulphides, and mortars were not attacked by sulphate except that in the some pavements of one airport, as indicated by x-ray diffraction analysis (XRD), differential thermodynamic analysis (DTA) as well as petrographic examinations (Shu et al. 1993). The airport localized in the North of China where the average temperature ranged from -16 ℃ to 0 ℃ in the coolest month. However, the deterioration of concrete did not reveal the characteristics of freezing and thawing. The concrete was not reinforced and might not be affected by corrosion of steel.

Thus, it was suspected that the damage of concrete pavements was mainly due to alkali-aggregate reactions — another factor that lowered durability of concrete. This paper describes the evaluation of alkali-reactivity of aggregates, petrographic examinations of concrete cores and prevention of alkali-carbonate reaction.

ALKALI-AGGREGATE REACTION IN PAVEMENTS OF AIRPORT

The aggregates used in one of the damaged airport were collected, and were submitted to tests of alkali-reactivity. The concrete cores derived from the airport were petrographically examined.

Components and microstructures of aggregates

The fine aggregate was river sands composed of well-crystallized quartz and feldspar. These minerals do not bring about concrete to be damaged by alkali-aggregate reaction.

The coarse aggregates come from two local quarries in the east of Shandong and were designated as JD and CL in this paper, respectively. They both were crushed slightly dolomitic limestones. Their chemical compositions are shown in Table 1. The aggregates consisted of calcite, dolomite and a very small amount of quartz according to x-ray diffraction analysis (XRD). The contents of dolomite and calcite listed in Table 1 were calculated by assuming that CaO and MgO were completely combined with CO_2 to form calcite and dolomite. Petrographic and SEM-EPMA examinations revealed that the dolomite was localized in strips that composed of 10-40 μm dolomite, 4-8 μm calcite and 4-20 μm quartz and feldspar.

Table 1 The chemical and mineral compositions of coarse aggregates

Sample	Chemical composition (%)							Mineral (%)			
	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	L.O.I	Dolomite	Calcite	Residue	Total
JD	4.56	1.21	0.48	50.49	1.70	0.30	41.24	7.77	86.03	6.25	99.98
CL	4.19	1.14	0.45	51.39	0.89	0.40	41.48	4.07	89.64	5.79	99.50

Alkali-reactivity of aggregates

The fine and coarse aggregates were subjected to micromortar bar test (CECS 48-93) to evaluate their alkali-silica reactivity. The aggregate JD was separated into JD1 and JD2 according to the difference in color. Similarly, the aggregate CL was separated into CL1, CL2 and CL3. Results are shown in Table 2. Expansions of sand and rocks JD1, JD2, CL1 and CL2 are far below the limit 0.10%. Therefore, they were not alkali-silica reactive. The rock CL3 contained more acid insoluble residue in which quartz was in several to 20 μm, and was alkali-reactive. The reactive rock CL3 occupied a small portion in coarse aggregates.

The coarse aggregates were also subjected to rock prism and microconcrete bar tests to determine their alkali-carbonate reactivity. The rock prisms were in 10mm ×

Table 2 Results of micromortar bar test on the aggregates

Ratio of cement to aggregate	Expansion (%)					
	Sand	JD1	JD2	CL1	CL2	CL3
10	0.040	0.032	0.055	0.038	0.051	0.065
5	0.048	0.046	0.046	0.047	0.066	0.070
2	0.051	0.063	0.063	0.057	0.068	0.121

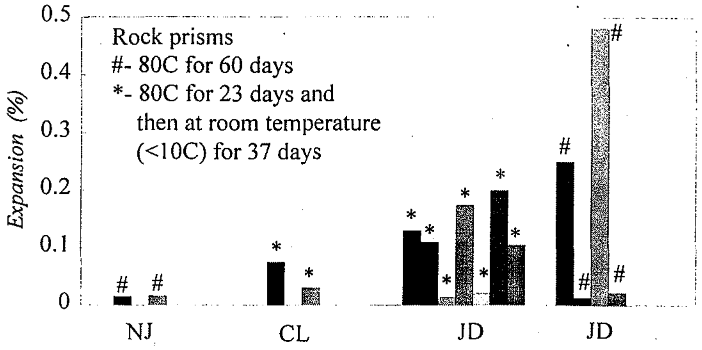

Fig. 1 Expansions of rock prisms cut from the coarse aggregates

10mm × 32mm and were stored in 1mol/L KOH solutions at 80℃. Expansions of prisms cured for 60 days are demonstrated in Fig. 1. The rock NJ was nonreactive dolostone from Nanjing used for control sample. It is obvious that the coarse aggregates were expansive. Some prisms cracked in the region richened in dolomite. This seems to indicate that the coarse aggregates were alkali-carbonate (dolomite) reactive.

The relationships between expansion of 20mm × 20mm × 60mm microconcrete bars (Tang et al. 1994) and autoclave time are shown in Fig. 2. Size of aggregate in the specimens was 5-10mm. Ratio of cement to aggregate was 1. W/C was equal to 0.30. The alkali content of Portland cement was boosted from 0.43% to 1.50% Na_2Oequiv. by adding KOH into the mix water. The bars demolded after 24 hours were precured in 100℃ steam for 4 hours, and then autoclaved in 10% KOH solution at 150℃. The aggregate NJ is nonreactive dolostone from Nanjing used for control sample. The concrete with NJ expanded very small. However, the aggregates CL and JD both caused a large expansion in concrete. The bars were cracked after autoclaving for 6 hours. Examination of polished samples shows that the cracks either originated the coarse aggregate particles or extended from interior of aggregate particles to mortar. These manifest that the aggregates were alkali-carbonate (dolomite) reactive.

Petrographic examinations of concrete cores

The concrete cores drilled from the pavements of airport were cut and polished. Many cracks occurred in the whole 180mm thick concrete pavements. It is observed on the polished plates and thin sections that the cracks originate from the coarse aggregates and a few of coarse aggregate particles suffered cracking. The cracked portion of the aggregate particles was rich in dolomite. This appears to indicate that the cracks

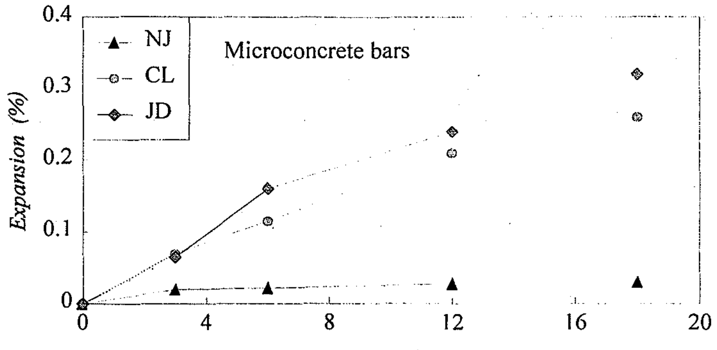

Fig. 2 Expansion of microconcrete bars vs. autoclave time

was caused by expanding of the aggregate particles due to alkali-carbonate reaction. The fine aggregate, crystals of quartz and feldspar, was not attacked and did not cause distress.

PREVENTING OF ALKALI-CARBONATE (DOLOMITE) REACTION

For the highly reactive dolomitic limestone from Kingston, Canada, low alkali Portland cement and mineral admixtures such as slag and fly ash are ineffective in suppressing expansion due to alkali-carbonate (dolomite) reaction (Deng et al. 1993 & Rogers. 1992). Compared with the Kingston reactive rock, the rocks used as coarse aggregates in the airport are moderately reactive. In this part, the effectiveness of mineral admixtures and low alkali cements in minimizing expansion due to these moderately reactive rocks was investigated

Materials and procedure

The low alkali cements used were Portland cement (OPC), sulfo-aluminate cement (SAC) and ferroaluminate cement (FAC). Their alkali contents were 0.51%, 0.32% and 0.18% Na_2Oequiv., respectively. Pulverized fly ash (PFA) and granulated blast furnace slag (BFS) were used, which both were commercially available. Their chemical compositions and specific surface area are shown in Table 3. The reactive rocks were

Table 3 The chemical compositions and specific surface area of materials

Material	Chemical composition (%)										Blaine
	SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	SO_3	Na_2O	K_2O	L.O.I	Total	(m^2/kg)
OPC	20.72	4.91	4.23	61.39	1.23	2.30	0.29	0.33	4.08	99.48	313
SAC	8.92	1.72	31.32	38.54	1.83	11.49	0.10	0.33	4.09	98.34	400
FAC	9.89	4.99	23.97	41.18	1.39	12.12	0.08	0.15	5.61	99.38	412
PFA	48.12	4.60	32.10	3.77	1.15	1.36	0.62	1.05	6.50	99.27	433
BFS	35.52	3.24	11.08	38.82	9.07	0.40	0.34	0.55	-	99.02	443

the coarse aggregates used in the airport. Nonreactive dolostone from Nanjing, China was used for control sample. Sand was nonreactive.

Concrete specimens were cost in 20mm × 20mm × 60mm. Each set composed of 6 concrete prisms. The coarse aggregate was in 5-10 mm. Cementitious materials: rock: sand: water=1:1.5:1:0.35. The specimens were cured in 20℃ moist room for 24 hours. Then, they were demolded and cured at 40℃ & 100% RH.

Results

To assess the effectiveness of low alkali cements, PFA and BFS on minimizing the expansion due to alkali-dolomite reaction, the difference between length changes of concrete prisms with reactive aggregate and with nonreactive aggregate was calculated. In question, this difference may approximately be considered as the expansion caused by the reactive aggregate.

Table 4 shows the expansion brought about by the reactive aggregate in the three low alkali cements and one high alkali cement HPC. The sample HPC was derived from the low alkali Portland cement by boosting its alkali content to 1.50% Na_2Oequiv. The moderately reactive rocks in concrete made with the high alkali Portland cement caused a large expansion and cracked the concrete prisms. When incorporated in concrete made with low alkali cements OPC, SAC and FAC, they caused almost no expansion. On other word, low alkali cements OPC, SAC and FAC may effectively contract the expansion of moderately reactive rocks in concrete at least in the test duration.

The effectiveness of PFA and BFS on inhibiting alkali-dolomite reaction of the moderately reactive rocks is demonstrated in Table 5. PP30 and PP50 were low alkali

Table 4 The influence of cements on expansion of moderately reactive rocks in concrete

Cement	Expansion (%)							
	30d	60d	120d	180d	270d	360d	550d	720d
HPC	0.028	0.022		0.033		0.099		
OPC	0.004	0.005	0.008	0.005	0.007		0.004	0.016
SAC	0.003	0.002	0.002	0.000	-0.021		-0.018	-0.024
FAC	0.007	0.010	0.008	0.007	0.006		0.000	0.002

Table 5 Influence of PFA and BFS on expansion of alkali-dolomite reaction

Cement	Expansion (%)							
	30d	60d	120d	180d	270d	360d	550d	720d
HPC	0.028	0.022		0.033		0.099		
OPC	0.004	0.005	0.008	0.005	0.007		0.004	0.016
PP30	0.000	0.000	0.000	-0.005	-0.005		0.005	0.007
PP50	0.004	0.006	0.008	0.006	0.000		-0.005	-0.015
PB30	0.003	0.003	0.001	0.004				
PB50	0.002	0.000	0.001	-0.001	-0.001		0.012	0.016

Portland cement (OPC) with 30% and 50% PFA, respectively. PB30 and PB50 were OPC with 30% and 50% BFS, respectively. The results seem to show that low alkali Portland cements with 30-50% PFA or BFS may effectively minimize the expansion due to alkali-dolomite reaction of moderately reactive rocks during the test period.

The deterioration of alkali-aggregate reaction usually occurs in a long time, especially for those moderately and weakly reactive aggregates. The effectiveness of the low alkali cements and mineral admixtures needs to be observed further.

CONCLUSION

The concrete pavements of airport in China have suffered deterioration of alkali-carbonate (dolomite) reaction. The small amount of alkali-silica reactive aggregate might distress the concrete. Low alkali sulfoaluminate cement, ferro-aluminate cement, Portland cement and Portland cement with 30-50% PFA or BFS may significantly restrain the expansion due to alkali-carbonate (dolomite) reaction of the moderately reactive rocks.

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ALKALI AGGREGATE REACTION IN WESTERN AUSTRALIA: INVESTIGATIONS ON THE CAUSEWAY BRIDGE AND SOME AGGREGATE SOURCES

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ABSTRACT

Alkali-aggregate reaction (AAR) was identified in the Causeway Bridge over 10 years ago, and because of this the Main Roads Western Australia became involved in investigations related to various aspects of AAR, through the following programs in collaboration with CSIRO.

1. The microstructural characterisation of the concrete and the reaction products and the identification of reaction phases.
2. Determination of residual expansion of the affected concrete, with a view to repair the structure at a suitable time.
3. Assessment of various aggregate sources of interest to Main Roads for AAR, development of test methods and criteria for rapid evaluation of their alkali-reactivity and determination of alkali tolerance of the aggregates.
4. Assessment of mineral additives to suppress the AAR expansion of concrete made with the identified reactive aggregates.

The paper provides data relevant to the above aspects and suggests an approach for dealing with future Main Roads projects to ensure durability of their concrete structures.

Keywords: alkali aggregate reaction, ettringite, concrete cracking, expansion, testing, fly ash.

INTRODUCTION

The Causeway Bridge was constructed during 1940-51 using two types of aggregate from the same quarry (granitic and metadolerite) and several types of cement, some imported from overseas. The presence of AAR in the Causeway Bridge, Perth, was suspected after an inspection late in 1982, and confirmed after drilling and examination of cores from various parts of the structure in 1983. The findings were reported elsewhere (Shayan & Lancucki, 1986), and provided some microstructural details on the affected concrete as well as mineralogical and chemical data on the reaction products. The two aggregate types used in the concrete, ie. a metadolerite and a gneissic granite were both found to have reacted in the concrete. Certain parts of the bridge containing the same aggregates but presumably having low alkali cement, did not show any sign of AAR.

These findings prompted the Main Roads to investigate some of its potential aggregate sources, and as a result of this work, done at CSIRO during 1983/84, some reactive aggregates were identified (Shayan, et al., 1986). It was found that the existing Australian Standard test methods for determining the alkali-reactivity potential of aggregates (AS1141 sections 38, 39) were inadequate. Subsequently more rapid test methods and criteria were developed for assessing the slowly reactive Australian aggregates, (Shayan et al., 1988; Shayan, 1989).

This paper provides details of a more recent investigation of the Causeway Bridge, including the measurement of residual expansion of drilled cores. Additional studies of the reactivity of WA aggregates and prevention of their AAR expansion in concrete are discussed.

RECENT INVESTIGATIONS OF THE CAUSEWAY BRIDGE

An inspection of the bridge in 1992, nearly ten years after initial identification of AAR showed that cracking in some zones have worsened and crack width increased considerably. Spalling at joints had occurred in a number of locations. Figure 1 shows some features of the cracking observed in the 1992 inspection. In order to initiate a repair strategy, it was decided to determine the residual expansion potential of the concrete from various parts of the bridge, using drilled cores. Figure 2 shows the condition of the concrete in various zones where cores were drilled.

Fig. 1. **Some features of cracking in various parts of the bridge, observed in 1992. Left and middle pictures show cracking in the top of a pylon, where some parts of the concrete have become separated (arrow). The right picture shows map cracking in the ground section of the pylon with some spalling.**

These cores were about 350mm long and 100mm in diameter and correspond to those locations sampled for the earlier study (Shayan and Lancucki, 1986). The composition of the cores was the same as the earlier cores, confirmed by petrographic examination.

MICROSTRUCTURE OF CORES

Cores 1, 2, 3, 4 and 5 contained reacted and cracked aggregate particles, with Core 2 being the most internally cracked; consistent with the appearance of the concrete in that location. No evidence of AAR was seen in Cores 6 and 7. Details of scanning electron microscope (SEM) examination and a variety of crystalline and gel-like reaction products found on fracture surface of cores from affected zones were reported in the 1986 study. In this work sawn surfaces of 10mm slices cut from the cores were examined. Therefore, crystalline materials normally seen as reaction rims on fracture surfaces of reacted aggregate particles had been removed, and only gel phases could be observed. This was to enhance any veins of reaction products that could be present.

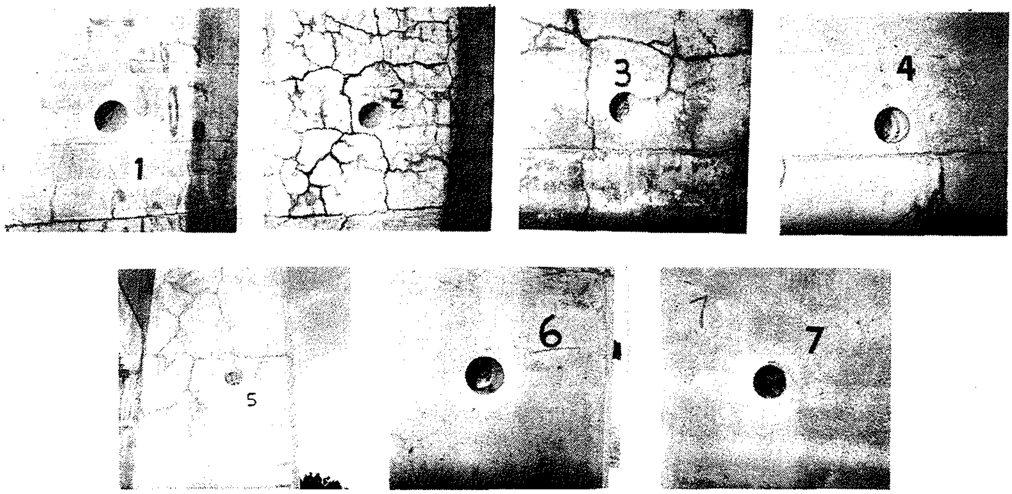

Fig. 2. **Condition of concrete in various zones where cores were taken. Cores 1 - 5 came from the east abutment pier and columns, and cores 6, 7 from the west abutment columns.**

The SEM examination revealed that in the concrete samples from reacted zones the mortar phase was impregnated with AAR gel generated in the reacted particles. This feature was more prevalent in the more extensively affected concretes. Even Cores 1 and 4 from the vicinity of cracked zones contained considerable amounts of gel. Cores 6 and 7 were free of AAR gel. Figure 3 shows examples of the gel in the mortar phase. These gels particularly those that were present in the cracks within reacted aggregate particles (ie. less access to Ca from the cement phase) are highly rich in silica and alkali, largely potassium. Some gels present on the mortar phase had a considerable amount of Ca as a result of contact with this phase.

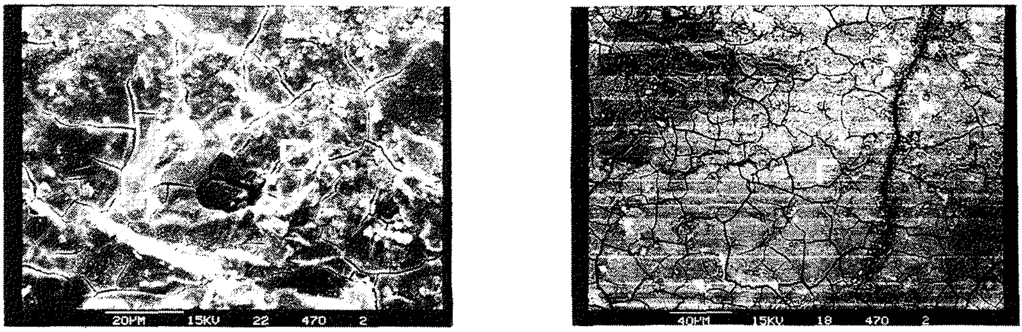

Fig. 3. **Scanning electron micrographs showing AAR gel on sawn surfaces of core slices. (A) secondary electron image, (B) back scattered electron image. For more microstructural features and compositions see Shayan and Lancucki (1986).**

In addition to the AAR products reported by Shayan and Lancucki (1986), this study provides details of the ettringite phase in the AAR affected samples. Cores 1 and 4 contained ettringite needles largely in holes and cracks, but massive ettringite mats in veins were not common, whereas no ettringite mats was noted in Core 3, 6 and 7. In Cores 2 and 5, where large amounts of AAR gel are present, mats and veins of ettringite were common, and were found both at the mortar/aggregate interface and in veins in the mortar phase. Figure 4 shows examples of the ettringite phase in the various cores.

This form of ettringite is a consequence of AAR cracking (Shayan and Quick 1992), and does not cause expansive forces (Scrivener and Taylor 1993) nor deterioration of the concrete.

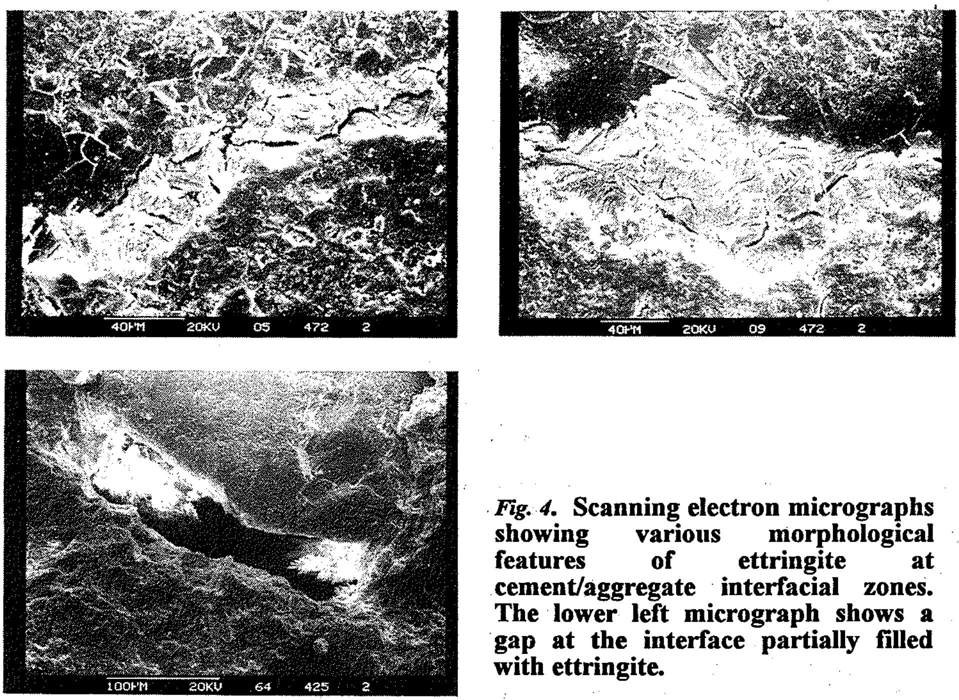

Fig. 4. **Scanning electron micrographs showing various morphological features of ettringite at cement/aggregate interfacial zones. The lower left micrograph shows a gap at the interface partially filled with ettringite.**

RESIDUAL EXPANSION OF CORES

Figure 5 shows representative expansion curves for the cores under the two different storage conditions indicated on the curves. The curves show that the specimens stored in the NaOH solution expanded deleteriously and cracked, or further cracking developed in those taken from the cracked zones (or existing micro cracks widened). Cores from locations 1-5 behaved similarly whereas cores from completely uncracked location 6 and 7 expanded less in the NaOH solution, probably because of either smaller amounts of reactive components or use of low alkali cement, or both, in these parts and/or higher strength of the mature concrete.

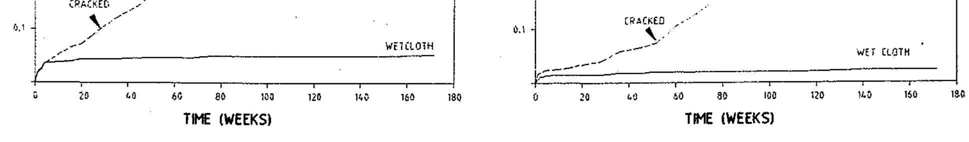

Fig. 5. **Expansion curves for cores under the two different storage conditions. Cores from locations 1, 2, 4 and 5 behaved similarly to those from location 3 (left). Cores from location 7 expanded half those from location 6 (right).**

Expansion in the NaOH solution indicates that reactive components are still present in the concrete, and further expansion may result if alkali penetrates into the concrete.

Lack of expansion under the moist conditions, without excess alkali, may indicate that expansion has ceased in the concrete due to consumption of some of the alkali in the original concrete, although stress relief during the time elapsed between drilling and start of the measurements may have dissipated some of the expansion potential. As indicated earlier possible leaching of alkali during the brief soaking period may also have contributed to the lack of expansion.

ON-SITE MEASUREMENT OF EXPANSION

Demec disks were fitted to the structure at several locations both across cracks and on the adjacent uncracked positions. The strains measured over the past twelve months indicate that a number of the cracks have ceased to move whilst in the case of others the situation has been less clear. Some readings have shown significant variations making judgements on crack stability difficult to assess. To supplement the data from the demec readings a short length of crack was filled with a weak material (poly filler) to determine visually if the cracks were widening. None of the filled cracks has shown any sign of opening after a six month period, although this is too short to make a judgement.

It appears that much of the expansion potential of the AAR affected concrete has been exhausted. The existing conditions of the concrete calls for limited repairs to be undertaken to inject and seal the cracks with suitable materials to prevent secondary deterioration of the concrete materials and the steel reinforcement bars.

ASSESSMENT OF WA AGGREGATE SOURCES

In previous studies using both conventional (Shayan et al., 1986) and accelerated methods (Shayan et al., 1988; Shayan, 1989) three siliceous river gravels (HC, TKA and SPG) two metadolerites (GSN and HHM), a sandstone (NBB) showing the pessimum effect and a granitic rock (GSG) were identified as reactive. However, another granite rock and two metadolerites (TRM and JDM) were classed as non-reactive.

The initial work employed a concrete mix containing 407 kg cement/m^3 and cement alkali levels of 0.84 and 1.38% Na_2O equivalent, yielding concrete alkali contents of 3.4 and 5.6 kg Na_2O/m^3.

To assess the behaviour of the aggregates under much higher alkalinity a new concrete mix containing 580kg cement/m^3, W/C = 0.4, and a cement alkali level of 1.84% Na_2O equivalent was used. This yielded a concrete alkali content of 10.7 kg/m^3. A similar mix containing a low Ca fly ash (25% by mass of cement) was used to evaluate the effectiveness of the fly ash in preventing AAR expansion.

Concrete prisms (75 x 75 x 285mm) and concrete blocks (300 x 300 x 300mm) were made from mixes without and with fly ash. The specimens were demoulded 24 hours after casting, and after one week of curing in a fog room at 23°C (protected from leaching under plastic cover) were placed in containers of water and stored in a room at 50°C. The initial length measurement was made twenty-four hours after conditioning at 50°C, and all the subsequent measurements were also made at this temperature for about two years. A non-reactive basalt aggregate was treated similarly for comparison.

Figure 6 shows expansion curves for the concrete blocks and prisms. Except for the sandstone (NBB) and the non-reactive basalt, all the aggregates caused deleterious expansions, and cracking occurred in concrete blocks at ages varying from 3.5 to 13 months. Similar results were obtained for concrete prisms except that the shapes of the expansion curves were different and cracking ages did not correspond to those of concrete blocks. In addition, one of the metadolerites (TRM) did not cause cracking in concrete prisms. The flyash was effective in reducing AAR expansion (Fig. 6C)

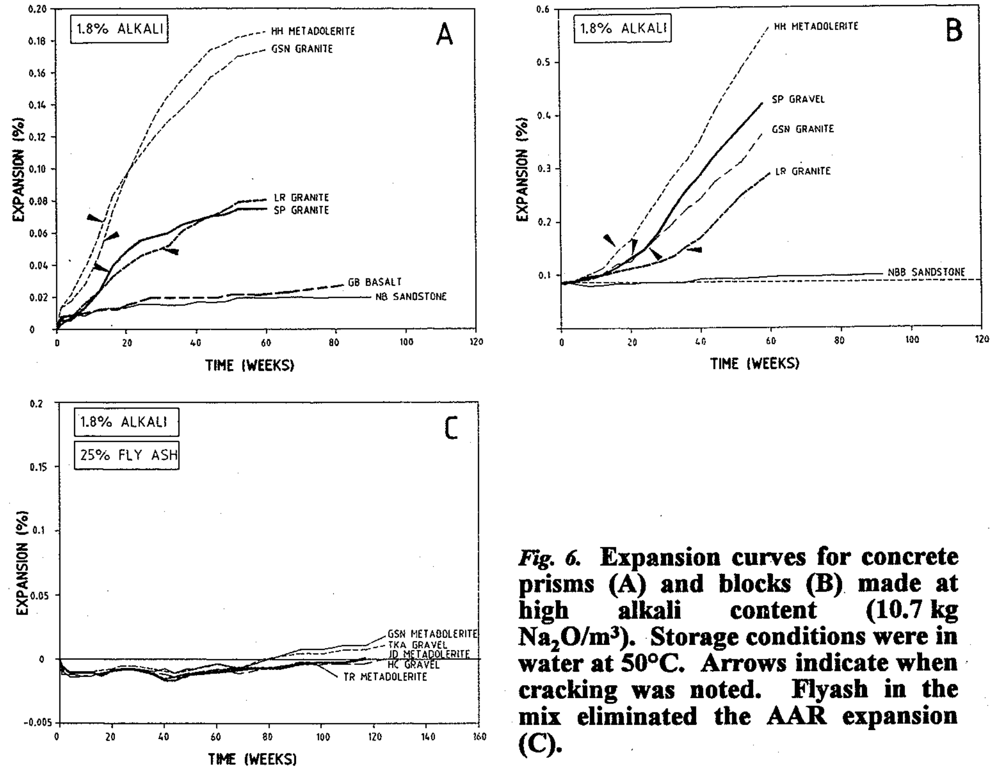

Fig. 6. **Expansion curves for concrete prisms (A) and blocks (B) made at high alkali content (10.7 kg Na_2O/m^3). Storage conditions were in water at 50°C. Arrows indicate when cracking was noted. Flyash in the mix eliminated the AAR expansion (C).**

These results show that the concrete mix containing 580kg cement/m^3 was a more severe test than the accelerated test in 1M Na OH, as more aggregates were classed as reactive. The water/cement ratio of 0.4 and cement alkali level of 1.84%, yields a pore solution of about 1.5M Na OH concentration, compared to 1M NaOH in the accelerated test, making the concrete test more severe.

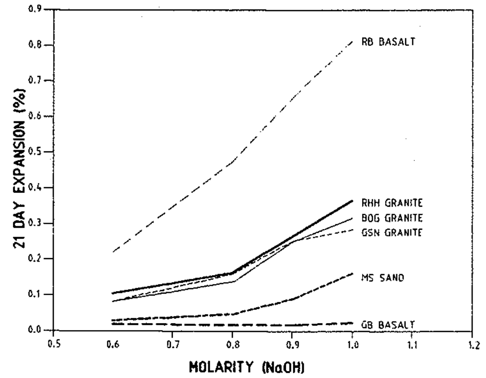

Fig. 7. **Results of accelerated testing of mortar bars made with the various aggregates at different Na OH concentrations (80°C).**

ASSESSMENT OF ALKALI TOLERANCE OF W.A. AGGREGATES

In more recent studies a short-term and a long-term approach have been adopted to determine the alkali tolerance of some WA aggregates with the aim of correlating the two and validating the short-term approach. In the former, the mortar bar accelerated test (Shayan et al. 1988) was applied at NaOH concentrations of 0.6, 0.8, 0.9 and 1.0 M and 21 day expansion values measured. Considering the cement content used and W/C = 0.43, the NaOH concentrations employed correspond to concrete alkali contents of 3.2, 4.4, 5.0 and 5.5 kg Na_2O equivalent / m^3. In the long-term tests, a concrete mix (410 kg cement/m^3, W/C = 0.43) was used at alkali contents of 2.3, 3.0, 4.0, 5.5 and 6.5 kg Na_2O equivalent/m^3 to manufacture concrete prisms (75 x 75 x 285 mm), blocks (300 x 300 x 250 mm) and blocks (300 x 300 x 300 mm) for storage at 40°C, 100% RH; 50°C in water and outdoor exposure, respectively, and to measure their long-term expansion.

Figure 7 presents the results of the accelerated tests and clearly shows the different tolerances of the various aggregates. The failure criteria for NaOH concentrations for lower than 1.0 M would be different from the 0.1% expansion at 21 days because some reactive aggregates would certainly not expand to 0.1% in 21 days in 0.6M NaOH solution. This lower concentration corresponds to a cement alkali level of 0.8% Na_2O equivalent, at which deleterious expansion is to be expected when using these aggregates in the proposed concrete mix.

The program needs to be extended to cover lower NaOH concentrations of 0.5M and 0.4M to correspond to concrete alkali contents of 3.0 and 2.30 kg Na_2O equivalent/m³ in the proposed concrete mix.

Figure 8 shows expansion curves for concrete prisms stored at 40°C, 100% RH. The three granite aggregates behaved similarly at the age of 60 - 70 weeks, concrete prisms that contained the granite aggregates at the two highest alkali levels have expanded deleteriously and cracked as shown by the arrows (Fig. 8A). Surprisingly, the basalt that had shown high sensitivity to alkali in the accelerated test has not caused much expansion in the concrete prisms, even at high alkali levels (Fig. 8B). The reason for this difference may be a particle size effect. Petrographic examination of the basalt has shown that it contains patches of glass which within one coarse aggregate particle are not exposed to alkali. Crushing of these coarse particles produces a finer particle size which exposes the glassy patches, and these are exposed to alkali attack in the accelerated test. This hypothesis needs further investigation.

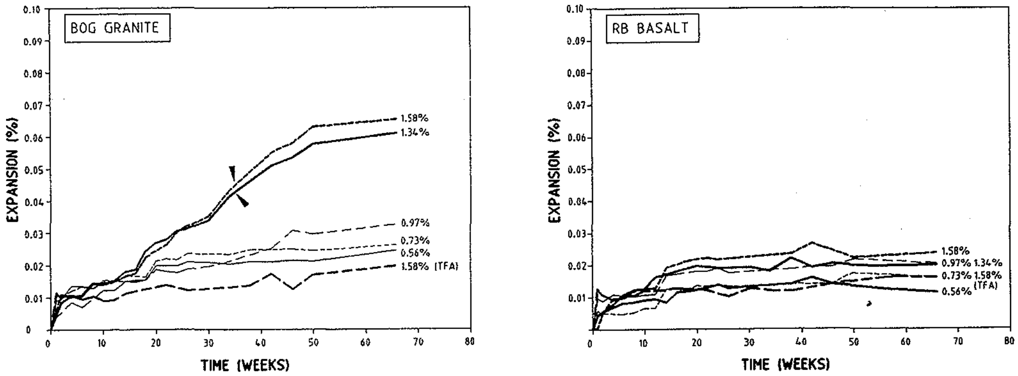

Fig. 8. **Expansion curves for concrete prisms made at different alkali contents and stored at 40°C, 100% RH. Arrows indicate when cracking was noted.**

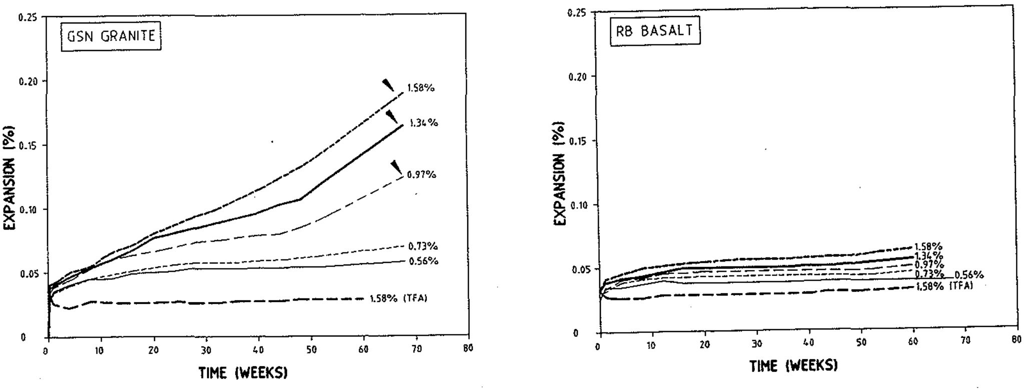

Fig. 9. **Expansion curves for concrete blocks stored in water at 50°C. Numbers on the curves indicate cement alkali level. Cement content of concrete was 410 kg/m³. TFA indicates 25% fly ash replacing cement on mass basis. Arrows indicate when cracking was noted.**

Figure 9 shows expansion curves for the corresponding concrete blocks, stored in water at 50°C. Specimens made with two of the granites (GSN and BOG) which are nearly 79 weeks old have cracked at alkali levels down to 0.97%. Those made with the other granite (PHH) are younger and may crack at a similar age, but those made with the basalt have shown much less expansion, as for the concrete prisms.

Expansion of concrete prisms made at alkali levels below 1.35% and concrete blocks at alkali levels below 0.97% have been slow and longer term expansion data are required for reliable interpretation of the expansive behaviour of concrete mixes at these alkali levels. The correlation with the accelerated test would also need to wait until the concrete data for low alkali level is available.

CONCLUSIONS

Research work carried out on ARR in Western Australia has shown that:

1. The premature deterioration of the Causeway bridge has resulted from AAR, and that both the granitic and metadolerite aggregates used in the concrete are reactive. Other secondary reactions like formation of ettringite in the AAR - induced spaces are taking place although this may not be of much consequence. There are indications that expansion of the AAR - affected concrete has ceased, but this would need confirmation. Appropriate remedial action is recommended.
2. A number of Western Australian aggregates which are of interest to the Main Roads Department have been identified as potentially reactive in concrete of high alkali contents. However, incorporating 25% of a low Ca fly ash, replacing cement on a mass basis, eliminated the AAR expansion. Use of effective mineral admixtures is recommended.
3. Work on determining safe alkali levels for potentially reactive aggregates, using both concrete specimens and the accelerated mort bar test, is in progress and promising results have been obtained. Further longer term data are needed for correlating the results of the accelerated test with those of the concrete tests in order to determine whether the accelerated procedure can be used to estimate safe alkali levels for aggregates in concrete.

ACKNOWLEDGMENTS

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RAILWAY TIES AFFECTED BY ALKALI-AGGREGATE REACTIONS

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ABSTRACT

A study was conducted on a railway tie showing longitudinal cracking between the rail seats and random cracking in the ends beyond the rail seat. Results show that reactive siliceous and dolomitic aggregates were used, and that deterioration of the concrete was primarily due to the combination of alkali-silica reaction and alkali-dolomite reaction.

Keywords: Alkali-aggregate reaction, deterioration, railway tie

INTRODUCTION

Railway ties manufactured at a plant in Henan province, China were showing random cracking in the ends beyond the rail seat and longitudinal cracking between the rail seats. Some of them were then broken up. These railway ties were produced during 1983-1984 and were put into service in late 1985 or early 1986. The on-line and spare (set beside) ties began to crack in 1987. The cracking continues to extend. A field survey carried out in 1990 shows that more than 90% railway ties were damaged. The railway ties laid in Northeast and North China were more seriously deteriorated than that placed in the central China. Railway ties used in the same places from other plants were in good condition.

The cracking of the spare ties shows that the deterioration was not caused by load. The railway ties cracked in a temperate climate of the central China. This seems to indicate that the damage was not primarily due to freezing and thawing. However, the low temperatures (lower than -10℃) in the winter of the Northeast and the North may aggravate the deterioration. It was suspected that the cracking was due to alkali-aggregate reactivity or sulfate attack.

BACKGROUND INFORMATION

The concrete railway ties had been made with ordinary Portland cement from Pingdingshan, Henan province. Alkali and SO_3 contents of the cement were 1.19% Na_2Oequiv. and 2.95-3.10%. C_3S, C_2S, C_3A and C_4AF were 49.73%, 23.61%, 7.40% and 13.88%, respectively. The fine aggregate was derived from rivers. Its fineness module was 2.2. The coarse aggregates were crushed calcitic dolostone, dolomitic limestone and metamorphic quartzite. They were in 5-20 mm. The contents of cement, sand, rock and water in 1 m^3 concrete were 510 kg, 523 kg, 1344 kg and 140kg, respectively. To obtain a good workability, 0.5-0.7% of a water reducing admixture was added. The admixture was a sodium salt of sulfonated naphthalene-formaldehyde condensate with about 25% of sodium sulfate. The ties were cured in about 80℃ steam for 10 hours.

EVALUATION OF ALKALI-REACTIVITY OF AGGREGATES

Coarse aggregates were collected from the cracked railway ties and the quarry. They were submitted to X-ray diffraction analysis, petrographic examination and evaluation of alkali-reactivity. The calcitic dolostone contained 70-80% rhombhedral dolomite in 2-20 μm, 5-10% calcite and 15-20% acid insoluble residue consisted of <10-60 μm quartz, kaolinite and illite etc. Dolomite crystals in some of the dolostone had been attacked. Fig. 1 shows the textures. The dolomitic limestone was composed of 10-20% 10-30 μm rhombhedral dolomite, 75-85% calcite and 2-5% impurities such as <10-50 μm quartz and microcline. Its microstructure is also shown in Fig. 1. Mineral in the metamorphic quartzite was strained quartz.

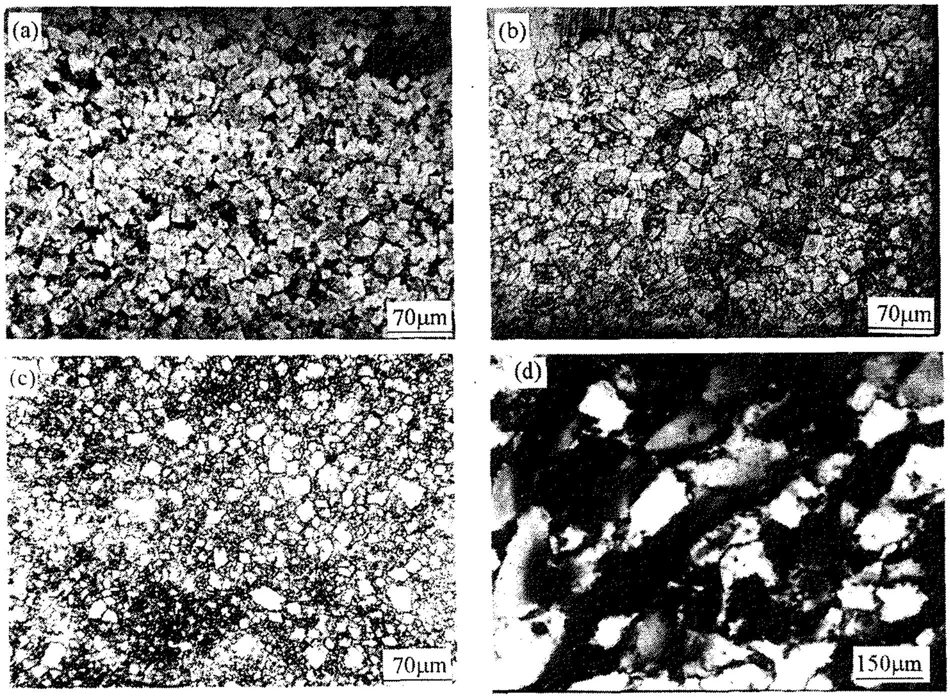

Fig. 1 The microstructures of the aggregates used. (a) Calcitic dolostone; (b) Calcitic dolostone with attacked dolomite; (c) Dolomitic limestone; (d) Metamorphic quartzite

Alkali-silica reactivity was tested with the Chinese autoclave method (CECS 48-93). Table 1 lists the results. The expansions caused by the calcitic dolostone (PCD) and the dolomitic limestone (PDL) are not in excess of the 0.1% limit. Thus, they seem nonreactive with respect to ASR. Some of quartz crystals in sample PCD were fine-grained and could react with alkali solutions to form gel (14% of quartz was reacted when powder of PCD was soaked in a large amount of 1 mol/L KOH solution for 60 days). This reactivity may be responsible for the relatively large expansion. The metamorphic quartzite (PMQ) gives rise to a large expansion, thus is reactive.

Table 1 Expansion of micromortar bars for evaluating alkali-silica reactivity

Ratio of Cement to Aggregate	Expansion (%)		
	PCD	PDL	PMQ
10	0.053	0.038	0.097
5	0.091	0.056	0.160
2	0.090	0.038	0.234

The calcitic dolostone and dolomitic limestone were also subjected to rock prism and microconcrete bar tests to determine their alkali-dolomite reactivity. The rock prisms in 8mm × 8mm × 25-32mm were precured in 80℃ water for 2 days. After the prisms were cooled, their original lengths were measured. Then, they were immersed in 1 mol/L KOH solutions at 80℃ for 91 days and at room temperatures for another 570 days. The expansion of the prisms is demonstrated in Fig. 2. The calcitic dolostone (PCD) was very expansive. The dolomitic limestone appeared less expansive and the maximum expansion is 0.119%.

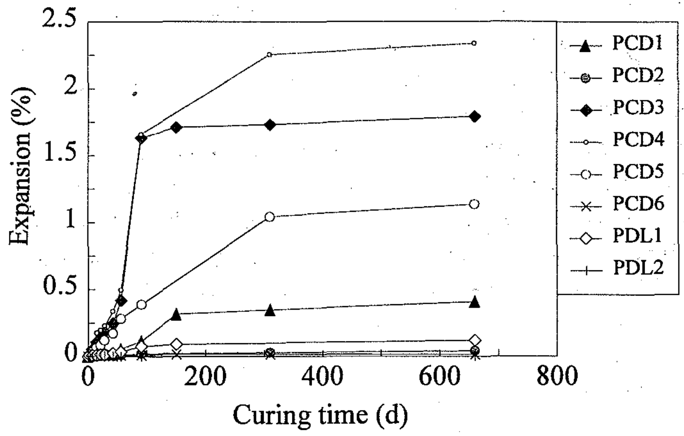

Fig. 2 Expansion of calcitic dolostone (PCD) and dolomitic limestone (PDL) prisms cured in 1 mol/L KOH solutions at 80℃ for first 91 days and at room temperatures for another 570 days

Fig. 3 shows the expansion of microconcrete bars (Tang et al. 1994) caused by the dolomitic limestone (PDL) and carbonate aggregates (PDM) derived from the ties. The alkali content of Portland cement used was boosted from 0.66% to 1.50% Na_2Oequiv. Aggregate was in 5-10mm. Cement/aggregate was 1. The demoulded specimens in 20mm × 20mm × 60mm were cured in 100 ℃ steam for 4 hours and then were autoclaved in 150 ℃ 10% KOH solutions. The expansions of PDL and PDM are

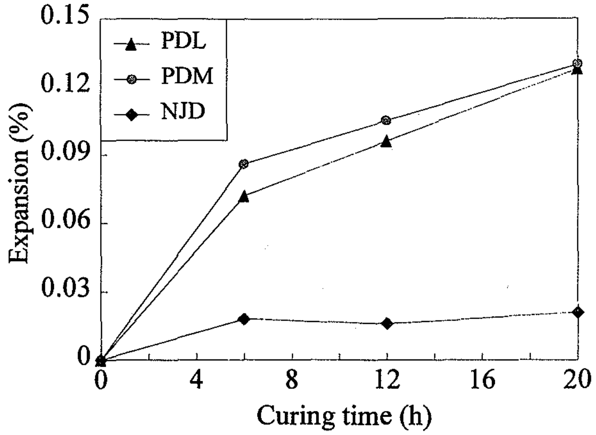

Fig. 3 Expansion of microconcrete bars with dolomitic limestone (PDL), carbonate aggregates (PDM) and nonreactive dolostone (NJD)

significantly larger than that of nonreactive Nanjing dolostone (NJD). Microscopic check reveals that the bars were cracked due to the expanding of the two aggregates. Therefore, the carbonate rocks are recognized as alkali-dolomite reactive.

PETROGRAPHIC EXAMINATION OF THE CRACKED CONCRETE

The 10cm × 10cm × 10cm concrete specimens placed outside the laboratory room of the plant cracked at age of 4-6 years. When cured in a moist container, alkali-silica gel was observed to squeeze out through the cracks. This indicates that the concrete was affected by alkali-silica reaction.

Observation on the polished and thin sections of cracked railway ties reveals that the quartzite has been eroded and cracks mainly originated from the coarse aggregates. Some cracks spread to mortars along the corners or boundaries of quartzite and carbonate rocks. A few of cracks extended from the dolomitic zones of the aggregates to mortars. Others were perpendicular to the boundaries of coarse aggregates. Fig. 4 illustrates the cracks in the concrete ties.

The fine aggregates were almost feldspar and crystalline quartz. A very small amount of particles composed of strained quartz and microcrystalline quartz were incorporated in sands. Some of these particles also cracked the mortars.

Ettringite phase was not observed in cracks, voids and interfaces between aggregates and hardened cement pastes.

DISCUSSION

The alkali content of the concrete ties from cement and water reducing admixture was 6.4 kg/m^3. This value is twice as large as the safe threshold 3.0 kg/m^3 for alkali-silica reaction (Hobbs.1988) and far larger than the safe alkali content for alkali-dolomite reaction(Rogers and Hooton.1992). The railway ties placed out of doors were easy to gain moisture. Thus, the reactive siliceous and dolomitic aggregates may make

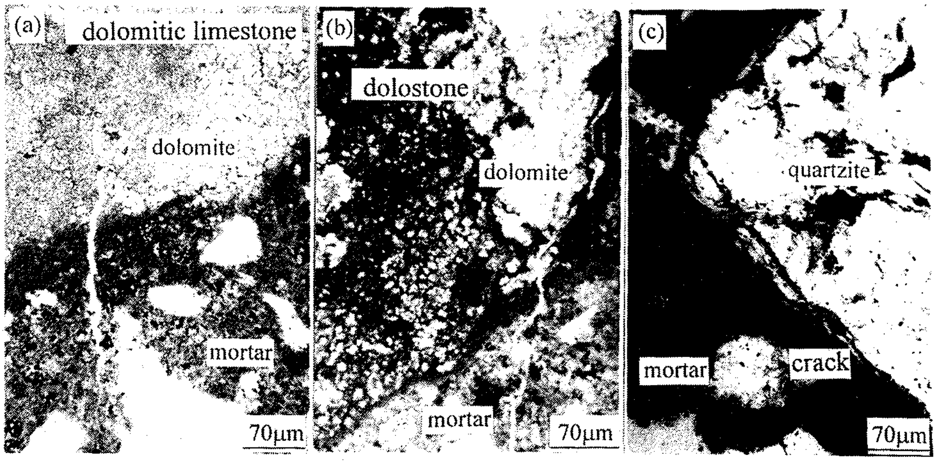

Fig. 4 The cracks in the deteriorated railway ties

the ties to suffer distress by alkali-silica reaction and alkali-dolomite reaction. The ties cracked by alkali-aggregate reactions may be further damaged by the motive load, and by freezing and thawing when laid in Northeast and North China. Eventually, the railway ties may be collapsed, as found in Herongjiang and Shanxi provinces.

The railway ties were precast units and were treated with heat steam during production. It was suggested that these ties might be deteriorated by secondary ettringite formation (Heinz and Ludwig. 1989). The molar ratio of SO_3 to Al_2O_3 in the cement was 0.66-0.69. The ties were almost placed in a climate with less than 95% relative humidity. W/C was 0.30. According to Heinz and Ludwig (1989), the railway ties in question might not be damaged by secondary ettringite. Petrographic examination shows that there was not a large amount of ettringite in the concrete. Therefore, the secondary ettringite formation was not, at least, the main reason of the deterioration. The cracking of concrete specimens suffered no heat treatment further confirms this conclusion.

CONCLUSION

The calcitic dolostone and dolomitic limestone aggregates in the railway ties were potentially alkali-dolomite reactive and the metamorphic quartzite included was alkali-silica reactive. Petrographic examination of the deteriorated concrete revealed the presence of typical features of alkali-dolomite reaction and alkali-silica reaction. It appears that alkali-aggregate reactions are a prerequisite for the deterioration of the concrete.

Acknowledgment

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ALKALI-AGGREGATE REACTION EXPERIMENTS IN ISRAEL

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ABSTRACT

Four concrete aggregate samples were tested: **(1)**A Mediterranean Sea quartz-sand; **(2)** A calcite dolomite from "Shefar-Amm", a large Israeli rock-quarry; **(3)** A limestone containing ~ 4% SiO_2 , from a north Israeli rock quarry; **(4)** A complex mixture of stones, from a south Israeli river basin quarry , containing various proportions and types of amorphous SiO_2 and carbonates. Mortar or concrete specimens, containing per cubic meter 1.5 -7.5 kg Na_2O equivalent, have been prepared and tested:

The samples have been tested for AAR using five known standard methods : **(I)**The 24 hour chemical method; **(II)**The two week "NBRI" test (immersion of mortar bars in 1N NaOH solution at 80°C, with periodical length change measurements); **(III)**The long term (2 years) mortar bar expansion method, at 38°C in 100% relative humidity; **(IV)** The same as in "**(III)**" but on concrete prisms; (e) The same as in "**(IV)**" but at 22°C. A major difficulty was finding a technique to allow an equal distribution of the air humidity to cover all the sides of the specimens.

The following results have been obtained: **(1,2)** The quartz-sand and the calcite dolomite samples are not reactive; **(3)** The silica including limestone might be dangerous to use . Its expansion kinetics on concrete prisms, suggests a new case of " strong late expanding alkali-silicate/silica reaction"; **(4)** The mixture of silica & carbonate river stones might be dangerous to use, being suspected of a strong alkali - silica reactivity. A discussion is made on the AAR development conditions specific in Israel and on the significance of the standard AAR laboratory methods.

Keywords:, Aggregate, Alkali-reaction, Carbonate, Concrete, Durability , Israel , Portland cement, Silica , Silicate, Standard requirements, Test methods.

INTRODUCTION

The Israeli standard specifications for mineral aggregates (IS-3) for Portland-cement (IS-1), the Concrete Code (IS-466) or other standards do not dwell on the AAR problems, the possibility being of deleterious alkali-reactive aggregates or do not limit a maximum amount of alkalis in the cement or in the concrete. The cement users are generally unaware of AAR and its possible effects on the durability of the concrete structures, as was the case some 10 years ago in Belgium [Van Gemerth.1989], in the United Kingdom [Hoobs.1989], in Japan [Okada.1989] and in other countries where awareness on AAR arose. The authors of the present paper contend that AAR research is important in Israel since various concrete aggregates may be suspected of different types of potential reactivity (alkali-silica, alkali-carbonate etc.). The present paper reports the results obtained after some four years of AAR research in Israel..

DESCRIPTION OF TESTED AGGREGATES

(1)A non alkali-reactive quartz-sand extracted from the Mediterranean seaside near the city of Ashdod, containing ~94% SiO_2. The sea sand is frequently used in the manufacture of concrete as a part of the fine aggregate to improve the workability of the fresh concrete made with crushed aggregate. In the manufacture of laboratory mortar bar and concrete prisms, the 0.15-0.60 mm sea sand was graded together with the 0.60-4.75 mm aggregate 2,3or4 bellow, according to ASTM C227.

(2) The Shefar-Amm aggregate is a calcitic dolomite (ASTM C294). Chemical analysis [Harel.1990] gave the following stoichiometry: 87.3% $CaMg[CO_3]_2$ (dolomite); 12.1% $CaCO_3$ (limestone); 0.30% SiO_2; 0.06% Na_2O+K_2O; 0.19% $Al_2O_3+Fe_2O_3$. Alkali-carbonate reaction is theoretically possible.

(3) A limestone aggregate sample containing ~4% SiO_2 ,that turned out to be particularly interesting for alkali reactivity research. This sample, supplied from a quarry situated in the north of Israel, was known to be a "pure limestone" and was considered to be a comparative blank (non alkali-reactive) sample. *The chemical analysis corroborates the assumption of non-reactivity:* The sample is not suspected of alkali-silica reactivity since it contains less than 5% insoluble residue in HCl [Fournier & Berube.1993] and is not susceptible to alkali-carbonate reactivity according to a new CSA test, consisting of plotting on a graph, the CaO/MgO ratio as function of the Al_2O_3 content: for a CaO/MgO ratio ≥ 24 and for a content of $\leq$ 0.37% Al_2O_3 the sample is placed on the non ACR zone of the graph (CSA A23.2-26A-94) The limestone sample was studied by several AAR methods which led to inconclusive results. *The non-reactivity assumption was finally refuted* when our investigations comprised simultaneously: (a) high alkali content concrete specimens, stored in 38°C wet air; (b) measurements at late ages (11 months and more); (c) testing by the AS/SR version of the concrete prism expansion method. *For the above conditions, concrete containing this aggregate, does show a strong late-expanding alkali-silicate/silica reaction.*

Note: The former edition of the CAN/CSA-A.23.1-M90&CAN/CSA-A.23.2-M90 including the AAR regulations (as A23.1-90-Appendix B or A23.2-14A) were replaced in June 1994 when our concrete specimens were 18 month old. The AS/SR designation no longer exists in the new edition.

(4) Aggregates from a South Israeli (Negev) river basin quarry, representing *a complex mixture of stones* containing various proportions and types of amorphous SiO_2 and carbonates (amorphous silica minerals colored from bright brown to white, hard limestone - bright colored, hard dolomite - brown colored) or *fragments of the above described stones* (silica stone with chalk and limestone inclusions, chalk and limestone with or without silica inclusions, and calcitic dolomite). A visual examination of the laboratory sample, shows that a third of the coarse aggregates (grain size 14-19 mm) granules are silica minerals or carbonates including silica minerals. The sample is strongly suspected to be potentially alkali reactive, especially for ASR.

EXPERIMENTAL PROCEDURES

The 24 hour ASTM C289 chemical test with an altered graph (Fig.1)The test consisting of the immersion of graded aggregate sample in 1N NaOH solution for 24 hours at 80°C, is followed by chemical analysis of the filtered solution for dissolved silica (Sc) and for reduction of alkalinity (Rc). The results are plotted on a graph showing four regions of aggregates: **(A)** "innocuous"; **(B)**"generally non-reactive", but in this field the interference of carbonates in the reaction between SiO_2 and NaOH leading to inconclusive results requiring long term mortar bar or concrete prism expansion methods; **(C)** "highly reactive, potentially deleterious", but if the pessimus (worst) percentage of reactive silica is exceeded, the mortar bars might display non significant expansions; **(D)** "deleterious reactive". The original ASTM C-289 graph comprises only three regions, the **(A+B)** region being considered innocuous.The ASTM chemical test was excluded from the last CAN/CSA. We think that it should be considered as a more suitable alternative method for future AAR test of siliceous limestone aggregates. The alternative method consists of performing the ASTM procedure on the insoluble residue obtained by dissolution of the carbonates in concentrated hydrochloric acid [Dron.1992, Berube & Fournier.1993]. In order to find the worst proportion of reactive particles the chemical method has been altered in the French standard NF-P18-589.

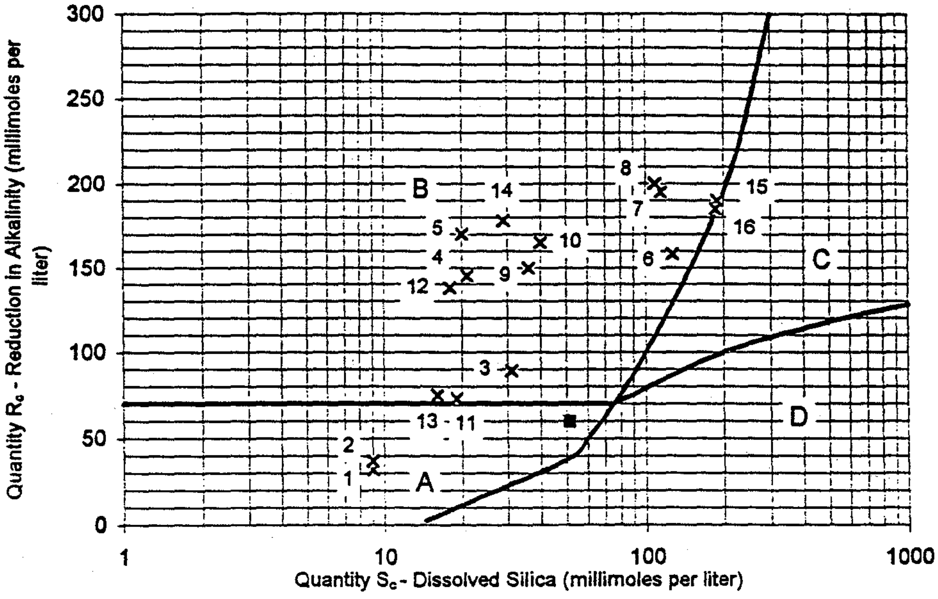

Fig.1 Evaluation of the AAR results by the 24 hour chemical method ASTM C289 with an altered graph. Sc is plotted on a logarithmic scale[Bredsdorf et al 1960, Romanian Standard STAS 5440]: **x1...x5** *- Limestone aggregate samples,* **x6...x16** *-Aggregates from a river basin quarry* **x6...x8**- *20% silica stones + 40% carbonates + 20% sea sand,* **x9** & **x10** - *6% silica stones +54% carbonates+20% sea sand,* **x11** & **x12** - *60% carbonates+40% sea sand,* **x13** & **x14** - *100% carbonates,* **x15** & **x16** - *100% silica stones* ■ - *Sea sand sample from Ashdod*

(II) The 2 week NBRI mortar bar test (ASTM C1260 or CSA A23.2-25A). Mortar bars, 25 by 25 by 285-mm ASTM C227, are maintained at 80°C, one day in water up to the zero reading and afterward in 1N NaOH solution. Fournier&Berube (1993) and CAN/CSA A23.1-94.Appendix B suggested the following mortar bar expansion limits in 14 days: <0.10% for innocuous aggregates and ≥ 0.15% for deleterious aggregates. Shayan et al (1988) suggested that: (i) mortar bar expansion of 0.10% in 10 days and 0.10% in 22 days to be taken as indicating reactive aggregate of short induction period and long induction period, respectively; (i,i) for some slowly reacting carbonate rocks a concrete prism test is more suitable than mortar bars for assessing alkali reactivity.

(III)The long term mortar bar test. (ASTM C227 and CAN/CSA A23.1-90. Appendix B). The reactivity limits are 0.05% at 3 months or 0.1% at 6 months or more, for mortar bars with high alkali content (1.25% Na_2O equivalent relative to the cement weight or 7.5 kg Na_2O equivalent per $1m^3$ of mortar) maintained at 38°C, in wet air According to [Hooton & Rogers.1989] the absorbent material used in the ASTM containers might cause diminished expansions due to the alkali- leaching of the mortar bars. The test is not included in the 1994 edition of CSA Standard.. For our future work we consider an alternative method which would replace the mortar bars 25 by 25 by 285 mm with mortar prisms 40 by 40 by 160 mm according to the European RILEM experience [Grattan-Bellew.1989] and to the Japanese standard JIS A 5308..

(IV)The long term concrete prism expansion methods. Two versions (ACR and AS/SR) of the former CSA A23.2-14A, and the French Standard NF P18-857 were applied. Each former version of the CSA was intended to test concrete aggregate suspected to suffer from alkali carbonate reaction (ACR) or Alkali Silicate/Silica Reaction (AS/SR). The main difference between the two former versions is the storage temperature of the specimens in moist air, being 23°C for ACR and 38°C for AS/SR. The former CSA was replaced in June 1994 with the new CSA A23.2-14A-M94. The main modifications are: (i) the cement content increased from 310 to 420 kg/m^3 and an alkali content of 1.25 %Na_2O equivalent (eq.) by mass of cement, resulting in an increase of the alkali content in concrete from 3.90 to 5.25 kg. Na_2O eq./m^3 ; (i,i) a 38°C storage temperature in moist air is required for all the specimens, independent of the AAR type. The French standard looks like the new CSA, with a cement content of 410 kg/m^3, corresponding to 5.13 kg Na_2O eq./m^3 of concrete. *ACR expansions are considered deleterious*: depending on the micro-climate to which the structure is to be exposed, if the length change exceeds 0.010% after 3 months or 0.025-0.040% after 12 months (former CSA); independent of the micro-climate, if the length change exceeds 0.015% after 3 months or 0.025% after 6 months or 0.030% after 12 months (ASTM C1105). *AS/SR expansions were considered deleterious by the former Canadian standard*, depending on the micro-climate to which the structure is to be exposed, if the length change exceeds 0.040-0.075% after 12 months. *The new Canadian Standard considers that expansions are deleterious*, independent of the AAR type, if the length change exceeds 0.040% after 12 months , noting that "in critical structures such as those used for nuclear containment or large dams, a lower expansion limit may be required" (CAN/CSA A23.1-94. Appendix B). *The French test conditions* [NF-P18-589] are more strict than the Canadian. To the best of our knowledge there are no French standard limits for critical AAR expansions, a 0.040% limit at 8 months seems to be accepted by scientists to indicate reactivity but it is still being discussed [Corneille & Bollotte.1994]. However it seems that the interpretation of the French test results is covered by the Canadian standard limits.

In the future *significant AAR compositions for mortar bars and concrete prisms methods* are to be prepared taking into account the experimentally found worst pessimum") content of the reactive particles (NF-P18-589 and Corneille & Bollotte.1994)

AAR STUDIES ON MEDITERRANEAN SEA QUARTZ- SAND

The samples of Mediterranean Sea quartz-sand are placed in region A, "innocuous aggregates" of the ASR reactivity graph (Fig. 1)

AAR STUDIES ON SHEFAR-AMM CALCITIC DOLOMITE

All the four AAR test methods gave results that did not indicate alkali reactivity for the investigated concrete aggregate sample.

AAR STUDIES ON SILICEOUS LIMESTONE AGGREGATE

The methods I, II and III gave inconclusive results or gave results that did not indicate alkali-reactivity for this aggregate. Only the fourth method (IV) showed under certain conditions deleterious alkali- aggregate reactivity.

(I) The 24 hours chemical test ASTM C289 On the graph (Fig.1) five tested samples are placed as follows: two samples in region A, three others on the doubtful region B, due probably to the presence of carbonates interfering with the reaction between NaOH and the dissolved silica. In order to clarify if the tested aggregate is reactive, long term tests (III) or (IV) are required by ASTM C33.

(II) The 2 week NBRI mortar bar test After 14 days of storage at 80°C in 1N NaOH solution, an average expansion of 0.08% was recorded showing no alkali reactivity for the tested concrete aggregate.

(III) The long term mortar bar test ASTM C227 Even for mortars with high alkali content ,obtained by adding NaOH in the mixing water, the length changes were small (up to 0.013 % in 2 years), less than the reactivity limits

(IV) The long term concrete prism expansion methods

Concrete mixtures, experimental variants and results. These studies comprise four concrete mixtures and six experimental variants (Table 1). Each experimental variant was labeled with the name of the concrete mixture (M1, M2, M3 or M4) and the temperature (in °C) of the wet air environment in which the specimens were stored. Storage. temperatures were 22°C or 38°C. The experimental variant M4/38 is in accordance with the requirements of the new Canadian and French specifications

According to the specifications of the ACR version of the former Canadian standard, the small length changes recorded for specimens M4/20, prove that the tested aggregate is not alkali carbonate reactive, resulting logically from the chemical analysis according to the new CSA requirements(CSA A23.2-26A-94). Fig. 2 shows the length changes vs. time up to 2 years for the six experimental variants. The length changes for each specimen of the experimental variants M1/22, M1/38, M2/38, M3/22 and M4/22 are small, sometimes within the error of measurement, and they are placed on the hatched surface. Trying to distinguish, at different ages, the non-reactive, the marginally or the very reactive aggregates, the expansion kinetics of each of the three specimens of the experimental variant M4/38 is compared with Canadian characteristic AAR expansion curves comprised in the former CSA. This comparison shows that the tested aggregate generates a strong late-expanding alkali-silicate/silica reaction. It is also obvious that in comparison with the Canadian limits the beginning of the expansions appears much later and in a sudden manner at ~11 month, with subsequent expansions developing abruptly to very high values.

Table 1 Concrete mixtures and experimental variants for AAR testing

Name of the concrete mixture	M1	M2	M3	M4
Water- cement ratio (w/c)	0.63	0.62	054	0.52
Portland Cement content in concrete (kg/m^3)	310	310	410	410
Additional alkalis through the mixing water	No	Yes	No	Yes
Alkali content (as Na_2O equivalent) provided from the cement and from the mixing water				
-Relative to the cement weight (%)	0.47	1.25	0.47	1.25
-Relative to the concrete (kg/m^3)	1.46	3.90	1.93	5.13
Slump of the fresh concrete (mm)	0	0	~80	~80
Density (kg/m^3)	2360	2350	2350	2370
Compressive strength (MPa): -7 days	31	21	38	23
-28 days	42	34	55	34
-180 days			66	59
Average expansion at 2 years (%):				
-Storage in 100% humid air at 22±2°C	-0.005	-	-0.008	0.002
-Storage in 100% humid air at 38°C	0.007	0.011	-	0.188
Negative (-)values represent shrinkage changes				

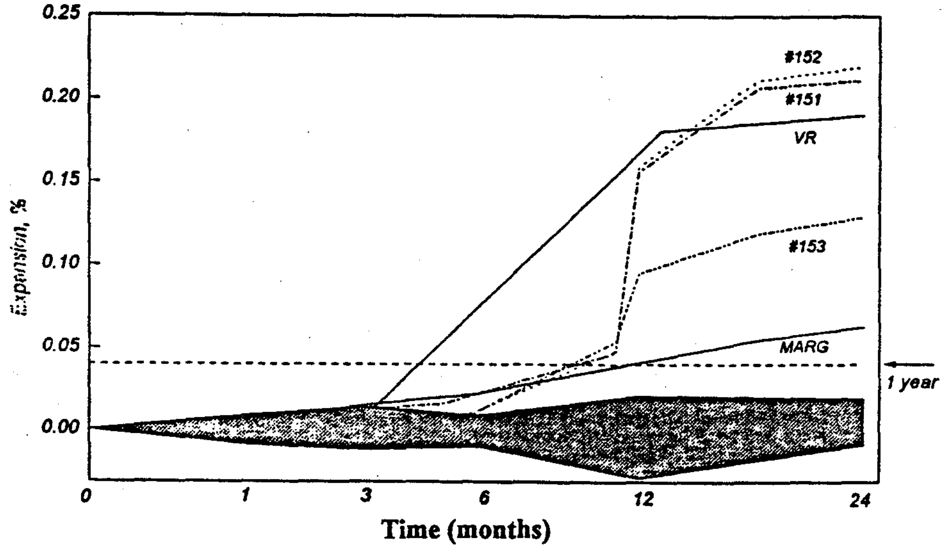

Fig 2 Expansion kinetics for the experimental variants studied, compared with the Canadian "characteristic expansion curves for concrete prisms made with the greywacke-slow/late-expanding alkali-silicate/silica reactive aggregates" (time is plotted on a square cubic root, the graph is adopted from page 100 of the former CSA A23.1-90): ***MARG*** *- Expansion behaviour of marginal reactive AS/SR aggregate;* ***VR*** *- Expansion behaviour of very reactive AS/SR aggregate* ***#151,#152,#153*** *Expansion kinetics for each of the three M4/38 specimens;* ***The hatched surface*** *- Expansion kinetics for M1/22, M1/38, M2/38, M3/22 and M4/22).*

AAR STUDIES ON A SOUTH ISRAELI (NEGEV) RIVER BASIN QUARRY

(I) The 24 hours chemical test ASTM C289. Eleven mixtures were tested: The results are plotted on the Sc/Rc graph (samples 6 to 16, Fig. 1). All the 11 samples are placed in zones B and C, with a recommendation to study that reactivity by accelerated and long term expansion methods.

(II) The 2 week NBRI mortar bar test. An average thermal expansion of 0.066% was recorded for mortar bars after 24 hours of storage at 80°C in water. From the "zero measurement", the average expansions recorded for the bars stored at 80°C in 1N NaOH solution were: 0.078% after 7 days; 0.112% after 10 days; 0.120% after 11 days; 0.143% after 14 days. The expansion values after 14 days exceeds 0.150% for three of the eight bars studied at the same time. The expansions are at the limit of the deleterious values according to CAN/CSA A 23.1-94 and according to the limits suggested by Shayan et al (1988).

(III,IV) Long term expansion methods.

Mortar bar test ASTM C227 For mortars with high alkali content ,obtained by adding NaOH to the mixing water (up to 1.25% relative to the cement mass), the recorded average expansions were : 0.029% after 6 month, 0.038% after 12 month, 0.035% after 24 month. The recorded expansions are under the limits of 0.1% at 6 month..

Concrete prism expansion methods Expansions measurements were performed on prisms prepared with 410 kg. cement per cubic meter of concrete. The alkali content was increased up to a Na_2O equivalent of 1.25% relative to the cement mass by the addition of NaOH to the mixing water. For storage in a humid air at 22°C the length changes are small, within the error of measurement. For the concrete prisms stored in humid air at 38°C, the following average expansions were recorded: 0.025% after 6 months, followed by a stabilized expansion of 0.034-0.042% up to 30 month. According to CAN/CSA, the expansions are at the limit of deleterious expansion. *The very heterogeneous composition of the quarry* does not allow us to reach definite conclusions regarding the reactivity of all the aggregates in the quarry.

DISCUSSION

One can summarize the first AAR results obtained on Israeli aggregates as follows:

(1) Four aggregates were tested, two of them might be dangerous to use: a limestone including ~4% silica and a mixture of silica & carbonate river stones. Consequently, it is highly recommended to continue and to develop the AAR studies in Israel.

(2) The long term expansion kinetics of concrete prisms with the limestone aggregate suggests a new case of " strong late expanding alkali-silicate/silica reaction". This behaviour was not foreseeable by accelerated test methods and becomes obvious after some 11 months of measurements. Based on a detailed AAR study of Quebec rock-quarries, Fournier & Berube (1993) concluded that carbonate aggregates containing less than 5% insoluble residue or involving expansions less than 0.1% after 14 days in the NBRI method are innocuous. However, our limestone concrete aggregate sample which passed both requirements is a deleterious aggregate. Unfortunately, the field of validity of each accelerated test method depends on the chemical, physical and geological peculiarities of the tested aggregate.

(3) The mixture of silica & carbonate river stones might be dangerous to use, being suspected of a strong alkali -silica reactivity.

For future AAR research based on the results shown here we would propose the following: (i) improvements to the NBRI mortar bar expansion method [Shayan et al.1988] and the use of autoclave methods [Fournier & Berube(1993) and the French standard NF P18-590]are to be taken in account to predict the AAR behaviour of slowly reacting aggregates; (i,i) alternative methods to the 24 hours ASTM chemical method,described in the "EXPERIMENTAL PROCEDURE", are to be taken in account for the AAR test the reactivity of the carbonate aggregates.

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Testing

for

Alkali-Aggregate Reactivity

RECENT DEVELOPMENTS IN TESTING FOR ASR IN NORTH AMERICA

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ABSTRACT

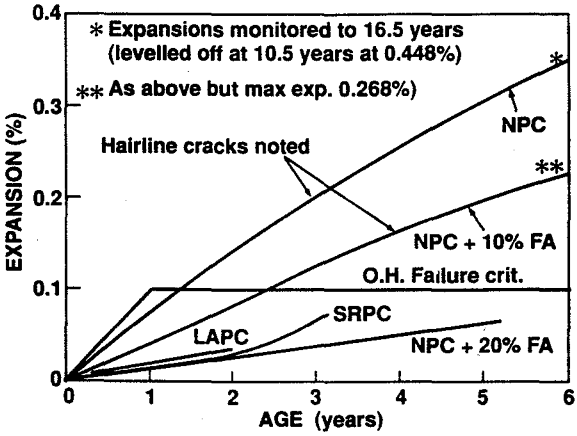
Traditional ASTM test methods have been found to be deficient for detection of several types of reactive aggregates. As a result, a new rapid mortar bar expansion test method, ASTM C1260-94 (formerly Proposal P214-89) has been standardized in the US as well as in Canada (CSA A23.2-25A-M94). This rapid test is suitable for screening aggregates but should not be used in isolation for rejection of aggregates. As a longer term referee test, the modified CSA A23.2-14A-M94 concrete prism test has been approved as ASTM C1293-95. It is thought that combined with the petrography, the appropriate tools are now in place to develop a new ASTM Guide document offering better guidance for prevention of ASR.
Keywords: alkali silica reactivity, ASTM, CSA, testing.

INTRODUCTION

Until recently, the only standard ASTM test methods for alkali silica reactivity (ASR) had been those developed in the 1940's and 1950's. These are the C227 mortar bar expansion test at 38°C, the C289 quick chemical test, and the little used C342 Conrow mortar bar expansion test. Since then, little attention appears to have been paid to their adequacy for predicting performance since they had originally appeared to be suitable for the narrow range of aggregates on which their development was based. As a result, engineers and specifiers became complacent and as new aggregate sources were developed in different parts of North America, and sometimes gave false satisfactory test results, unwittingly deleterious aggregate-cement combinations were used in some structures. One of the specific problems with C227 was the problem of alkali leaching from the mortar bars (Rogers & Hooton, 1991).

In the 1960's, Ontario Hydro (O.H.) discovered by examination of old hydraulic structures such as the Lady Evelyn Lake dam, that greywacke-argillite rock found in Ontario was deleteriously reactive but that ASTM C227 expansions and C289 quick chemical test results were satisfactory (Sturrup, *et al.* 1983; Dolar-Mantuani, 1969). Laboratory testing indicated that C227 bars initially expanded at a slow rate but continued to expand steadily for many years. While the Appendix to ASTM C33 suggests a safe expansion limit of 0.10% at 6 months, some of these aggregates did not exceed 0.10% until 12 to 24 months (Fig. 1). Later, similar experiences in eastern Canada were documented by Duncan, *et al.* (1973) in Nova Scotia. Surprisingly though, there has been no action to tighten limits for C227 expansion.

This led the Canadian CSA A23 standards subcommittee on AAR to consider use of a concrete prism test to evaluate ASR. This evolved from its original development by Swenson & Gillott (1964) for detection of alkali-carbonate reactive aggregates and has undergone numerous revisions to make it suitable for reliably detecting alkali-silica reactive aggregates and the latest version, CSA A23.2-14A-M94, is the one that has been recently adopted as C1293-95 by ASTM. Unfortunately expansions must be monitored for up to 12 months.



Mat'ls	K_2O	Na_2O	Na_2O Equiv.
NPC	0.85	0.37	0.93
LAPC	0.37	0.11	0.35
SRPC	0.58	0.01	0.38
Fly Ash	Detroit Edison, LOI = 4.16%		

Fig. 1. ASTM C227 mortar bar expansions-greywacke with argillites crushed rock-Lower Notch GS - Effects of cement and fly ash.

In addition, there was interest in development or adoption of a more rapid, reliable screening test for alkali silica reactivity. Numerous methods were tried and one which attracted a lot of interest was the rapid 14-day mortar bar expansion method published by Oberholster and Davies (1986). (This method was developed at the National Building Research Institute (NBRI) in South Africa and has been referred to since as the South African or NBRI test).

DEVELOPMENT OF ASTM C1260/CSA A23.2-25A

From an initial investigation of several of the over 40 published accelerated test methods, Hooton and Rogers (1989) found that the NBRI test was the only one that successfully separated a range of known deleteriously reactive from non-expansive aggregates in Ontario. This method was also used by others (Grattan-Bellew 1989)(Fournier & Berube, 1991)(Durand *et al.* 1991)(Hooton, 1991).

Standard ASTM C227 mortar bars are cast (with the exception that a fixed W/C is used rather than adjusting for constant flow as is done in C227. Note that in C1260, W/C = 0.47 is used for all tests, whereas the CSA test uses W/C = 0.50 for crushed coarse aggregate tests). After the molds are stripped at one day, the bars are placed in water which is then heated up to 80°C. On the second day, the initial length readings are made and the bars are transferred to a 1N NaOH solution at 80°C where they are stored. Expansions are monitored for a period of only 14 days in the sodium hydroxide solution at which point the expansions obtained are comparable or higher than those obtained after one year in the ASTM C 227 test (38°C at 100% humidity) (Oberholster & Davies, 1986)(Hooton & Rogers, 1989).

After initial success, the author drafted a standard test method in 1988 for the ASTM subcommittee on Chemical Reactions in Concrete. After several modifications, and with input from many others, this method was published as a ASTM Proposal standard P 214 in 1989. Proposal standards are not adopted, but also can be published for 2 years while the technical committee is finalizing its development. C1260 was adopted in 1994. Simultaneously in Canada, a similar draft method was developed for CSA which was adopted as CSA A23.2-25C in 1994.

This very accelerated test has become widely used but there are several points that need to be mentioned with respect to ASTM C1260 (CSA A23.2-25A).

Effect of Cements

Because the mortar bars are immersed in a 1N NaOH solution at an age of 2 days, the alkalies rapidly penetrate to the aggregate and there is a relatively unlimited supply and OH^- available to promote reaction. This, combined with the 80°C storage temperature means that reaction and expansion will occur very quickly. The 1N solution of NaOH is also higher than the alkalinity of the pore solutions normally found in a high alkali, high cement content concrete. Therefore, the alkali content of the cement used in this test has little effect on expansion (Oberholster & Davies, 1986, Hooton & Rogers, 1992). This means that C1260 is only suitable for evaluation of potential reactivity of aggregates and not cement-aggregate combinations.

In addition, the periclase (MgO) content of the portland cement can influence expansions. At 23°C or even 38°C, periclase only slowly hydrates to brucite ($Mg(OH)_2$) and has not been a concern. However, at 80°C this expansive hydration reaction is greatly accelerated and can influence 14 day expansion results. This was first noted by Delaware DOT, and more extensive unpublished work by Iowa DOT and by the author have confirmed this effect. As a result, the C1260 test requires the use of a portland cement with an ASTM C151 autoclave expansion (3 hours in 216°C steam) of less than 0.2%. This is much more restrictive than the 0.8% autoclave expansion limit required for ASTM C150 portland cements or the 1.0% limit for CSA A5 portland cements.

Use for Evaluation of Supplementary Cementing Materials (SCM)

Oberholster and Davies (Oberholster & Davies, 1987) had published a paper showing that while the NBRI test was not useful for evaluation of the benefits of low-alkali portland cement, it appeared to give reasonable results for appropriate replacement levels of SCM. Similar results were obtained by others (Plaxton 1987), (Berube & Duchesne, 1992). However, the ASTM Committee thought that the understanding was lacking of why the test appeared to work for SCM in combination with deleteriously reactive aggregates. After the short two day curing period prior to immersion in the hot NaOH solution, it was thought unlikely that the SCM have hydrated sufficiently to become so impermeable as to prevent NaOH from penetrating the small cross section of the mortar bars. However, it has been suggested that with silica fume (Fournier & Berube, 1991), large reductions of the alkalies in the pore solution may result from increased incorporation of alkalies in CSH at lower Ca/Si ratios. The reduction of calcium hydroxide contents in the paste fraction and the reduction in porosity of the paste-aggregate interfacial zones may also play a role.

As a result of these uncertainties, neither the ASTM C1260 nor the CSA A23.2-25A test methods currently suggest their use for the evaluation of SCM.

Interpretation of Expansions

In the author's research on Ontario Aggregates, it was found that all aggregates with known deleterious ASR field performance expanded more than 0.10% after 14 days in NaOH solution (16 days after casting the bars) (Hooton, 1991). This is similar to the results reported by Oberholster and Davies (1986) and others (Grattan-Bellew, 1989, Fournier & Berube, 1991, Stark *et al.* 1993). However, it was found that in South-Western Ontario gravel deposits containing leached chert particles resulted in 14 day expansions of 0.17 to 0.18%. Many of these aggregates had a long satisfactory history of field performance in concrete with respect to ASR (the leached chert contents are restricted since they do absorb water and result in pop-outs due to freezing and thawing). The range of expansions of these so-called "marginal aggregates" is shown in Fig. 2. Unlike the reactive aggregates, the expansions of these cherty sands and gravels slowed down before 28 days and the author suggested a secondary 28 day expansion limit of 0.33%, if an expansion of 0.15% at 14 days was exceeded (Hooton 1991).

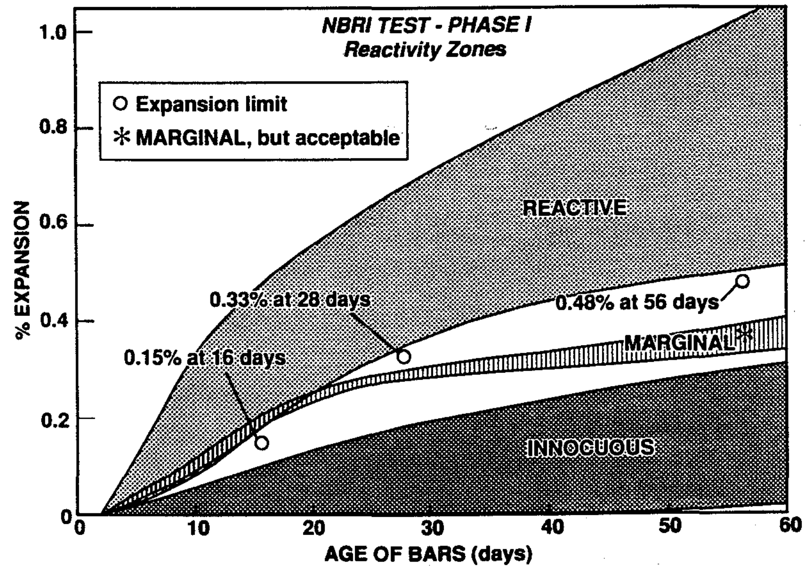


Fig. 2. Range of NBRI mortar bar expansions for aggregates with known field performance (Hooton, 1991).

As a result, in the 1994 Canadian Standard A23.1 Appendix B, a 14 day expansion limit of 0.15% has been adopted but a footnote mentions the possibility of also using 0.33% at 28 days. Other footnotes to the stated limit mention that for some siliceous limestones, a lower limit of 0.10% can be exceeded even when concrete prism expansions are less than the 0.04% limit. This latter footnote, combined with other documentation in the CSA standard, make the point that the longer term, concrete prism expansions should take precedence over the accelerated mortar bar expansions.

This may seem very complicated but it is important to recognize that a single expansion limit may not be suitable for all types of aggregates. Also, if a test limit is set too low in order to "catch" every potentially of deleterious aggregate, then numerous aggregates of good field performance and alarms will be raised for numerous likely satisfactory new sources of aggregate.

Also, the C1260 test is very accelerated and should only be used, together with petrography, as a rough screening test and not as a basis for rejection of aggregate. This lack of understanding has become a headache to aggregate producers in some regions of the U.S. where agencies have adopted the C1260 test and a 0.10% 14 day expansion limit for rejection of aggregates. Because there has been until 1995 no ASTM concrete prism test for ASR, and no ASTM guidance document exists (other than the terse Appendix to C33), the aggregate producers have had no recourse.

Possibilities for Modification of C1260

Stark (1993) has suggested that if the strength of the NaOH storage solution is reduced to match that of the pore solution expected for a particular portland cement (eg: low-alkali cement), then the C1260 test could be modified to evaluate cement aggregate combinations. It is suggested that for a water to cement ratio of 0.50 (close to the 0.47 used in C1260), the appropriate solution concentrations would be as follows:

Equivalent Alkali Content of Portland Cement, %	Storage Solution Strength (Normality)
0.6%	0.49 N
0.8%	0.62 N
1.0%	0.76 N
1.2%	0.90 N
1.4%	1.03 N

If this sort of test was adopted and appropriate expansion limits developed then it could possibly be used in lieu of the longer term, and often unreliable C227 test.

CONCRETE PRISM EXPANSION TEST

As mentioned previously, the current Canadian concrete prism test (CSA A23.2-14A-M94) evolved from one that was originally designed to detect alkali carbonate reactive rocks. This original test is the one on which ASTM C1105 is based. Several researchers in the 1960's and 1970's found that concrete prisms stored moist at either 23°C or 38°C were also effective in detecting some alkali silica reactive aggregates. Concrete testing was also thought to be preferable to mortar bars since coarse aggregates did not have to be crushed to sand sizes as in ASTM C227.

However, it was found that some slowly reactive aggregates such as the greywacke argillites in Northern Ontario did not expand very much unless the alkali loading of the concrete was raised to over 5 kg/m^3 (Magni *et al.* 1987).

Therefore, the 1994 version of the CSA concrete prism test has been substantially changed from the previous 1990 version. The concrete mixture is proportioned with a cement content of 420± 10 kg/m^3 using a normal portland cement with an equivalent alkali content (Na_2O + 0.658 K_2O) of 0.90 ± 0.10%. NaOH is then added to the mix water to provide the equivalent of a 1.25% alkali cement. The alkali loading of the concrete is therefore, 420x1.25% = 5.25 kg/m^3. As can be seen in Fig. 3 (Rogers 1988), this is more than sufficient to cause deleterious expansions in the reactive Spratt quarried siliceous limestone from near Ottawa, and it also causes deleterious expansions in slowly reactive Sudbury gravels containing greater than 15% greywacke-argillites (Fig. 4 (Rogers 1988)).

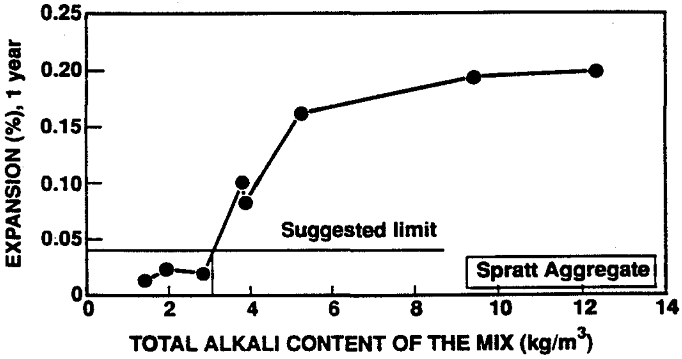


Fig. 3. Concrete prism expansions as a function of alkali loading for reactive Spratt aggregate (Rogers 1988).

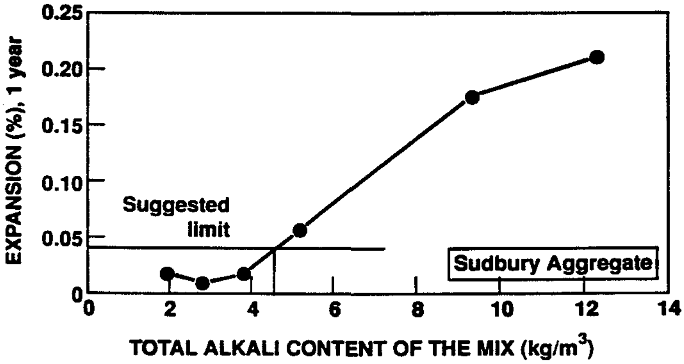


Fig. 4. Concrete prism expansions as a function of alkali loading for slowly reactive Sudbury aggregate (Rogers 1988).

The newly adopted ASTM C1293 test is identical, except for minor editorial issues, to the 1994 version of the CSA concrete prism test method.

The expansion limit used in the CSA standard is 0.04% at one year. This expansion is approximately where cracking and signs of distress are first observed on the prisms. (Mortar bars on the other hand do not exhibit visual damage until approximately 0.10% expansion). It also relates well in terms of observed field performance. France has recently adopted the CSA concrete prism test and has also used a 0.04% limit.

The 12-month time is a concern, but is necessary unless the temperature is raised or other changes are made to accelerate expansion. For a test to be used for acceptance further acceleration of the test may have undesirable side effects.

CONCLUSIONS

The lack of reliability of traditional ASR test methods such as ASTM C289 or C227 when applied to certain aggregate types has led to development of an accelerated mortar bar test (ASTM C1260-94/CSA A23.2-25A-M94) and a concrete prism test (ASTM C1293-95/CSA A23.2-14A-M94). Interpretation of results is not simple and aggregates should not be rejected, but can be accepted, based solely on C1260 expansions. If expansions in C1260 are considered potentially deleterious, then the concrete prism test is suggested to confirm the results before an aggregate is either rejected or limitations are put on its use (for example, a potentially reactive aggregate could be used only if a low alkali cement was used or if a minimum quantity of mineral admixture was used.

While the expansion limits placed on these new test methods, may not be universally acceptable, these tests appear to be far more reliable than the older test methods. In fact, the 1994 CSA standard does not recommend the use of either ASTM C227 or C289.

A guidance document on how to use these new test methods need to be developed by ASTM as was done by the CSA.

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THE ASTM C 289 METHOD AND THE KINETIC TEST IN THE STUDY OF ALKALI-SILICA REACTIVITY OF ITALIAN ALLUVIAL DEPOSITS

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ABSTRACT

The authors carried out, from 1984 to 1993, a wide program of study on the alkali-silica reactivity of Italian alluvial deposits. The study involved all the Italian peninsular territory south of river Po; over 300 samples were collected, and their reactivity was studied by means of petrographic (ASTM C 295) and chemical (ASTM C 289) methods. It appeared that, unless the almost constant presence of flint as the main reactive mineral, the chemical method was often unable to correctly distinguish among alluvia with different flint contents (and this also after the elimination of carbonates before testing). It was so decided to re-examinate the samples studied using the kinetic test, in order to verify if it was possible to obtain a better correspondence between petrographic and chemical results. The work is still in progress; here the results obtained for some alluvial deposits of the Adriatic coast, which the experience has shown to be among the most dangerous in Italy, are presented. Unfortunately, the tests carried out showed a very bad reliability among the results of the kinetic test and the potential reactivity, so that the ASTM-C 289, if carried out on the aggregates after the elimination of carbonatic components and if slightly modified in the dangerous limits, appears to be at the moment, and despite to its often uncertain repetitivity, the most reliable chemical method for Italian aggregates.

Keywords: AAR, alluvial deposits, kinetic test, Italy.

INTRODUCTION

The alkali-silica reaction started to be studied in Italy only about fifteen years ago, in connexion with the firsts serious cases of structural damages that occurred in the country, using petrographic (ASTM C 295, even modified) and chemical (ASTM C 289) techniques. In various cases, however, the ASTM C 289 method indicated as non-reactive or low-reactive (Baronio 1984) aggregates having caused structural damages, and confirmed as clearly reactive by optical analyses (Barisone 1985).

In order to better evaluate the reliability of these tests for italian aggregates, a study was carried out (Barisone & Restivo 1992) to compare the two methods cited above; the analysis of over 40 samples (corresponding to 23 alluvial deposits of 16 different rivers of the Italian adriatic coast) has shown on one side the high presence (often over the 5 %) of potentially reactive minerals (mainly flint) in the aggregates , on the other side the scarce reliability and a certain under-estimation of the reactivity by the ASTM C 289 methodology.

It was so decided to use the kinetic test (Sorrentino et al 1991) for a further analysis of the most representative samples already studied, in order to verify the suitability and reliability of this new method in evaluating the potential reactivity of Italian aggregates.

GEO-MINERALOGICAL OUTLINES

All the samples come from actual and recent alluvial deposits, formed by the rivers streaming down from the eastern side of Appennines range (Adriatic italian coast). These deposits are mainly formed by pebbles, gravels and sands originated by the erosion of the sedimentary rocks (limestones, marls, calcareous sandstones) that widely outcrop in the mountains, and whose age varies from Trias to Miocene (Crescenti et al. 1969).

The main reactive mineral found in the alluvia is flint (a mix of cripto and micro crystalline quartz, with variable amounts of chalcedony and, sometimes, opal), that comes from the erosion of chert-rich sedimentary formations, mainly limestones of the Mesozoic "complete series" and sandstones of some Miocenic stages (Langhian, Aquitanian).

SAMPLING AND ANALYSES

The sampling

The samples to be studied were collected, whenever possible, in the stock piles of crushed sand in quarries, in order to obtain a good representativity of the coarse-grained alluvial deposits without collecting huge amounts of material.

Petrographic analysis

It was chosen due to its rapidity and good reliability (Barisone 1984). Each sample, after quartering and carbonates elimination by means of diluted chloridric acid, was sieved in order to obtain eight granulometric classes; each class was examinated by a petrographic microscope, identifying a number of grains progressively increasing at the decreasing of average size dimensions (from about 200 in the 0.500-0.250 mm class to more than 500 in the 0.075-0.040 mm class).

The percentage of the various reactive minerals in each sample was calculated as the ponderate average of the eight determinations. The residue after the chemical attack and the total amount of flint (both referred to the original weight of the sample) are listed in Tab.1.

ASTM-C 289 test

As it is well known, the outlines of this method are (ASTM 1952): a 25-g sample crushed to 0.30-0.15 mm and treated at 80 °C for 24 hours with 25 ml of a 1N NaOH solution; determination on the filtrate of reduction in alkalinity (Rc) and dissolved silica (Sc). It must be noted, however, that the high carbonates content of the alluvia studied would be able to considerably alterate the results of the tests (Fournier & Berubè 1989); to avoid this fact, the samples were preliminarly submitted to a chemical attack with diluted chloridric acid, in order to eliminate the carbonates, and the ASTM C 289 test was made using a weight of residue corresponding to 25 g of the original sample.

The effects of such a treatment on the results of the ASTM C 289 test were verified by means of some untreated samples, on which the test furnished a marked under-evaluation of potential reactivity.

The results obtained by the ASTM C 289 method on treated and untreated samples are synthetized in Tab. 1.

Kinetic test

In synthesis, this methodology (Sorrentino et al. 1992) is based, such as the ASTM C 289 method, on the treatment of a crushed sample (25 g) with 25 ml of a 1N NaOH solution at 80 °C. Nevertheless, important differences are: the sample granulometry (0.30-0.00 mm, with about a 40 % smaller than 0.10 mm); the execution of analyses after 24, 48 and 72 hours of etching; the replacing of Rc with the Na_2O solution content; the choice of the SiO_2/Na_2O ratio as reference parameter.

Also in this case the tests were carried out after carbonates elimination, using a weight of residue corresponding to 25 g of the original sample. The results obtained are listed in Tab. 1.

Table 1a Petrographic examination, ASTM C 289 and kinetic tests results

Sample	Residue (%)	Petrogr. Flint (%)	ASTM-C 289 Rc (mmol/l)	ASTM-C 289 Sc (mmol/l)	Attack time (h)	Kinetic test Na_2O (mmol/l)	Kinetic test SiO_2 (mmol/l)	Kinetic test SiO_2/Na_2O
3	untr.	7.8	245	30	24			
	37.2		8	150	24	184	115	0.62
					48	195	132	0.68
					72	224	196	0.87
7	12.0	4.0	70	93	24	125	217	1.73
					48	134	251	1.87
					72	139	295	2.12
10	untr.	4.4	117	22	24			
	10.5		33	112	24	193	112	0.58
					48	208	129	0.62
					72	232	188	0.81
18	untr.	8.8	127	19	24			
	15.6		38	302	24	211	55	0.26
					48	224	68	0.30
					72	256	82	0.32
21	43.6	9.4	11	600	24	245	24	0.10
					48	250	66	0.26
					72	271	88	0.32
25	22.8	10.7	35	260	24	214	24	0.12
					48	233	44	0.18
					72	244	95	0.39

Table 1b Petrographic examination, ASTM C 289 and kinetic tests results

Sample	Residue (%)	Petrogr. Flint (%)	ASTM-C 289 Rc	ASTM-C 289 Sc (mmol/l)	Attack time (h)	Kinetic test Na_2O	Kinetic test SiO_2 (mmol/l)	Kinetic test SiO_2/Na_2O
29	untr.	17.3	132	37	24			
	28.4		45	143	24	214	8	0.04
					48	233	22	0.09
					72	244	89	0.37
38	untr.	2.5	27	29	24			
	94.4		55	110	24	121	217	1.80
					48	130	251	1.93
					72	133	294	2.21
40	71.6	7.3	42	58	24	227	221	0.97
					48	244	252	1.03
					72	250	316	1.27
42	40.0	13.5	140	146	24	190	16	0.08
					48	218	26	0.12
					72	315	49	0.16
59	64.0	17.5	115	46	24	210	133	0.63
					48	220	163	0.74
					72	249	208	0.84
60	27.6	4.6	115	51	24	244	383	1.57
					48	245	537	2.20
					72	274	614	2.24
62	untr.	14.5	150	12	24			
	67.6		90	130	24	259	155	0.60
					48	267	194	0.73
					72	282	220	0.78
121	25.4	5.4	85	200	24	212	150	0.70
					48	241	186	0.77
					72	256	199	0.78
124	9.9	1.6	75	110	24	110	220	2.00
					48	115	253	2.20
					72	132	306	2.32

The ASTM-C 289 method and the Kinetic test require both, to classify the degree of risk of the aggregates tested and the placement of the data in diagrams. In our case, the great number of points to be placed could cause a bad readability of these diagrams, so it was decided to simply point out, in the following Tab.2, the potential reactivity showed by the alluvia examined by means of the three testing methods used (optical, ASTM and Kinetic).

It must be remembered that the potential reactivity is not strictly corresponding to dangerousity, this parameter being almost null for low and very high reactive minerals content. So, the aggregates classed in table 2 with a potential reactivity low or high are normally classed as innocuous in the dangerousity diagrams.

Table 2 Potential reactivity of the aggregates tested with optical, ASTM and Kinetic methods

Sample	River	Potential reactivity Optical	ASTM	Kinetic	Sample	River	Potential reactivity Optical	ASTM	Kinetic
3 *	Biferno	M	M	M	40	Santerno	M	L	M
7 *	Sangro	ML	ML	H	42 *	Sillaro	H	L	L
10	Pescara	ML	ML	M	59	Fortore	H	L	M
18	Salinetto	MH	M	L	60 *	Cervaro	ML	L	H
21	Tronto	MH	MH	L	62	Ofanto	H	ML	M
25	Chienti	MH	M	L	121 *	Trigno	M	ML	M
29	Esino	H	ML	L	124	Treste	L	ML	H
38	Lamone	L	ML	H					

Potential reactivity: L = low, M = medium, H = high

Optical tests. Flint: < 3 % = L, 3÷6 % = ML, 6÷8 % = M, 8÷12 % = MH, > 12 % = H

* Rivers whose aggregates surely caused structural damages

CONCLUSIONS

The tests carried out show a fairly satisfactory correlation between the results furnished by optical and ASTM-C 289 methods, correlation that would be good slightly changing (moving rightward) the lower part of the separation curve between the "innocuous" and "deleterious" fields in the ASTM diagram.

On the contrary, the kinetic test results are unsatisfactory, mainly for the aggregates with very low and very high flint contents, that are normally respectively classed as highly and lowly reactive (and, for the high flint contents, with very low amounts of dissolved SiO_2).

The study is still in progress, and the data collected are for the moment in a reduced number; nevertheless, the ASTM-C 289 method once more proved to be the most reliable chemical method (in spite of its scarce reproducibility, perhaps due also to the high variations in the composition of the alluvia sampled and to the reduced weight of the samples tested, not always representative in spite of the accuracy in quartering) for potential reactivity testing of aggregates having, as the italian ones, a low ratio between the reactivity degree and the flint content.

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ALKALI-REACTIVITY OF SOME JAPANESE CARBONATE ROCKS BASED ON STANDARD TESTS

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ABSTRACT

A series of standard alkali-reactivity tests were made of impure carbonate rocks from Japan. It was suggested that Japanese carbonate rocks that contain more than 10 % of cryptocrystalline quartz and/or acid-insoluble residue can be deleteriously reactive, according to the CSA revised concrete prism test and the CSA accelerated mortar bar test. Some non-dolomitic and dolomitic limestone showed an early deleterious expansion in the concrete prism test, comparable to an alkali-carbonate reactive Canadian aggregate, but were found related to the cryptocrystalline quartz contained in the siliceous and argillaceous matrix or chert nodules in these rocks. This paper discusses the findings and problems of several standard tests applied to Japanese rocks.

Keywords: Accelerated mortar bar test, carbonate rocks, concrete prism test

INTRODUCTION

In Japan, commercially quarried carbonate rocks are generally massive and pure, containing more than 53% of CaO, and are lithologically different from alkali-reactive, impure bedded carbonate rocks in North America. This study focuses on the potential reactivity of some exceptionally impure carbonate rocks in Japan, to examine the potential for alkali-reactivity.

Several Japanese Paleozoic carbonate rocks, not commercially quarried, and two reference Canadian carbonate aggregates (*Table 1*) were selected for standard aggregate testing for alkali-reactivity, such as the CSA concrete prism and accelerated mortar bar tests, the JIS mortar bar test, the ASTM chemical and rock cylinder tests, and petrographic examinations to check the presence of reactive phases. This paper describes the results of these tests.

Table 1 Compositions of some Japanese Paleozoic carbonate rocks

no.	Sample	SiO_2	TiO_2	Al_2O_3	Fe_2O_3	MgO	CaO	Na_2O	K_2O	P_2O_5	Ig. loss	Insol.
1	Limestone	4.16	0.04	0.32	0.25	0.76	51.68	0.01	0.05	0.03	42.45	4.51
2	Limestone	3.03	0.09	0.49	0.80	1.48	50.77	0.01	0.12	0.04	42.67	5.11
3	Limestone	5.97	0.09	0.46	0.67	1.06	49.42	0.05	0.14	0.01	41.13	7.64
4	Limestone	8.10	0.10	1.74	1.15	2.34	47.86	0.14	0.14	0.00	38.06	9.33
5	Limestone	7.99	0.14	0.80	1.08	1.03	48.16	0.14	0.17	0.08	38.96	11.11
6	Limestone	12.21	0.02	0.78	0.34	1.68	44.69	0.12	0.24	0.14	38.35	12.42
7	Limestone	10.72	0.06	0.35	0.50	2.00	46.20	0.02	0.10	0.07	39.60	12.33
8	Argil. ls	26.53	0.34	2.93	2.13	1.67	35.46	0.31	0.50	0.10	29.27	28.33
9	Ls (chert, meta)	28.39	0.05	1.15	0.44	0.88	37.48	0.16	0.20	0.03	30.95	30.15
10	Ls (chert)	39.57	0.02	0.20	0.13	0.45	32.74	0.03	0.05	0.04	26.23	40.17
11	Ls (chert)	50.57	0.01	0.14	0.29	1.07	25.16	0.01	0.02	0.01	22.44	52.12
12	Dol. limestone	2.25	0.05	0.34	0.84	7.40	45.27	0.01	0.05	0.05	43.34	3.48
13	Dol. limestone	6.36	0.31	1.59	1.70	10.29	38.10	0.10	0.34	0.08	40.03	8.92
14	Dol. limestone	8.74	0.09	2.20	0.78	5.53	42.36	0.08	0.54	0.01	39.46	9.66
15	Dol. ls (chert)	30.98	0.05	0.23	0.73	10.88	24.67	0.05	0.10	3.81	28.17	38.26
16	Dolostone	0.38	0.00	0.02	0.05	18.31	34.20	0.05	0.04	0.07	46.54	0.49
17	Dolostone	0.55	0.00	0.13	0.04	18.87	33.80	0.02	0.02	0.03	46.16	2.12
18	Dolostone	1.23	0.02	0.18	0.19	18.42	34.88	0.01	0.02	0.10	44.84	2.86

Canadian reference: no. 6 Spratt (near Ottawa), no. 14 Pittsburg (Kingston)

PETROGRAPHY OF CARBONATE ROCKS

Paleozoic carbonate rocks from Japan and Canada contain similar impurities, consisting mainly of illite, chlorite, authigenic cryptocrystalline quartz and detrital quartz (***Table 2***). They are also common to matrix constituents of argillaceous and arenaceous rocks which cause an alkali-silica reaction due to crypto- to microcrystalline quartz (e.g.Lewczuk et al.1990), fomerly called the alkali-silicate reaction. Thus the potential reactivity of carbonate rocks should also be examined in light of the alkali-silica reaction of this mineral.

Limestone

Exceptionally impure portions from Paleozoic limestone beds, containing chert nodules, argillaceous laminations or stylolite seams, were selected for examination. Crypto- to microcrystalline quartz occurs either in the matrix of a bedded limestone (no.7) similar to the Spratt aggregate (no.6), or as chert nodules in association with chalcedony, sometimes replacing fossil fragments (no.11,12). The size of the cryptocrystalline quartz is generally finer than that of common Paleozoic bedded cherts, suggesting a higher reactivity. By contrast, a weakly metamorphosed limestone, interbedded with chert layers (no.9) of coarsely-grained quartz, suggests lower reactivity.

Dolomitic limestone

Japanese Paleozoic dolomitic limestones are generally massive and coarsely crystalline, containing dolomite rhombs of 100-200 microns across (no.12,13) along with recrystallized calcite of crinoid fragments. They are coarser in texture than Pittsuburg aggregate (no.14). Chalcedony and cryptocrystalline quartz occur as chert nodules and a siliceous matrix in a dolomitic limestone (***Fig.1***,no.15), which suggests a potential for alkali-silica reactivity.

Some impure dolomitic rocks form a "floating texture" of dolomite rhombs surrounded by argillaceous to siliceous matrices. This is because impurities in rocks present abundant nuclei for crystallization of dolomite. A laminated argillaceous dolomitic limestone from Japan has such a floating texture, composed of zoned dolomite of 40-100 microns across (***Fig.2***,no.13), similar to the typical texture of Pittsburg aggregate. One might therefore expect that this rock would show alkali-carbonate reactivity (CSA-A23.1, Appx B3.2, 1994).

The Pittsburg aggregate is a finely laminated rock, rich in argillaceous laminae and stylolite seams (***Fig.7***) consisting of cryptocrystalline quartz, illite and detrital quartz. The high alkali content of this rock is due to illite, which may favor duration of alkali-aggregate reaction in this rock.

Dolostone

A pure type of Japanese Paleozoic dolostone was examined. The samples had a coarsely grained mosaic texture composed of dolomite crystals of 200-350 microns across. Microscopic observation revealed that cryptocrystalline quartz is the main constituent of the acid insoluble residue in these rocks (no.17,18).

Fig.1 Chert nodule in dolomitic limestone (Japan,no.15)

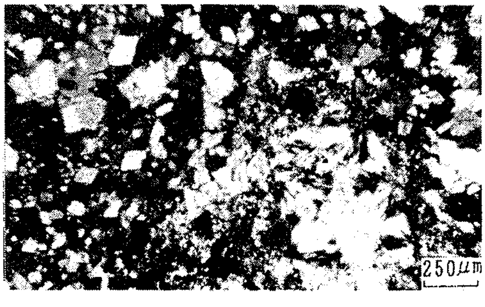
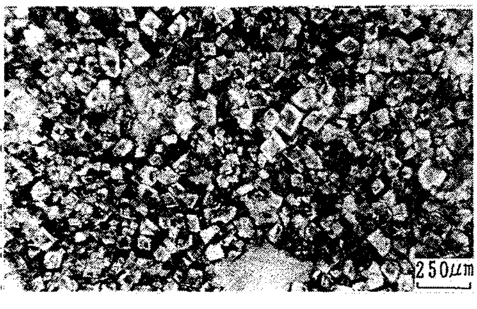


Fig.2 Floating texture in dol.limestone (Japan,no.13)



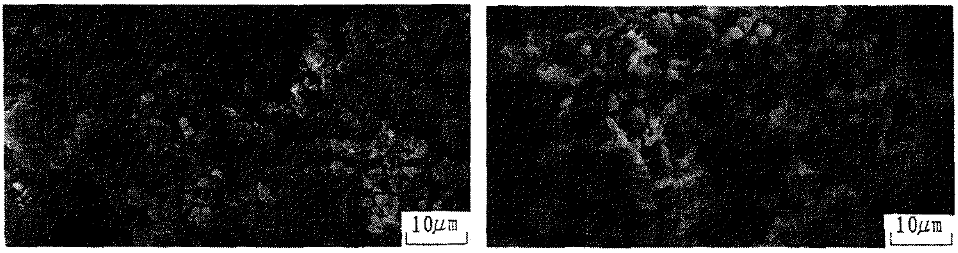
Crystallinity index of quartz

This index (CI) was determined by XRD analysis of HCl-acid insoluble residues. The CI ranged about 4-6 for rocks containing cryptocrystalline quartz (***Table2***), which is lower than that of common Paleozoic chert beds in Japan (Katayama & Futagawa 1989) and suggestive of a higher potential reactivity of this quartz in the carbonate rocks. Detrital quartz has a near perfect crystallinity, i.e. CI of nearly 10, and even in small amounts may overwhelm the intensity of poorly crystallized authigenic quartz associated. For instance, Pittsburg aggregate (no.14), dominated by cryptocrystalline quartz, presents a CI nearly 10 due to detrital quartz. Therefore, this method may not be usable for predicting alkali-silica reactivity of heterogeneous rocks (Wigum 1995).

Phosphoric acid treatment of carbonate rocks

This method completely dissolves carbonates, clay minerals and feldspars, and extracts only silica minerals hidden in the argillaceous matrix of carbonate rocks. SEM observation (***Fig.3***) of the extracts confirmed that Pittsburg aggregate is rich in cryptocrystalline quartz of less than 5 microns across, constituting 85% of the silica residue, and that detrital quartz visible in thin section is a minor constituent. Likewise, Japanese non-dolomitic and dolomitic limestones (nos.7,15) contain abundant cryptocrystalline quartz, which suggests the alkali-silica reactivity of these rocks. The cryptocrystalline quartz in the carbonate rocks has long been missed by thin section microscopy.

Fig.3 Cryptocrystalline quartz extracted by phosphoric acid



A) Limestone (Japan,no.7) ***B) Dol. limestone (Kingston,no.14)***

Table 2 Impurities in some Paleozoic carbonate rocks from Japan

no.	Sample	XRD of HCl insol. sm	ch	il	qz	fs	CI of qz	SEM of quartz (H_3PO_4 insol.) Cryptocryst.	Megacryst.
1	Limestone		+	+	++++		8.5		
2	Limestone	+		+	+++		9.1		
3	Limestone	+		+	+++	+	7.5		
4	Limestone		++	+	++++		9.7		
5	Limestone		+	+	++++		10.7		
6	Limestone		+	+	++++		6.0	3-10 μm 65%	35%*
7	Limestone			+	++++		4.8	1- 3 μm 98%	< 2%
8	Argil.ls		+	+	++++	++	6.1		
9	Ls (chert, meta)			+	++++		9.1		
10	Ls (chert nodule)				++++		5.1		
11	Ls (chert nodule)				++++		6.5		
12	Dol.limestone		+		++++		9.8		
13	Dol.limestone		+	+	++++		9.8		
14	Dol.limestone		+	++	+++	+	9.9	2- 5 μm 85%	15%**
15	Dol.ls (chert nod)				++++		4.4	2- 7 μm 95%	< 5%
16	Dolostone		+	+++	++	++			
17	Dolostone				++++	+	4.1		
18	Dolostone				++++		6.2		

CI: crystallinity index, normalized to 10.0 for a Japanese pegmatite quartz
sm: smectite, ch: chlorite, il: illite, qz: quartz, fs: feldspar
no.6 Spratt, no.14 Pittsburg, * mosaic of chalcedony, ** detrital quartz

STANDARD TESTS OF CARBONARE ROCKS

Concrete prism test

The CSA new version standard test (CSA A23.2-14A, 1994) was conducted up to 6 months, not 1 year as in the standard, due to facility limitations (***Fig.4***, ***Table 3***). Even though, carbonate rocks containing chert nodules were found to be deleteriously late-expansive, exceeding a 1 year limit of expansion of 0.04% even at 6 months. In contrast, rocks rich in cryptocrystalline quartz in the matrix (nos.7,14,15) were early-expansive, reaching the 0.04% limit as early as 1 week, and this is the first example that Japanese non-dolomitic and dolomitic limestones tested as deleterious in the Canadian concrete prism test.

The magnitude of the expansion rate in ***Fig.4*** is similar to that found in the so-called alkali-carbonate reactive dolomitic limestones of the Pittsburg quarry (Grattan-Bellew 1993). However, present test results suggest that a rapid expansion can be caused by the cryptocrystalline quartz with low CI (<5) in the Japanese rocks. This may also be the case with the Pittsburg aggregate (no.14) because it contains large amounts of cryptocrystalline quartz (***Table 2***, ***Fig.3***), and its continued expansion may be due to the high alkali content of this rock due to illite (***Table 1***) which likely supplied alkali to maintain the reaction. An impure alkali-silica reactive limestone aggregate in Ontario has been known to have supplied alkali from illite in the argillaceous matrix to a deteriorated field concrete structure (Grattan-Bellew 1994).

Fig.4 ***Results of CSA concrete prism test (420kg/m³, 38°C) of some Japanese carbonate rocks***

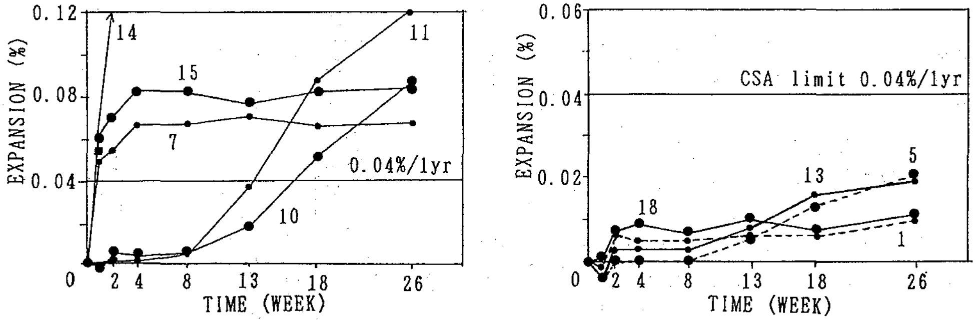


Table 3 ***CSA concrete prism test (420kg/m³, 38°C) of Japanese carbonate rocks***

no.	Sample	1w	2w	4w	8w	13w	18w	26w
1	Limestone	-0.001	0.006	0.005	0.005	0.006	0.006	0.010
2	Limestone	0.005	0.000	0.013	0.017	0.011	0.010	0.012
3	Limestone	0.001	0.000	0.003	0.004	0.005	0.007	0.011
4	Limestone	0.001	0.003	0.002	0.001	0.006	0.004	0.015
5	Limestone	-0.004	0.000	0.000	0.000	0.005	0.013	0.020
7	Limestone	0.049*	0.053*	0.067*	0.067*	0.070*	0.066*	0.067*
8	Argil.ls	-0.007	0.002	0.002	0.000	0.004	0.003	0.007
9	Ls (chert,meta)	-0.002	0.001	0.000	0.000	0.005	0.005	0.018
10	Ls (chert nod)	-0.001	0.005	0.004	0.006	0.019	0.052*	0.086*
11	Ls (chert nod)	0.000	0.002	0.002	0.005	0.038	0.087*	0.120*
12	Dol.limestone	-0.001	0.005	0.004	0.003	0.005	0.007	0.009
13	Dol.limestone	-0.004	0.003	0.003	0.003	0.008	0.016	0.019
14	Dol.limestone	0.055*	0.122*	0.187*	0.220*	0.261*	0.278*	0.297*
15	Dol.ls (chert)	0.061*	0.069*	0.083*	0.082*	0.078*	0.082*	0.083*
16	Dolostone	0.002	0.000	0.008	0.004	0.004	0.002	0.003
17	Dolostone	0.002	0.008	0.010	0.009	0.011	0.008	0.012
18	Dolostone	0.001	0.007	0.009	0.007	0.010	0.007	0.011

* Expansion > 0.04%, Canadian reference: no.14 Pittsburg

Accelerated mortar bar test

This North American new standard test (CSA A23.2-25A 1994) was made on some Japanese carbonate rocks, to compare with the JIS A 5308 mortar bar test done in parallel. Both ***Fig.5*** and ***Table 4*** show that all of the impure carbonate rocks from Japan and Canada, that contain more than 10% of insoluble residue and/or chert nodules (crypto to microcrystalline quartz), produced deleterious expansion in this test, exceeding the 0.15% CSA limit before 14 days. In contrast, the conventional JIS mortar bar test detected deleterious expansion from only one limestone that contained 50% of chert nodules (see ***Table 1***).

The accelerated mortar bar test has been used successfully in Quebec, Canada to evaluate impure carbonate aggregates, but a more critical 0.10% expansion limit after 14 days has been suggested based on concrete prism tests and the field performance of concrete (Fournier & Berube 1991). In Japan, no field record is available of impure carbonate rocks as to judge which test is suitable for evaluating these rocks, because they are not commercially quarried for concrete aggergates in Japan. The CSA concrete prism test, although with limited data up to 6 months (***Table 3***), seems to give a moderate result between the accelerated and conventional mortar bar tests. It will therefore be necessary to examine the expansion limit in the accelerated mortar bar test, when applying this method to various Japanese carbonate rocks.

Fig.5 ***Results of CSA accelerated mortar bar test of some Japanese carbonate rocks***

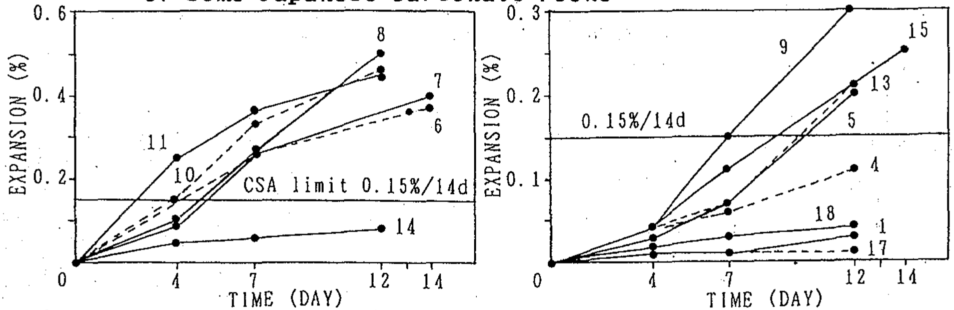


Table 4 ***CSA accelerated mortar bar test & JIS mortar bar test of some Japanese carbonate rocks***

no.	Sample	Accelerated mortar bar test 4d	7d	12d	13d	14d	JIS mortar bar test 2w	4w	8w	13w	26w
1	Limestone	0.01	0.01	0.03			0.005	0.006	0.012	0.010	0.012
2	Limestone						0.000	0.001	0.007	0.009	0.010
3	Limestone						0.000	0.001	0.006	0.009	0.012
4	Limestone	0.04	0.06	0.11			0.006	0.007	0.018	0.016	0.021
5	Limestone	0.03	0.07	0.20*			0.006	0.008	0.020	0.018	0.025
6	Limestone		0.26*		0.36*	0.37*					
7	Limestone	0.09	0.26*			0.40*	0.001	0.005	0.014	0.019	0.021
8	Argil.ls	0.10	0.27*	0.50*			0.002	0.002	0.012	0.013	0.015
9	Ls(chert, meta)	0.04	0.15*	0.30*			0.006	0.008	0.019	0.022	0.027
10	Ls (chert nod)	0.15*	0.33*	0.46*			0.007	0.011	0.033	0.042	0.049
11	Ls (chert nod)	0.25*	0.36*	0.45*			0.013	0.024	0.085	0.124+	0.134+
12	Dol. limestone	0.02	0.01	0.02			0.005	0.005	0.015	0.010	0.015
13	Dol. limestone	0.04	0.07	0.21*			0.007	0.005	0.018	0.017	0.023
14	Dol. limestone	0.05	0.06	0.08			0.029	0.037	0.052	0.056	0.070
15	Dol. ls (chert)	0.04	0.11			0.25*	0.000	0.005	0.014	0.018	0.019
16	Dolostone						0.000	0.004	0.011	0.014	0.015
17	Dolostone	0.01	0.01	0.01			0.005	0.009	0.012	0.010	0.014
18	Dolostone	0.02	0.03	0.04			0.008	0.009	0.015	0.014	0.019

* Expansion > 0.15%, + Expansion > 0.10%
Canadian references: 6 Spratt (CSA interlaboratory test, 1994), 14 Pittsburg

Rock cylinder test

Carbonate rocks without chert inclusions were tested according to ASTM C 586. Since these rocks are heterogeneous, having bedding planes and laminations, cylinders were taken in three directions from each sample, i.e. two crossing directions (x,y) parallel to, and one (z) perpendicular to the bedding plane.

Japanese carbonate rocks comprise the 1) steadily shrinking type, and the 2) late-expansive type which expanded after a long period of contraction (no.13) (*Table5*). A reference Pittsburg aggregate (no.14), representing the 3) most deleterious early-expansive type, expanded more than 0.1% by 28 day. All these types are known in the North American Paleozoic carbonate rocks (Lemish & Moore 1964), but their petrography is different in Japan.

The late-expansive dolomitic limestone (no.13) is a laminated rock with a "floating texture" of coarse-grained dolomite rhombs (*Fig.2*), resembling the typical alkali-carbonate reactive dolomitic limestone (CSA-A23.1). This rock was deleterious in the accelerated mortar bar test, and a rock cylinder perpendicular to the lamination (z) produced expansion after 4 months (*Fig.6*).

The Pittsburg aggregate examined (no.14) is a finely-laminated dolomitic limestone, with numerous parallel laminations of argillaceous materials, sometimes stylolitic, composed mainly of cryptocrystalline quartz and illite (*Table 2, Fig.7*). This highly expansive rock does not present a typical floating texture, because dolomite crystals are sparse, less than 30% in the carbonate fraction. Thus, the well-known relationship between the petrography and expansivity of carbonate rocks should be reinvestigated without prejudice.

Fig.6 Rock cylinder test of Japanese carbonate rocks

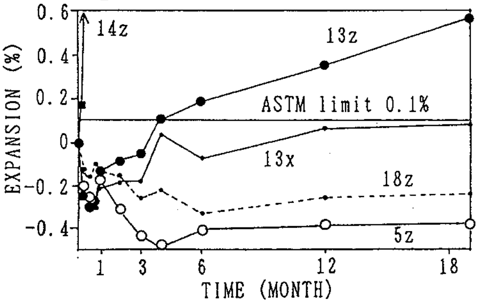


Fig.7 Laminated texture of dol. limestone (Kingston,no.14)

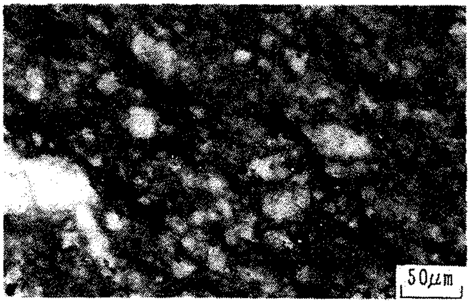


Table5 Rock cylinder test of Japanese non-cherty carbonate rocks

no.	Sample		7d	14d	21d	28d	2m	3m	4m	6m	1y	1y7m
5	Limestone bedded	x	-0.17	-0.24	-0.27	-0.17	-0.30	-0.43	-0.43	-0.43	-0.40	-0.43
		y	-0.19	-0.26	-0.28	-0.19	-0.31	-0.43	-0.46	-0.42	-0.41	-0.45
		z	-0.20	-0.26	-0.26	-0.17	-0.32	-0.43	-0.49	-0.40	-0.38	-0.38
8	Argil.ls bedded	x	-0.15	-0.22	-0.21	-0.22	-0.30	-0.39	-0.40	-0.47	-0.41	-0.43
		y	-0.16	-0.20	-0.22	-0.21	-0.28	-0.37	-0.39	-0.43	-0.36	-0.38
		z	-0.12	-0.18	-0.19	-0.21	-0.29	-0.36	-0.34	-0.43	-0.35	-0.33
12	Dol.ls bedded	x	-0.20	-0.23	-0.24	-0.18	-0.28	-0.37	-0.33	-0.40	-0.34	-0.38
		y	-0.21	-0.27	-0.28	-0.23	-0.31	-0.40	-0.43	-0.45	-0.41	-0.43
		z	-0.24	-0.28	-0.23	-0.25	-0.34	-0.40	-0.45	-0.46	-0.45	-0.46
13	Dol.ls laminated	x	-0.23	-0.29	-0.29	-0.21	-0.18	-0.18	0.03	-0.07	0.06	0.08
		y	-0.25	-0.29	-0.28	-0.16	-0.21	-0.21	-0.12	-0.09	0.01	0.06
		z	-0.24	-0.30	-0.27	-0.13	-0.08	-0.06	0.10	0.18	0.34	0.57
14	Dol.ls finely laminated	x	0.44	1.23	2.11	2.72	5.64	7.15	8.11	8.49	8.84	9.51
		y	0.36	1.09	2.03	2.67	5.50	7.12	8.25	8.90	9.21	9.45
		z	0.18	1.18	2.52	3.31	7.78	10.45	12.25	13.40	14.14	13.60
18	Dolostone massive	x	-0.12	-0.17	-0.16	-0.19	-0.27	-0.36	-0.34	-0.42	-0.37	-0.38
		y	-0.14	-0.15	-0.15	-0.15	-0.16	-0.23	-0.27	-0.39	-0.37	-0.34
		z	-0.13	-0.17	-0.10	-0.13	-0.16	-0.26	-0.22	-0.32	-0.25	-0.23

no.14 Pittsburg aggregate, x, y: parallel, z: perpendicular to bedding plane

Chemical test

Impure carbonate rocks were subjected to the ASTM C289 chemical test (***Table 6***). Dissolved silica (Sc) increased with the increased content of insoluble residue in these rocks, proportionally to the content of chert nodules composed mainly of crypto- to microcrystalline quartz. Weakly metamorphosed limestone, containing bedded chert, has a low Sc, because the included quartz is coarsely grained and thus less reactive.

A modified chemical test was made for selected samples using HCl-insoluble residue, to remove interferences from carbonate minerals on Sc & Rc values. A tentative result is shown in the ASTM diagram (***Fig.8***), with the Sc & Rc values normalized to a residue amount of 12.5g, to compare with Canadian data. Many researchers use only Sc values for consideration, since Rc values are affected by many factors. In Quebec, Canada, values with 1) Sc>100 mmol/l, or 2) corrected Sc*>10 mmol/l (Sc*= Sc x insol.%/100), or 3) insoluble residue > 5%, are considered as indications of deleterious Paleozoic carbonate rocks, based on the correlative data of the CSA concrete prism and the accelerated mortar bar tests (Fournier & Berube 1990). Impure carbonate rocks from Japan have the same magnitude of Sc & Sc* as the Canadian rocks (***Table6***), giving a prospective impression for the items 1) & 2), but further study coupled with other tests is needed to apply this method to Japanese carbonate rocks.

Fig.8 Chemical tests of some impure Japanese carbonate rocks

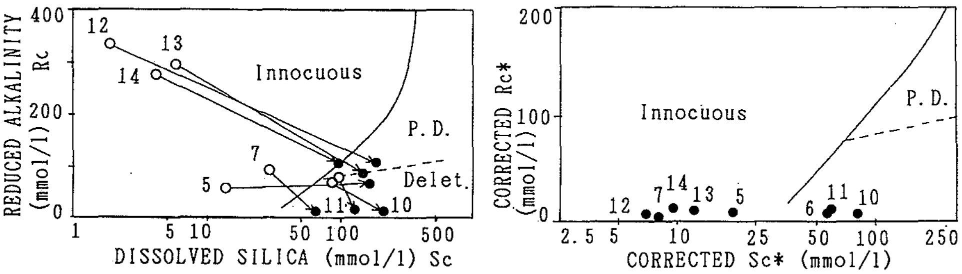


Table 6 Chemical tests of some Japanese carbonate rocks

no.	Sample	ASTM C289 Sc (mmol/l)	ASTM C289 Rc (mmol/l)		Modified test for insol. Sc (mmol/l)	Modified test for insol. Rc (mmol/l)	Sc* (mmol/l)	Rc* (mmol/l)	Expansion tests CP	Expansion tests AM	Insol. (%)
1	Limestone	6	29	I							4.51
2	Limestone	9	54	I							5.11
3	Limestone	9	49	I							7.64
4	Limestone	19	55	I							9.33
5	Limestone	12	57	I	177	67	20	7		#	11.11
6	Limestone				449	66	56	8		#	12.42
7	Limestone	27	97	I	67	9	8	1	#	#	12.33
8	Argil.ls	37	98	I						#	28.33
9	Ls (chert, meta)	11	39	I						#	30.15
10	Ls (chert nodule)	84	68	D	194	18	78	7	#	#	40.17
11	Ls (chert nodule)	97	88	D	118	19	61	10	#	#	52.12
12	Dol.limestone	2	334	I	195	108	7	4			3.48
13	Dol.limestone	6	299	I	147	94	13	8		#	8.92
14	Dol.limestone	4	282	I	93	115	9	11	#		9.66
15	Dol.ls (chert nod)	65	320	I					#	#	38.26
16	Dolostone	1	395	I							0.49
17	Dolostone	0	372	I							2.12
18	Dolostone	1	384	I							2.86
19	Chert nodule	148	125	PD							

no.6 Spratt, no.14 Pittsburg, no.19 isolated from limestone no.10
* Corrected value, multiplied by insoluble residue content
I: innocuous, PD: potentially deleterious, D: deleterious
CP: concrete prism test, AM: accelerated mortar bar test, #: deleterious

CONCLUSIONS

- The SEM observations of the extracted residue from phosphoric acid treatment revealed that some impure dolomitic (Pittsburg) and non-dolomitic limestone (Spratt) from both Canada and Japan, contained abundant cryptocrystalline quartz, which suggested their higher potential for alkali-silica reactivity.
- The CSA concrete prism test gave moderate expansions among the testing methods examined, which revealed that some of Japanese non-dolomitic and dolomitic limestones, containing more than 10% of cryptocrystalline quartz and/or acid insoluble residue, were deleterious.
- The CSA accelerated mortar bar test was very sensitive and overestimated the potential reactivity of Japanese carbonate rocks, while the conventional JIS mortar bar test clearly underestimated the reactivity. The expansion limit of this accelerated test needs investigation when applied to Japanese rocks.
- The ASTM rock cylinder test revealed the presence of a late-expansive dolomitic limestone in Japan, also deleterious in the accelerated mortar bar test. The Canadian modified chemical test appeared applicable to siliceous carbonate rocks in Japan, but needs further study coupled with other methods.

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INTER-LABORATORY STUDY ON THE CSA A23.2-14A CONCRETE PRISM TEST FOR ALKALI-SILICA REACTIVITY IN CONCRETE

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ABSTRACT

Two alkali-silica reactive aggregates were used in this study in which 24 laboratories from Canada, one from the U.S.A. and two from France participated. Each participant was asked to make three concrete mixtures according to the CSA A23.2-14A-M94 Concrete Prism Test method, to cast a number of test prisms from each one of these mixtures and to test them for expansion in different storage conditions.

The results obtained in this study showed that the use of well-controlled testing conditions or parameters in the concrete prism test procedure, such as a reference sand, a reference cement, a standard storage container and fixed concrete mixture proportioning, can greatly contribute in reducing the between-laboratory variability of the test results. For both aggregates investigated, the between-laboratory standard deviation and the coefficient of variation were significantly lower for the test prisms in the NaOH solution than for those subjected to control storage condition.

Key words: Aggregate, Alkali-aggregate reaction, Concrete prism test, Interlaboratory study, Test methods.

INTRODUCTION

In the late 1980's and early 1990's, concern had been raised that the potential alkali-reactivity of a number of concrete aggregates used in Eastern Canada, particularly some marginally or slowly reactive quartz-bearing rocks, was not correctly appraised by the CSA A23.2-14A-M90 Concrete Prism Test. Consequently, a series of changes were adopted for the 1994 edition of the above Standard, which consisted mainly of: 1), increasing the cement content in the concrete mixture from the 310 kg/m^3 to 420 ± 10 kg/m^3, 2) raising the total alkali content in the mixture to 1.25% Na_2O equivalent, by mass of cement, i.e. total of 5.25 kg/m^3 expressed as Na_2O equivalent, 3), adopting more precise mixture proportioning parameters, and 4), using a "standard" storage container. An interlaboratory study was then undertaken by CANMET to investigate the between-laboratory variation of the modified test procedure.

INVESTIGATION PROGRAM

A total of 27 laboratories, 24 from Canada, one from the U.S.A. and two from France, participated in this study. Each participant received materials, supplies, and instructions to make three concrete mixtures according to CSA A23.2-14A-M94 Concrete Prism Test Method. Table 1 gives the details on the various concrete mixtures made in this study (Fournier *et al.* 1995). For mixture 1, specific quantities of the Spratt limestone, CANMET control non-reactive fine aggregate, ASTM Type I high-alkali control cement, and NaOH pellets were prepared and preweighed in CANMET laboratories; these were supplied to the participants for mixing without any further preparation using a specified amount of tap water (Table 1). For mixtures 2 and 3, 100-kg of unscreened Spratt limestone (Mixture 2) and Sudbury Gravel (Mixture 3) aggregates were provided to the participants for use in concrete mixtures according to the CSA A23.2-14A-M94 test procedure; each participant was required to use a local fine aggregate and normal Portland cement meeting the requirements of the test procedure (Table 1). From each of the above three concrete mixtures, test cylinders for compressive strength determination at 28 days, and test prisms for length change determination under specified high temperature and humidity conditions, were to be made.

Table 1: Scope of the study

Mixture designation	Aggregates[2]		Cement		Water Content	Alkalies in the mix	Concrete Specimens to be cast	Storage conditions[4]
	Coarse	Fine	Type	Content				
1 "Ready to use"	**Spratt Limestone** batch: 57.3 kg	CANMET (control) (non-reactive)[3] batch: 38.2 kg	CSA Type 10 (ASTM Type I) (CANMET control) 0.85% Na_2O equivalent.	420 kg/m^3 batch: 22.3 kg	Effective w/c: 0.42 batch: 9.6 kg	1.25% (Na_2Oe) by cement mass i.e. total alkali content of 5.25 kg/m^3 In the batch: 115 g of NaOH	• Six + three (optional) prisms, 75 by 75 by 275 to 405 mm in size, for AAR expansion testing. • Two cylinders, 100 by 200 mm in size, for compressive strength determination at 28 days.	A) Prisms directly in plastic pails at 38°C and R.H. > 95%. B) Prisms wrapped in damp wipers, placed in plastic bags and then in a plastic pail; pail stored at 38°C. C) Optional: storage at 38°C but using containers currently being used by the participants.
2 According to CSA test procedure A23.2-14A[1]	**Spratt Limestone** 100 kg of < 25 mm particles provided by CANMET to be graded by the participant	Non-reactive M.F. of 2.70 ± 0.2 **local source**	CSA Type 10 (ASTM Type I) Alkali content: 0.9% ± 0.1 Na_2O equiv. **local source**	420 ± 10 kg/m^3	Effective w/c: 0.42 to 0.45	1.25% (Na_2Oe) by cement mass i.e. total alkali content of 5.25 kg/m^3 NaOH pellets to be added: **local source**	• Three + three (optional) prisms, 75 by 75 by 275 to 405 mm in size, for AAR expansion testing. • Two cylinders, 100 by 200 mm in size, for compressive strength determination at 28 days.	B) Prisms wrapped in damp wipers, placed in plastic bags and then in a plastic pail; pail stored at 38°C. C) Optional: storage at 38°C but using containers currently being used by the participants.
3 According to CSA test procedure A23.2-14A	**Sudbury Gravel** 100 kg of < 25 mm particles provided by CANMET to be graded by the participant	Non-reactive M.F. of 2.70 ± 0.2 **local source**	CSA Type 10 (ASTM Type I) Alkali content: 0.9% ± 0.1 Na_2O equiv. **local source**	420 ± 10 kg/m^3	Effective w/c: 0.42 to 0.45	1.25% (Na_2Oe) by cement mass i.e. total alkali content of 5.25 kg/m^3 NaOH pellets to be added: **local source**	• Six + three (optional) prisms, 75 by 75 by 275 to 405 mm in size, for AAR expansion testing. • Two cylinders, 100 by 200 mm in size, for compressive strength determination at 28 days.	A) Prisms directly in plastic pails at 38°C and R.H. > 95%. B) Prisms wrapped in damp wipers, placed in plastic bags and then in a plastic pail; pail stored at 38°C. C) Optional: storage at 38°C but using containers currently being used by the participants.

[1] CSA Test procedure: CSA-A23.2-14A. Potential Expansivity of Aggregates (Procedure for Length Change Due to Alkali-Aggregate Reaction in Concrete Prisms). CSA A23.2-M94: Methods of Test for Concrete, Canadian Standards Association, ISSN 0317-5669, Rexdale, Ontario, pp. 205-214.

[2] Coarse to fine aggregate ratio of 60 : 40 by mass.

[3] Non-reactive fine aggregate, i.e. producing expansion < 0.10% at 14 days in the following test methods: ASTM Designation C 1260-94. Standard Practice for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method). Annual Book of ASTM Standards Vol. 04.02, Concrete and Mineral Aggregates, American Society for Testing and Materials, Philadelphia, U.S.A. or CSA-A23.2-25A. Test Method for Detection of Alkali-Silica Reactive Aggregate by Accelerated Expansion of Mortar Bars. CSA A23.2-M94: Methods of Test for Concrete, Canadian Standards Association, ISSN 0317-5669, Rexdale, Ontario, pp. 236-242.

[4] A total of five 20 to 25-litre plastic pails were provided by CANMET to each participant to store concrete prisms of mixtures 1 to 3 under conditions (A) and (B).

CASTING, STORAGE AND TESTING OF SPECIMENS

Specimens for AAR Testing

A total of fifteen concrete prisms, i.e. six each for mixtures 1 and 3, and three for mixture 2, were to be made by the participants (Table 1). Enough material was sent to the participants so that those interested in casting additional sets of test prisms to evaluate their own storage containers may do so. The test prisms had dimensions of not less than 75 by 75 by 275 mm and not more than 75 by 75 by 405 mm.

Storage of test specimens

Each participant was provided with five storage containers. These consisted of 22-litre plastic pails for the participants using 300 to 360 mm-long concrete prisms, and of 25-litre plastic pails for the participants using 360 to 400 mm-long concrete prisms. Two of these pails were to be used in each one of the mixtures 1 and 3 and one pail for mixture 2 (Table 1). Each plastic pail was lined with an absorbant terry-cloth material in order to assure proper humidity conditions in the storage container during the test. Each participant also received a roll of 180-mm wide polyethylene tubing and sheets of an absorbant wiper to be used for the new proposed storage condition B (Table 1).

The participants were asked to store the concrete prisms in the storage conditions described in Table 1. In addition, four participants made an extra set of test prisms from each one of the mixtures 1 and 3, and stored them in a 1N NaOH solution at 38°C.

Testing of specimens

The changes in length of the concrete prisms were determined at various ages and the data reported by the participants to CANMET.

Various types of measuring devices were used by the participants in this study. These may be grouped as follows:

a) Prisms in a vertical position during measurement:
  - Mortar bar type measuring device using a dial gauge or a micrometer.
  - Swenson type measuring device using a micrometer (CSA A23.2-14A M94).

b) Prisms in an horizontal position during measurement.

AAR TEST RESULTS

Effect of the Storage conditions on the expansion test results

Figure 1 compares the curves of between-laboratory average expansion values versus time for the various series of test prisms from concrete mixtures 1 to 3, and subjected to the different storage conditions investigated. Table 2 gives the between-laboratory average 52- and 104-week expansion values for each one of the above mixtures.

Table 2: Summary of the 52- and 104-week between-laboratory expansion data with associated statistical information

Mix no.	Storage Conditions	52 weeks			104 weeks		
		Average	Std. Dev.*	C. of V.	Average	Std. Dev.*	C. of V.
1	A: Plastic pails (control)	0.17006	0.02077	12.2	0.19346	0.02516	13.0
	B: Plastic sleeves in pails	0.15013	0.02086	13.9	0.16746	0.02686	16.0
	C: Other containers	0.16582	0.02736	16.5	0.18948	0.03469	18.3
2	B: Plastic sleeves in pails	0.16196	0.03915	24.2	0.17612	0.05072	28.8
	C: Other containers	0.17643	0.04518	25.6	0.19524	0.05216	26.7
3	A: Plastic pails (control)	0.09333	0.03698	39.6	0.13906	0.06741	48.5
	B: Plastic sleeves in pails	0.08681	0.03317	38.2	0.11860	0.05571	47.0
	C: Other containers	0.09054	0.05293	58.5	0.12583	0.06342	50.4

* Std. Dev.: Reproducibility Standard Deviation ("S_R" as defined in ASTM E691). This value takes into account the repeatability (or within- laboratory) standard deviation ("S_r" in ASTM E691).

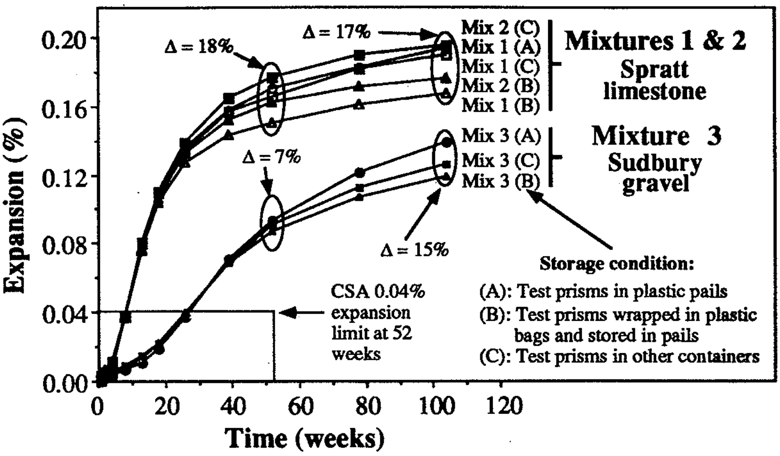

Fig. 1: Between-laboratory average expansion curves for series of test prisms cast from concrete mixtures 1 & 2 (Spratt limestone) and mixture 3 (Sudbury gravel).

Highly-reactive Spratt limestone (Mixtures 1 and 2)

- Between-laboratory expansion curves for the test prisms incorporating the Spratt limestone are all characterized by a rapid onset of expansion followed by the main expansion phase between the 4- and 18-weeks (Fig. 1). During this period, the average expansion values were basically the same for all the test prisms investigated. This is followed by a reduction in the expansion rate causing the expansion curve to level off progressively up to the end of the testing program.
- The differences between the highest and the lowest between-laboratory average expansion values obtained for the series of test prisms cast from concrete mixtures 1 and 2 and subjected to the various storage conditions investigated were about 18% after one year and about 17% after two years (Fig. 1); however, the differences between the individual test results obtained by the participants showed much greater variation. For example, the difference between the individual expansion values obtained for companion test prisms cast from mixture 1 and subjected to storage conditions A and B ranged from -6% (i.e. expansion in storage condition A being 6% less than the expansion in storage condition B) to about +34% at one year (Fig. 2A); the difference ranged from about -14% to + 32% for companion test prisms subjected storage conditions A and C (Fournier *et al.* 1995). For mixture 2, the difference in expansion for companion test prisms subjected to storage conditions B and C ranged from "-29%" to about "+23%" at one year (Fournier *et al.* 1995).

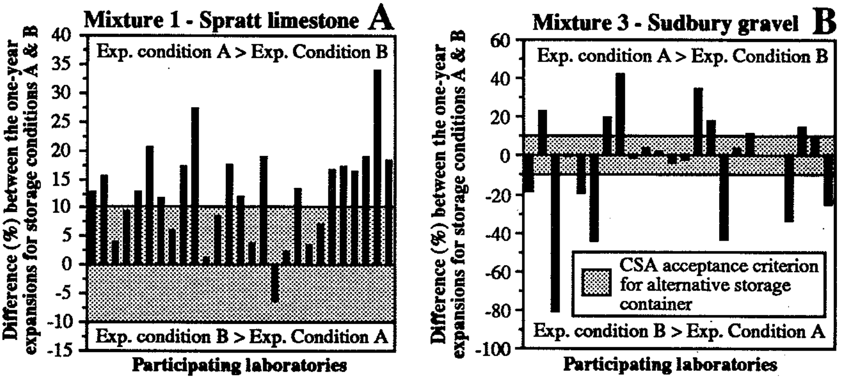

Fig. 2: Difference between the one-year expansion values obtained for companion series of test prisms cast from mixtures 1 and 3 and subjected to the various storage conditions investigated

- According to CSA A23.2-14A-M94, it is possible to use an alternative storage container to that specified in the standard (Condition A of this study); however, in order to be acceptable for use, the following two conditions must be specified: (1), the efficiency of the alternative must be evaluated with a standard alkali-reactive aggregate, and (2), the expansion at 52 weeks must be within 10% of that obtained using the specified container. The difference between the 52-week expansion values obtained for the companion series of test prisms cast from concrete mixture 1 and tested under conditions A and B was > 10% in 63% of the cases (Fig. 2A); the difference was > 10% in 27% of the cases for prisms tested under conditions A and C. For concrete mixture 2 and when using condition B as the control, the difference in expansion was > 10% in 55% of the cases for test prisms tested under conditions B and C (Fournier *et al.* 1995).

Moderately-reactive Sudbury gravel (mixture 3)

- Between-laboratory expansion curves for the test prisms incorporating the Sudbury gravel are all characterized by a relatively slow onset of expansion up to about 8 weeks, and then increases at a fairly constant rate up to 39 weeks. This is followed by a slow down in the expansion rate with the change in the slope being relatively more evident after the end of the first year (Fig. 1).
- The between-laboratory average expansion values obtained for the different series of concrete prisms subjected to the three storage conditions investigated ranged from about 0.087 to 0.093% at one year and from 0.119 to 0.139% at two years (Table 2). The test prisms placed directly in the plastic pails showed average expansion values of more than 7% at one year and 15% at two years than those stored in the plastic sleeves (Table 2). The differences between the individual expansion values obtained by the participants showed a wide variation. For example, the difference in expansion for companion test prisms subjected to storage conditions A and B ranged from "-81%" to about "+42%" at one year (Fig. 2B).
- As mentioned before, the criterion for an alternative storage condition to be acceptable for use in the Concrete Prism Test is that the expansion at one year with the proposed condition should be within 10% of that obtained using the specified container. The difference between the 52-week expansion values obtained for the companion series of test prisms tested under conditions A and B was > 10% in 58% of the cases (Fig. 2B).

As illustrated on Figs. 3A and 3B, for companion series of test prisms cast from the same mixture but tested under different storage conditions, the participants were generally consistent in getting systematically high or low expansion values compared with the between-laboratory average.

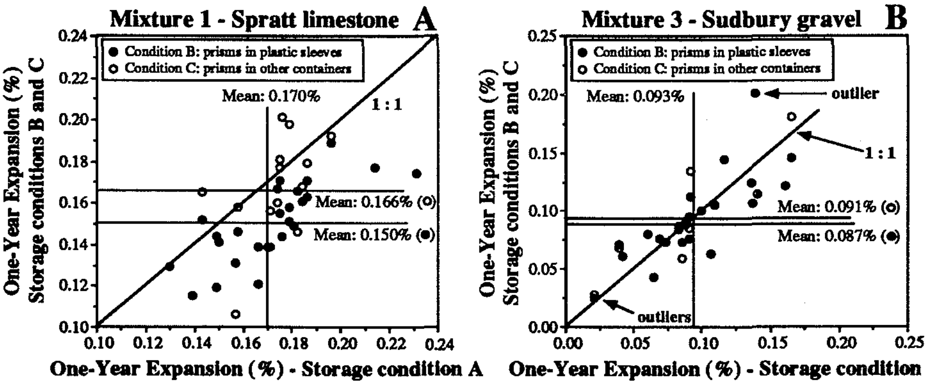

Fig. 3: Comparison between the expansion values of companion series of test prisms cast from concrete mixture 1 (A) and 3 (B) but subjected to different storage conditions

Between-Laboratory Variability of Test Results

Highly-reactive Spratt limestone (Mixtures 1 and 2)
For all the three storage conditions investigated, the between-laboratory C.V. for the various series of test prisms cast from mixtures 1 and 2 were found to decrease rapidly with time and become generally stable after about 18 to 26 weeks of testing (Fig. 4A). The best value of between-laboratory C.V. of 12.2% (at 52 weeks) was obtained for the series of test prisms cast from concrete mixture 1 and tested under storage condition A; the between-laboratory C.V. for the test prisms cast from concrete mixture 2 and tested under storage condition B increased to 24.2% (Table 2).

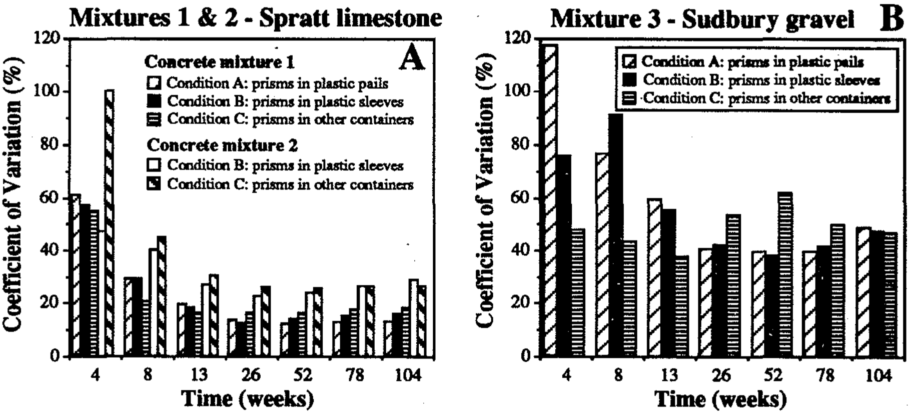

Fig. 4: Plot of the coefficient of variation against time for series of test prisms cast from concrete mixtures 1 and 2 (A), and 3 (B) and subjected to different storage conditions

Moderately-Reactive Sudbury Gravel (concrete mixture 3)
For all the three storage conditions investigated, the between-laboratory C.V. was found to decrease slowly but progressively with time, but remaining fairly variable up to the end of the two-year testing period (Fig. 4B). The lowest value of the C.V. of 38.2% at one year, was obtained for the test prisms subjected to the storage condition B, while the highest C.V. of 58.5% at one year was obtained for the test prisms subjected to the storage conditions C (Table 2).

Discussion on the between-laboratory variability of test results
The expansion results obtained by the participants for the series of test prisms cast from concrete mixture 1 suggest that a reasonnably low between-laboratory variation can be achieved by using a number of well-controlled testing conditions, i.e. a reference sand, a reference cement, a standard storage container and a fixed concrete mixture proportion. The between-laboratory variability then depends primarily on the process of concrete mixing, specimen preparation, the reliability of the 38°C storage facilities, of the measuring device and of the personnel in charge of taking the measurements. In fact, the value of the between-laboratory C.V. of about 12.2% obtained for the above series of test prisms is close to the 11.3% for the 28-day compressive strenght values reported by the participants for mixture 1 (Fournier *et al.* 1995). This suggests that the between-laboratory variability due to the above parameters is not unduly large.

The much higher between-laboratory values of the Reproducibility S.D. (SR according to ASTM E691) and the C.V. obtained for mixtures 2 and 3 of the program suggest that an additional and important part of the variability of the test method is related to the permissible range of mixture proportioning parameters allowed in the standard procedure, e.g. the use of local sources of cement and fine aggregates, and the permissible range in the w/c. The use of different storage containers also influences the expansion test results, but to a limited extent. As can be seen in Table 2, a between-laboratory C.V. of 13.9% was obtained for the series of test prisms cast from mixture 1 and stored in plastic sleeves. The use of different storage containers for the companion series of test prisms cast from the same mixture resulted in an increase in the C.V. of about 4% (vs condition A). A small difference was observed for the C.V. obtained for the companion series of test prisms cast from mixture 2, i.e. 24.2% for test prisms under storage condition B versus 25.6% for test prisms under storage condition C.

Effect of Other Parameters on the Expansion Test Results

Cement composition

As mentioned before, participants were asked to make concrete mixtures 2 and 3 using a local normal portland cement having an alkali content of 0.90 ± 0.10%. The total alkali content of the concrete mixture was then to be raised at 1.25% Na_2O equivalent by adding NaOH to the mixture water.

Fourty-six per cent of the participants have used a cement with an alkali content falling outside of the range prescribed in the standard test procedure; however, as can be seen in Fig. 5A, the alkali content of the cement used by the participants for the above mixtures did not seem to have a significant influence on the expansion values obtained in the concrete prism test.

Reactivity of the fine aggregate

The participants in this study were also asked to make concrete mixtures 2 and 3 using a fine aggregate from a local source. This sand was to be non-reactive, producing an expansion < 0.10% at 14 days in the Accelerated Mortar Bar (AMB) Test (CSA A23.2-25A, ASTM C 1260). Six out of the twenty-four sands used by the participants gave AMB expansions higher than the above limit; however, as can be seen in Fig. 5B, no correlation was observed between the one-year expansion values of concrete prisms cast from mixtures 2 and 3 (storage condition B) and the 14-day AMB expansion values obtained for the fine aggregates used by the participants for the above mixtures.

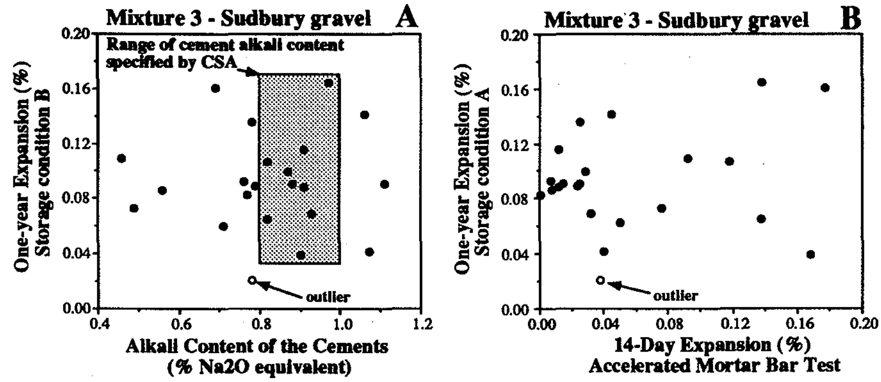

Fig. 5: Effect of the alkali content of the cement (A) and of the reactivity of the fine aggregate (B) used by the participants on the expansion values measured in the concrete prism test.

Alternative testing condition

Figure 6A shows the average expansion values plotted against time for the different series of test prisms cast from concrete mixture 1 and subjected to the various storage conditions investigated. At one year, the test prisms stored in the 1N NaOH and those subjected to storage condition A showed similar expansions of 0.180%; however, at two years, the test prisms in the NaOH solution showed 15% more expansion than those subjected to the storage condition A (Table 3).

The immersion testing showed some interesting results for the moderately reactive Sudbury gravel. Test prisms immersed in the 1N NaOH showed a more rapid onset of expansion than the companion series of test prisms subjected to storage conditions A and B (Fig. 6B). The test prisms immersed in the 1N NaOH solution showed 26 and 58% more expansion at the end of the one- and two-year periods, respectively, than the prisms subjected to control storage condition A (Table 3).

For both aggregates, the use of the immersion testing in the NaOH solution resulted in significant reductions in the values of the S.D. and the C.V. compared with the values obtained for the companion series of test prisms subjected to storage conditions A and B (Table 3).

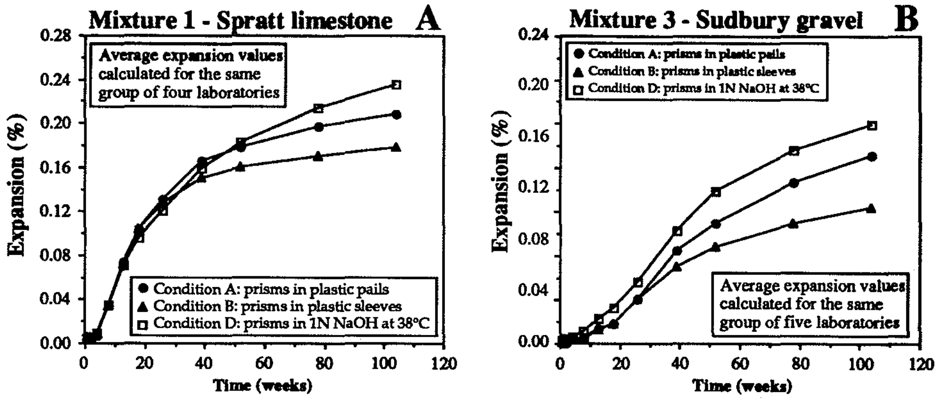

Fig. 6: Multi-laboratory average expansion curves for companion series of test prisms cast from concrete mixture 1 (A) and mixture 3 (B) and subjected to the different storage conditions.

Table 26: 52-week and 104-week Expansion Data with Associated Statistical Information for the Same Group of Laboratories that Have Performed the Immersion Testing in 1N NaOH at 38°C

Mix no.	Storage Conditions	Data at 52 weeks			Data at 104 weeks		
		Average	Std. Dev.	C. of V.	Average	Std. Dev.	C. of V.
1	A: Plastic pails	0.17808	0.03649	20.5	0.20742	0.04688	22.6
	B: Plastic sleeves in pails	0.15933	0.02970	18.6	0.17750	0.03348	18.9
	D: 1N NaOH at 38°C	0.18200	0.02549	14.0	0.23533	0.03189	13.6
3	A: Plastic pails	0.11047	0.04332	39.2	0.17147	0.10571	61.7
	B: Plastic sleeves in pails	0.08820	0.04394	49.8	0.12407	0.07234	58.3
	D: 1N NaOH at 38°C	0.13747	0.03349	24.4	0.19953	0.04358	21.8

CONCLUSION

The results obtained in this study showed that the use of well-controlled testing conditions or parameters in the concrete prism test procedure, such as a reference sand, a reference cement, a standard storage container and fixed concrete mixture proportioning, can greatly contribute in reducing the between-laboratory variability of the test results. Parameters such as the cement composition (alkali and the SO_3 contents), the potential alkali-reactivity of the fine aggregate and the size of the test prisms did not seem to have a significant influence on the test results when taken individually; however, when taken all together, these parameters probably contribute in increasing the between-laboratory variability of the test method.

For both aggregates investigated, the between-laboratory standard deviation and the coefficient of variation were significantly lower for the test prisms in the NaOH solution than for those subjected to control storage condition. This method appears particularly promising for testing marginally to moderately reactive aggregates.

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ABNORMAL EXPANSION OF COARSE-GRAINED CALCITE IN THE AUTOCLAVE METHOD

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ABSTRACT

Abnormal expansion of coarse-grained calcite at high temperature (150 ℃) was investigated in this research. Results show that coarse-grained calcite can undergo heat-induced expansion and the expansion value is proportional to the aggregate size. The heat-induced expansion has a negligible effect on the autoclave method identifying of ASR reactivity of aggregates. It is suggested that the expansion of carbonate aggregates in the autoclave method be appraised with the help of other methods, such as petrography.

Key words: Coarse-Grained Calcite, Alkali Reactivity, Autoclave Method

INTRODUCTION

Alkali-aggregate reaction (AAR) includes alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). Identification of the alkali reactivity of aggregates is important to prevent AAR and to determine whether AAR is the cause of concrete degradation. In ASR research, Ranc et al concluded that ASTM methods C227 and C289 were not reliable in detecting six deleteriously expansive coarse aggregates (Ranc et al.1994). Tang et al proposed an autoclave method for the identification of ASR reactive aggregates, which adopts a high temperature (150 ℃) and a high alkali content (1.5% Na_2O in specimens and autoclaved in 10% KOH solution) (Tang et al.1983). Inter-laboratory research in France has proved that the microbar method (P 18-588) which adopts 150 ℃ × 6h cure and the autoclave test (P 18-589) which adopts 127 ℃ × 18h cure lead to similar conclusions as to the reactivity or non-reactivity of most of the tested aggregates (Criaud et al.1994).

The most commonly adopted methods in the research of alkali carbonate reaction are ASTM C 1105 and CSA A 23.3-14A, which take a long time and do not detect late-expansive aggregates. It is necessary to develop rapid and effective methods. Based on autoclave method to detect ASR reactivity of aggregates, Tang changed the aggregate size and applied the method to determine the alkali reactivity of carbonate aggregates (Tang et al.1994). Preliminary research has proved that the method can distinguish between safe and harmful carbonate aggregates (Tong 1994).

In the case of ASR, the elevation of temperature only accelerates the reaction process and the corresponding expansion and thus makes it possible to obtain the final result in a shorter period. Therefore, high temperature has been employed in rapid methods to detect the ASR reactivity of aggregates. However, in the case of ACR, both alkali-dolomite reaction and alkali-calcite reaction are promoted at high temperatures in KOH solution. It is not difficult to understand that the rate of ion diffusion and product formation will be accelerated by the elevation of temperature in view of the dynamics. It is also noted that the ion product constant of water K_{H2O} changes at different temperatures, e.g., $K_{H2O,\ 298K}=10^{-14}$, $K_{H2O,\ 348K}=10^{-12.72}$ and $K_{H2O,\ 423K}=10^{-11.64}$. The ion product of water is related to H^+ and OH^- concentration in the solution. Therefore, the alkali-calcite reaction accelerates as the temperature rises. In recent research, it was found that coarse-grained calcite resulted in abnormal expansion of specimens at high temperature (Mu et al.1995). Therefore, attention was given to factors affecting the expansion behaviour and their possible effect on autoclave methods.

EXPERIMENTAL

Three kinds of calcite rocks were used in the research. L1 and L2 are two fine-grained calcite rocks from Nanjing. TS is coarse-grained calcite rock from Tangshan, Nanjing. To understand the performance of coarse-grained calcite at high temperatures, both in the rapid method for ASR and in the rapid method for ACR, TS aggregates were crushed to various groups of different sizes, from 0.15 to 10 mm. Parameters adopted in the experiments were the same as in the literature.

Table 1 Calcite aggregate sizes adopted in experiment

Name	TS1	TS2	TS3	TS4	TS5
Size range (mm)	5.0-10.0	2.5-5.0	1.25-2.5	0.80-1.25	0.15-0.80

RESULTS AND DISCUSSIONS

It is known that limestone is relatively inert aggregate which is widely used in concrete construction. In the present research, TS1, L1 and L2 were used as aggregates in the concrete microbar test to compare their performance at high temperatures.

It is shown in Fig. 1 that TS can result in expansion well above that of L1 and L2 in the concrete microbar test. The expansion value of 0.08% for TS1 after 6h is close to the threshold value (0.1%) for alkali-reactive carbonate aggregate in the concrete microbar test. To understand the effect of heat treatment on the abnormal expansion by TS, TS1 was used as an aggregate in the concrete microbar test with the same total autoclave time (60h) and various time periods (15h, 20h, 30h, 60h). Various heat treatment procedures thus can be expressed in terms of autoclave steps e.g. 1, 2, 3, 4 steps with a fixed toatl autoclave time. The are shown in Fig. 2.

Fig. 2 shows that the heat treatment procedure has a profound effect on the expansion behaviour of TS1 aggregate in concrete microbar test. With equal total autoclave time, the more the steps of a heat treatment procedure, the higher the expansion value. Since the total autoclave time is fixed at 60h, the chemical reaction involved can be regarded as the same. Therefore, the differences in expansion result from the different approaches of autoclave. In a control experiment with the same procedure, alkali-reactive aggregate from Canada (CK) was employed. The result is shown in Fig. 3. From Fig. 2 and Fig. 3, it is appropriate to consider the abnormal expansion caused by coarse-grained calcite as heat-induced expansion.

A series of experiments were conducted to evaluate the effect of aggregate size. The results under various conditions are shown in Fig. 4, Fig. 5 and Fig. 6. It is evident that the expansion caused by TS aggregate is closely related to the aggregate size. The expansion caused by TS aggregate is similar when autoclaved in alkali solution, water and vapour respectively. The larger the aggregate size, the higher the expansion value.

When cured in alkali solution and water at identical grain sizes, the average expansion values are wholly higher than those in vapour. The possible reason is that alkali solution and water curing can make it easier for the chemical reaction to proceed. The expansion values in alkali solution and water are very close, which may suggest that water curing is perhaps sufficient for the chemical reaction and external alkali provision has little effect on the expansion. This phenomenon may suggest that the alkali ions play different roles here compared with their roles in the expansion process resulting from alkali-silica reaction. In previous research, XRD proved that $Ca(OH)_2$ formed in the cleavages of calcite aggregates, which facilitated the expansion. To determine the effect of chemical reaction on expansion, an experiment was carried out to determine the expansion of TS in different alkali environments in the concrete microbar method.

Table 2 Expansion of specimens containing TS1 aggregate in various alkali environments

Samples	TS1	TS1P*	TS1W**
Alkali content ($Na_2O_{equiv.}$) in specimens (%)	1.5	0.46	0.23
Immersion solution	10% KOH a.q.	H_2O	H_2O
Expansion after 6h (%)	0.09	0.087	0.082

* *TS1P: Ordinary portland cement was used.*

** *TS1W: White cement with low alkali content was used.*

Table 2 shows that alkali-calcite reaction has a limited effect on the specimen expansion. This is in accordance with the conclusion on heat-induced expansion above.

It is noted that the difference between the rapid mortar bar method for ASR and the concrete microbar rapid method for ACR is in the aggregate sizes adopted, with the former of 0.15-0.80 mm and the latter of 5.0-10.0 mm. Experiments were done to compare the expansion behaviour of calcite aggregate in both autoclave methods. The results in Table 3 show that the mortar bar method for detecting ASR reactivity is not affected by the abnormal heat induced expansion of coarse-grained calcite.

Table 3 Expansion of TS calcite aggregate in autoclave methods

Name of autoclave method	Concrete microbar method	Mortar bar method
Expansion after 6 h (%)	0.091	0.015

CONCLUSION

The abnormal expansion of coarse-grained calcite aggregate in the concrete microbar test is mainly heat induced and is dependent on the particle size. Expansion resulting from alkali-calcite reaction at high temperatures is very limited, and will not contribute to the alkali reactivity of carbonate aggregates in the concrete microbar test. To get a complete understanding of the expansion of carbonate aggregates in the rapid methods, petrographic examination or control tests with very low alkali content are helpful.

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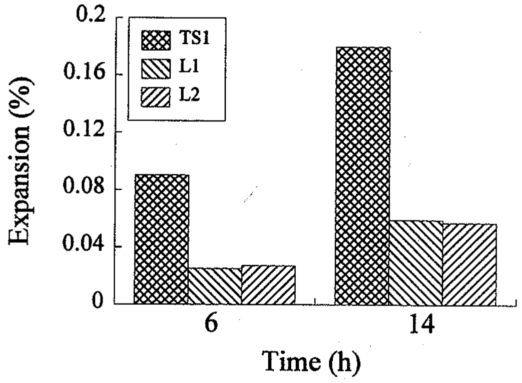

Fig. 1 Comparison of expansion behaviour of different calcite aggregates

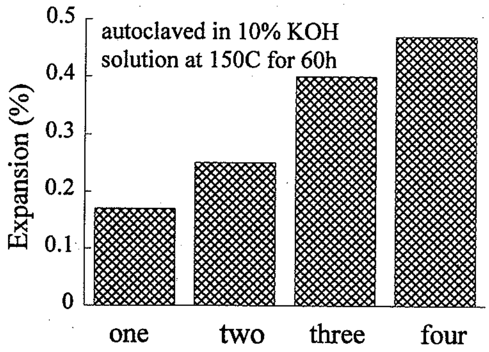

Fig. 2 Effect of various heat treatment procedures on expansion behaviour of TS1 aggregate

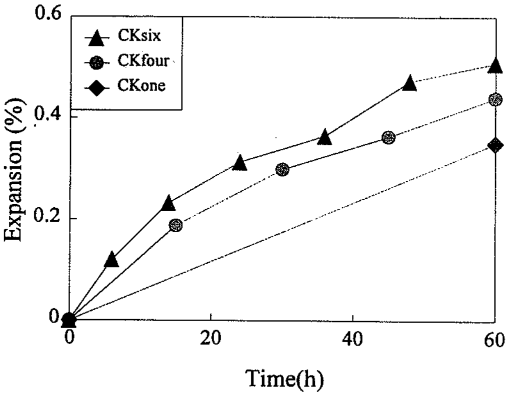

Fig. 3 Effect of various heat treatment procedures on expansion of CK aggregate

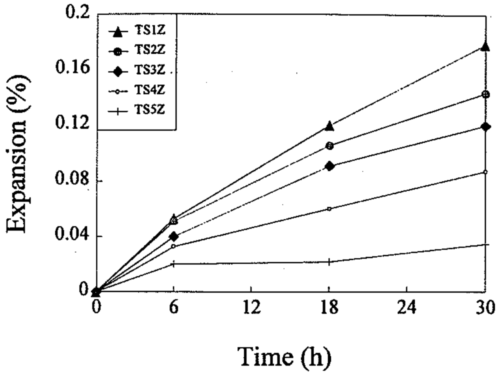

Fig. 4 Expansion of concrete microbars containing TS aggregates cured at 150 ℃ in vapour

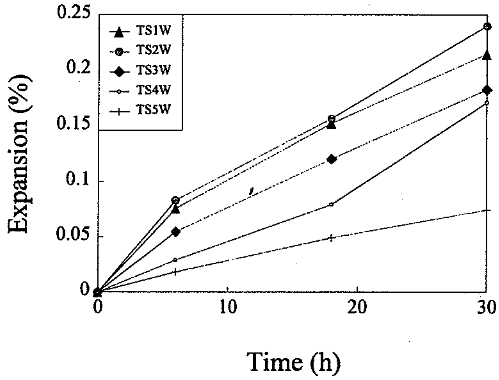

Fig. 5 Expansion of concrete microbars containing TS aggregates cured at 150 ℃ in water

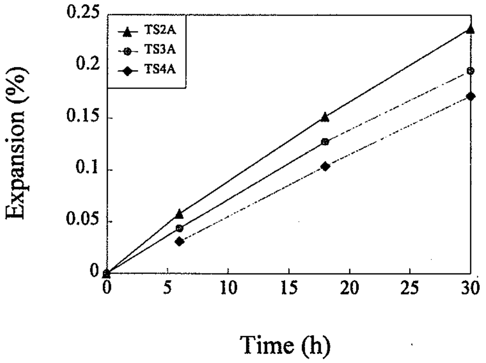

Fig. 6 Expansion of concrete microbars containing TS aggregates cured at 150 ℃ in 10% KOH Solution

LONG TERM MOVEMENTS DUE TO ALKALI-SILICA REACTION AND THEIR PREDICTION

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ABSTRACT

Expansion test results are reported for concretes made with a range of original alkali contents, 3 to 8 kg/m^3, and prepared using a number of U.K. siliceous aggregates, all but one of which contained chert and flint. The concretes were subject to three exposure regimes: moist storage at 38 °C, moist storage at 20 °C and external exposure in the South East of the UK. The duration of exposure for the concrete prisms stored at 20 °C and the concrete blocks stored externally is approaching 7 years. Expansion test results are also reported for concrete cores taken, at an age of between 36 and 55 months, from a number of the blocks, both cracked and uncracked, and then stored moist at either 20 or 38 °C. The prediction, from accelerated tests on laboratory stored concrete prisms and cores, of the unrestrained expansion-age relationships for externally exposed concretes is briefly discussed.

Keywords: Alkali-silica reaction, expansion, core testing.

INTRODUCTION

A previous paper (Hobbs, 1992), presented expansion data obtained up to an age of 36 months on a range of unreinforced concrete blocks stored externally, together with expansion data obtained on concrete prisms stored moist at 20°C and 38°C. The concretes had alkali contents in the range 3 to 8 kg/m^3 and were prepared using a number of UK siliceous aggregates. The monitoring of these samples has continued and the present paper includes expansion data obtained on these concretes up to an age of 84 months, together with expansion data obtained on cores taken from a number of the concrete blocks which were expanding due to ASR. These latter tests were carried out to check whether the future unrestrained expansion of concrete could be predicted from core expansion tests.

EXPERIMENTAL DETAILS

The cement used was a Portland cement with an alkali content expressed as equivalent sodium oxide content of 1.00 per cent by mass. The 14 sands and coarse aggregates used in the programme are listed in Table 1. Further details relating to the materials employed are given in the previous paper (Hobbs, 1992). Prisms, 75 x 75 x 250 mm in size, and blocks, 200 x 250 x 500 mm in size, were prepared from most mixes with alkali contents in the range 3 to 8 kg/m^3. The prisms were stored moist at 20 °C or 38 °C and the concrete blocks were stored externally in the South East of the UK. Periodically the expansion of the prisms and blocks was monitored. Further details relating to the concretes tested and the storage conditions to which they were subjected are given in the previous paper (Hobbs, 1992)..

Table 1: Location of the aggregates

	Aggregate
1.	A Trent Valley sand
2.	A Trent Valley coarse aggregate, from the same source as 1
3.	A siliceous sand, South West England
4.	A coarse siliceous aggregate, South West, from the same source as 3
5.	A Mendip limestone, crushed fines
6.	A Mendip limestone, crushed coarse, from the same source as 5
7.	A siliceous sand, Southern England
8.	A Thames Valley sand
9.	A sea dredged sand, Southern England
10.	A crushed quartzite coarse aggregate, South West
11.	A sharp siliceous sand, Southern England
12.	A Thames Valley coarse aggregate, 20-10 mm
13.	A Thames Valley coarse aggregate, 10-5 mm
14.	A Thames Valley sand, from the same source as 13 and 12.

At an age of between 36 to 55 months, cores, 75 mm diameter and 250 mm in length, were taken from a number of the concrete blocks stored externally. Sets of demec studs were fixed to each core to provide Demec gauge lengths in each of three rows at 120° to one another around the core. The cores were then covered to minimize moisture loss and stored in a laboratory maintained at 20 ± 1 °C. The gauge length used was 100 mm. The first demec gauge length measurements were made within 24 hours after the cores had been extracted. Each core was wrapped in a moist towel surrounded by a plastic sleeve and a small section of the sleeve and towel was removed to expose each demec stud. The wrapped prism was then placed in a plastic bag and stored in a sealed container in a room maintained at 20 ± 1 or 38 ± 1 °C. Periodically additional water was added to maintain the towel in a wet condition and the six gauge lengths on each core measured. Measurements were completed as quickly as possible and the wrapped cores replaced into their closed containers, which were then returned to the storage rooms.

RESULTS: PRISMS AND BLOCKS

Ages to abnormal expansion

The approximate ages at which abnormal expansion (~0.05%) commenced for each exposure condition are given in Table 2. No concretes with alkali contents below 4.8 kg/m^3 have exhibited abnormal expansion. For storage at 38 °C, the age to abnormal expansion ranged from 2 to 12 months, for storage at 20 °C from 8 to 45 months and for external storage from 18 to 60 months. An examination of Table 2 shows that the ages at which abnormal expansion commenced for concretes stored moist at 38 °C, stored moist at 20°C and exposed externally are approximately in the ratio 1:4:7 or 1:4:8 respectively in agreement with previously published work (Hobbs, 1993).

Expansion

The three most deleterious aggregate combinations were (3,6), (8,6) and (11,6). The expansion results obtained on concretes, stored at 20 °C, containing two of these aggregate combinations are shown plotted in Figs. 1 and 2. On the basis of these results it can be concluded that, for the particular aggregate combinations tested in this study, an original alkali content of about 4.8 kg/m^3 is necessary for deleterious expansion to result from ASR.

The expansions of the concretes at ages of 9 months for concretes stored at 38 °C, 3 years for concretes stored at 20 °C and the expansion of the top face of the blocks at 5 years for concretes stored externally are compared in Table 2 (i.e. at ages in the ratio 1:4:6.7). The expansions of each concrete are broadly similar, indicating that the alkali-silica reaction rates for concrete subject to the three exposure conditions, are approximately in the ratio 7:4:1. This is illustrated further in Fig. 3 to 5 where

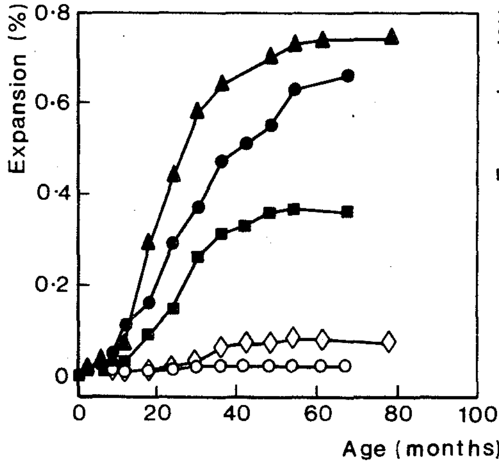


Fig. 1 Relationship between expansion and age. 20 °C. Aggregate (3,6).

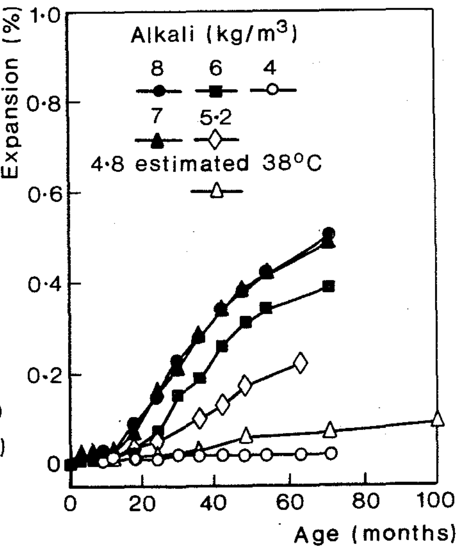


Fig. 2 Relationship between expansion and age. 20 °C. Aggregate (8, 6).

Table 2: Expansions(%) at ages of 9 months (38°C prisms), 3 years (20°C prisms) and 5 years (external blocks) together with the approximate ages (months) to abnormal expansion. (uc = uncracked)

Aggregate combination		Alkali content (kg/m^3) and exposure 'temperature'												
		4.8	5.2			6.0			7.0			8.0		
Sand	Coarse	38°C	38°C	20°C	Ext.	38°C	20°C	Ext.	38°C	20°C	Ext.	38°C	20°C	Ext.
1	2	-	0.01 (uc)	0.02 (uc)	0.02 (uc)	0.09 (7)	0.15 (23)	0.09 (52)	0.16 (3.5)	0.20 (18)	0.13 (42)	0.20 (3)	0.21 (12)	0.13 (26)
1	6	-	0.02 (uc)	0.03 (µc)	0.01 (uc)	0.03 (12)	0.05 (36)	0.08 (60)	0.06 (7.5)	0.09 (18)	0.05 (56)	0.06 (7.5)	0.08 (18)	0.09 (30)
3	4	-	0.01 (uc)	0.02 (uc)	-0.01 (uc)	-		-0.03 (uc)	0.02 (uc)	0.03 (uc)	0.03 (uc)	-	-	0.03 (uc)
3	6	-	0.01 (uc)	0.06 (30)	0.11 (54)	0.21 (6)	0.31 (14)	0.46 (18)	0.50 (2)	0.64 (10)	0.57 (18)	0.28 (2.5)	0.47 (8)	0.59 (18)
5	2	-	0.01 (uc)	0.04 (45)	0.01 (uc)	0.03 (11.5)	0.06 (30)	0.06 (60)	0.09 (4.5)	0.13 (19)	0.13 (50)	0.03 (22)	0.14 (12)	0.14 (41)
5	4	-	-	-	-	-	-	-	0.02 (uc)	0.05 (uc)	-	-	-	-
5	6	-	-	-	-	-	-	-	0.02 (uc)	0.04 (uc)	0.02 (uc)	-	-	-
5	10	-	0.02 (uc)	0.04 (uc)	0.00 (uc)	0.04 (12)	0.05 (30)	0.01 (uc)	0.04 (uc)	0.07 (18)	0.02 (uc)	0.04 (10)	0.08 (10)	0.03 (uc)
7	6	-	0.02 (uc)	0.03 (uc)	0.01 (uc)	-	-	-	0.04 (uc)	0.06 (26)	0.03 (40)	-	-	-
8	6	0.03 (12)	0.09 (7)	0.10 (26)	0.05 (59)	0.26 (4.5)	0.19 (21)	0.26 (45)	0.16 (4)	0.28 (15)	0.36 (28)	0.32 (2)	0.28 (13)	0.32 (28)
9	6	0.03 (uc at 26)	0.04 (10)	0.04 (uc)	0.02 (uc)	0.17 (3.5)	0.17 (17.5)	0.16 (44)	0.23 (3.5)	0.25 (13)	0.23 (30)	0.24 (3)	0.24 (12)	0.24 (36)
11	6	-	0.02 (uc)	0.04 (uc)	0.01 (uc)	0.09 (7)	0.26 (18)	0.29 (41?)	0.30 (3)	0.43 (12)	0.34 (24)	0.39 (3)	0.48 (11.5)	0.39 (24)
14	12,13	-	0.02 (uc)	0.03 (uc)	-	0.02 (uc)	0.03 (uc)	-	0.02 (uc)	0.06 (34)	-	0.04 (uc)	0.16 (21)	-

expansion is shown plotted against normalized age for a number of the concretes prepared using aggregate combinations (3,6), (9,6) and (1,2), the assumption being made that the reaction rates, for the three exposure conditions, are in the ratio 7:4:1. In one instance the agreement is very poor (see Fig. 3). This poor agreement occurred at an alkali content of 5.2 kg/m^3 and is due in part to the high expansion occurring close to the trowelled face of the exposed end face of the block. If this expansion is excluded, then the mean expansion at 75 months falls from 0.36 to 0.24%. From Figs. 3 to 5 it can be seen or deduced that for concretes stored at 38 °C, 20 °C and externally, the ages at which expansion is largely complete are probably 9 to 15 months, 4 to 7 years and 8 to 15 years respectively.

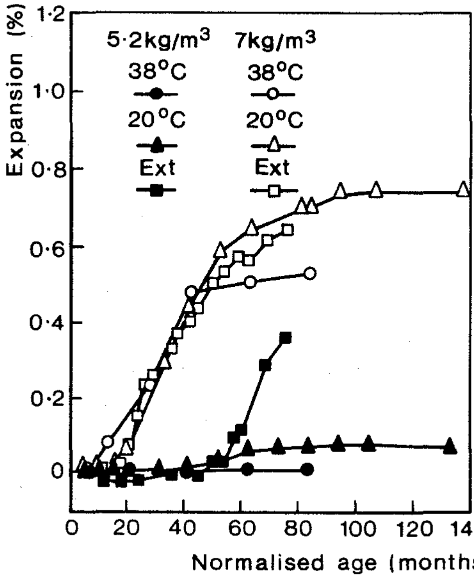


Fig. 3 Relationship between expansion and normalized age. Aggregate (3, 6).

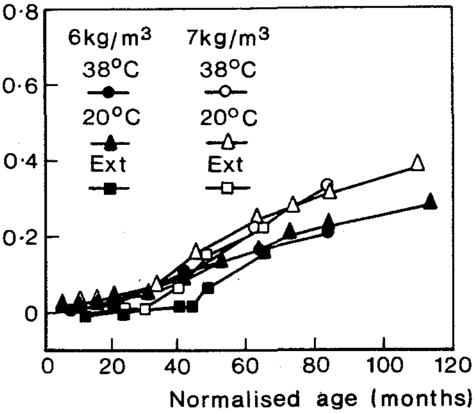
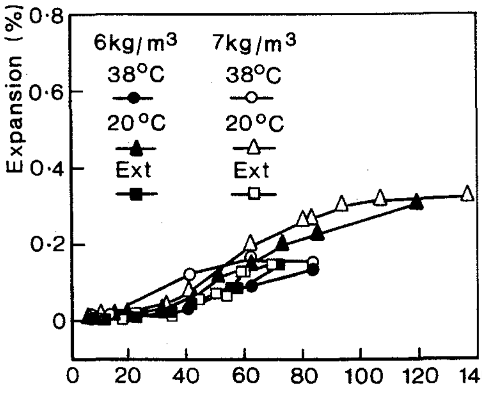


Fig. 4 Relationship between expansion and normalized age. Aggregate (9, 6).



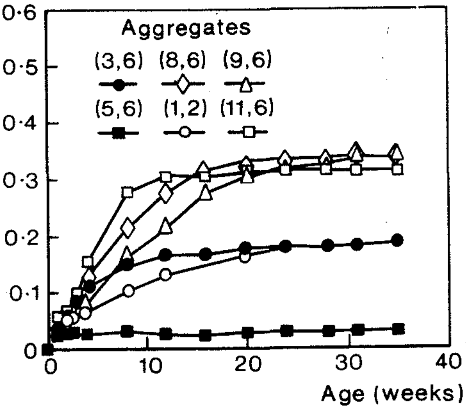


Fig. 6 Comparison of core expansions. 38 °C. 7 kg/m^3.

Fig. 5 Relationship between expansion and normalized age. Aggregate (1, 2).

RESULTS: CORES

The mean expansion results obtained on cores taken from a number of the unreinforced blocks are shown plotted against test age in Figs. 6 to 8. In the case of the cores stored at 38 °C, no correction has been made for the thermal expansion induced by raising the core temperature from 20 to 38 °C (possibly 200 to 300 micro-strain). Monitoring of the cores stored at 20 °C and the blocks is continuing. An examination of Figs. 6 to 8 leads to the following observations:

1. At 38 °C, expansion is largely complete after 20 to 30 weeks. (Figs. 6 and 8).
2. Similar expansion is observed after 20 to 30 weeks storage at 38 °C and 90 to 100 weeks storage at 20 °C (Figs. 6 and 7).

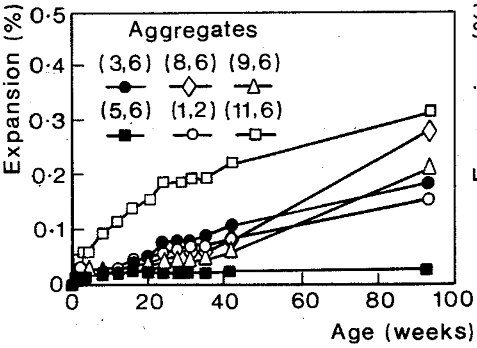


Fig. 7 Comparison of core expansions. 20 °C. 7 kg/m^3.

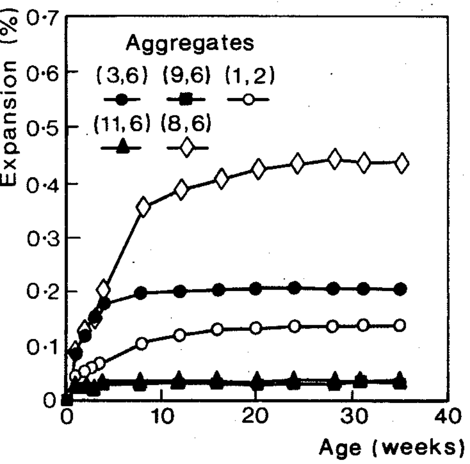


Fig. 8 Comparison of core expansions. 38 °C. 5.2 kg/m^3.

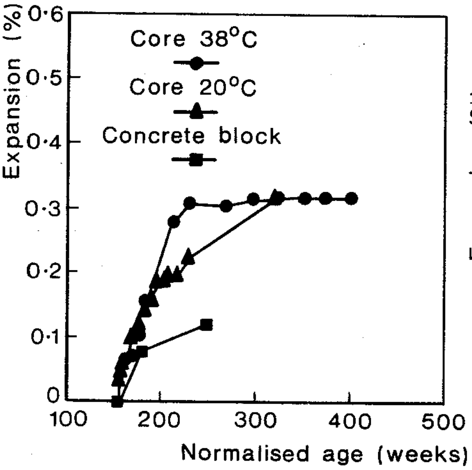


Fig. 9 Expansion predicted from core testing and actual expansion. Aggregate (11, 6). 7 kg/m^3.

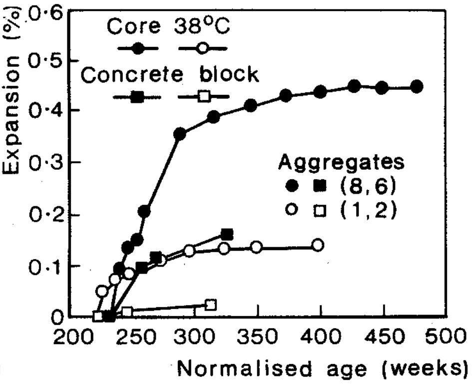


Fig. 10 Expansion predicted from core testing and actual expansion. 5.2 kg/m^3.

DISCUSSION

Assuming the rates of expansion for concretes stored moist at 38 °C, stored moist at 20 °C and exposed externally, are in the ratio 7:4:1, the test ages for the cores were normalized and the expansion compared with the expansion of the region of the block from which the cores were taken. Some of the comparisons are shown plotted in Figs. 9 and 10. It was found that the core expansions were substantially higher than the block expansions. In one instance the core exhibited abnormal expansion but the concrete block from which it was taken did not (Fig. 10). Two reasons may account for these disappointing results. Firstly, due to their higher surface area-volume ratio, the cores may have taken up more water than the blocks and secondly, the removal of the confining restraint may have allowed additional expansion to occur. This confining restraint is induced by the outer macro-cracked region of the blocks (Hobbs, 1988), and is removed when a core is taken.

When it was deduced that the normalized expansions were going to be substantially higher than those of the blocks from which they were taken, several blocks were selected for moist storage at 38 °C. Monitoring of these blocks has indicated substantially lower expansions than the relevant cores stored at 38 °C, indicating that removal of the confining restraint is probably the major factor resulting in enhanced expansion of the cores.

CONCLUSIONS

The following conclusions are applicable to the particular aggregate combinations tested and to the data obtained to date:

1. The approximate age at which abnormal expansion due to ASR is induced for concretes stored at 38 °C, 20 °C and externally, are approximately in the ratio 1:4:7 respectively.
2. After expansion has been induced, the expansion rates for concretes stored at 38 °C, 20 °C and externally, are generally approximately in the ratio 7:4:1 error respectively.
3. For concretes stored at 38 °C, 20 °C and externally, the ages at which expansion is largely complete are probably 9 to 15 months, 4 to 7 years and 8 to 15 years respectively.
4. The expansion monitored on cores stored moist at 38 °C and 20 °C can greatly over-estimate the unrestrained expansion of the concrete members from which they are taken.

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A COMPARISON OF DAMAGE RATING INDEX WITH LONG-TERM EXPANSION OF CONCRETE PRISMS DUE TO ALKALI-SILICA REACTION

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ABSTRACT

A damage rating system for quantifying the amount of distress in concrete structures due to alkali-silica reaction (ASR) was recently developed by the Institute for Research in Construction-National Research Council of Canada (IRC-NRC). The system involves identifying and counting the number of features characteristic of damage due to ASR in a prepared concrete sample. Such features are given a weighting factor to calculate the damage rating index (DRI). This system was applied to a number of test prisms especially made for the evaluation of alkali-aggregate reactivity (AAR) in concrete. The expansions of the prisms were measured periodically for up to 15 years under 23°C and 38°C laboratory conditions following appropriate CSA/ASTM/Ontario Hydro test procedures. A comparison of expansion against DRI demonstrated that no single correlation exists suggesting that the weighting factors used to calculate the DRI require review for evaluation of AAR in general. A multivariable statistical analysis was also conducted on these data in an attempt to determine whether a relationship exists between the linear expansion and the observed features of ASR. The statistical analysis indicates that a larger pool of data is required.
Keywords: alkali-aggregate reactivity (AAR), alkali-silica reaction (ASR), concrete distress, damage rating, damage rating index (DRI), expansion, statistical analysis

INTRODUCTION

In several regions of Canada, concrete deterioration due to alkali-aggregate reactivity (AAR), especially alkali-silica reaction (ASR) and less importantly alkali-carbonate reaction (ACR), has been diagnosed (Rogers & Worton, 1985; Grattan-Bellew, 1995a, 1995b; Thomas et al., 1992). Damage due to AAR has resulted in costly repair programs and, in some cases, structures have needed to be replaced.

To conduct a study of how to relate concrete deterioration to actual concrete expansion accompanying AAR, it is first necessary to select a damage rating system which takes into account all features which characterize damage due to AAR. A system must be chosen that assigns a damage rating index (DRI) to the concrete for the purpose of comparison. Several damage rating systems have been proposed such as the stiffness damage test by Crisp, Waldron & Wood (1993) and the thin section crack inventory method by Sims, Hunt & Miglio (1992) but there is no universally accepted method. The damage rating system chosen for this study is the one recently developed by Grattan-Bellew (1995a) at the Institute for Research in Construction-National Research Council of Canada (IRC-NRC) to evaluate distress in concrete structures. The method was designed for evaluating the condition of concrete damaged primarily by ASR.

Damage ratings were conducted on concrete prisms that had been monitored for long-term expansion to determine if a relationship existed between the DRI and expansion due to AAR. To date, no studies have been attempted to correlate the DRI to concrete expansion data.

Research significance

The damage rating system is a novel approach to quantifying deterioration of concrete due to AAR. However, the technique is still under development. This paper discusses the results of a preliminary evaluation of the technique to compare DRI to concrete expansion data. The current method of evaluating the potential expansiveness of aggregate-cement combinations due to AAR is by monitoring expansion of concrete prisms under laboratory condition for a period of one year. This method provides an indirect measure of the potential for AAR in concrete without quantifying nor describing the damage due to alkali-silica gel formation and concrete expansion. Finally, a multivariable statistical analysis was conducted on a limited data set in an attempt to revise the weighting factors and determine whether a statistical relationship existed between the linear expansion and the observed features of ASR.

The damage rating index

Damage ratings were performed on concrete prism according to the method described by Grattan-Bellew (1995a). Each feature characteristic of ASR (Table 1) was counted and the results were normalized for an area of 100 cm^2, multiplied by a weighting factor and summed to give the DRI. For this study, the gel fluorescence test (Natasaiyer & Hover, 1988) was employed prior to counting the features of ASR and was found to be effective in reducing the possibility of missing gel deposits during the damage rating analyses.

Table 1 Features and weighting factors assigned to characteristics of ASR

Feature measured	Feature description	Weighting factor
A	Coarse aggregates with cracks	0.25
B	Coarse aggregates with cracks and gel	2.00
C	Coarse aggregates debonded	3.00
D	Reaction rims around aggregates	0.50
E	Cement paste with cracks	2.00
F	Cement paste with cracks and gel	4.00
G	Air voids lined or filled with gel	0.50

gel refers to alkali–silica gel

Concrete prism specimens

Damage ratings were performed on 21 concrete prisms, ranging in age from 2 to 15 years, and were previously cast and tested according to CSA/ASTM/Ontario Hydro test procedures. Eleven prisms underwent accelerated expansion testing at 38°C only (Table 2), two prisms were tested at 23°C only (Table 3), and four prism sets (two prisms per set) were tested at both 38°C and 23°C (Table 4). The expansion and DRI results for each prism is recorded on Tables 2 to 4, inclusive. Prisms were cast from fourteen different coarse aggregate sources which have been classified into seven distinct types (Table 5).

The influence of fine aggregate on ASR is negligible in this study as known non-reactive aggregate was used to cast the prisms. The alkali content of the cements used was most commonly 1.17% (Na_2O equivalent) and, through the addition of NaOH during concrete mixing, was as high as 2.0%.

Table 2 Percent expansion and DRI results for concrete prisms stored at 38 °C and 100% rh

Sample number	Cement alk"%" content	Age (years)	Maximum "%" expansion	DRI
1	1.04	11	0.070	40
2	1.04	11	0.058	30
3	1.17	11	0.061	60
4	1.17	10	0.085	100
5	1.17	9	0.02	60
6	1.17	9	0.07	30
7	1.17	5	0.011	20
8	1.17	4	0.078	50
9	1.17	4	0.181	160
10	1.17	4	0.086	20
11	1.25	2	0.162	300

Table 3 Percent expansion and DRI results for concrete prisms stored at 23°C and 100% rh

Sample number	Cement alk"%" content	Age (years)	Maximum "%" expansion	DRI
12	2.00	15	0.029	120
13	2.00	15	0.063	60

Table 4 Percent expansion and DRI results for concrete prisms stored at 38 °C and 23 °C (*) and 100% rh

Sample number	Cement alk"%" content	Age (years)	Maximum "%" expansion	DRI
14−1	1.04	13	0.069	10
*14−2	1.04	13	0.027	40
15−1	1.04	13	0.018	30
*15−2	1.04	13	0.016	30
16−1	1.17	9	0.021	20
*16−2	1.17	9	0.015	20
17−1	1.17	9	0.085	100
*17−2	1.17	9	0.022	100

Alk= alkali content of cement (Na2O equivalent)

Table 5 Coarse aggregates used to cast prisms to test for potential AAR

Sample number(s)	Coarse aggregate source
3,4,5,11	limestone
15−1,15−2	40% limestone, 60% silicates
16−1,16−2	dolomite
8,9,10	greywacke
1,2,6	intermediate (intrusive?) gneiss
7	granitic gneiss
17−1,17−2	sandstone
12,13	slag from Canadian nickel producer
14−1,14−2	high density (magnetite−hematite blend)

Damage rating results

A plot of percent expansion against DRI for all prisms demonstrated the following results (Figure 1):

- There is no apparent correlation between expansion and DRI.
- All prisms with a DRI of less than or equal to 120 expanded less than 0.09%.
- Prisms having expansions greater than 0.09% have DRI's greater than or equal to 160.
- 67% of the prisms with greater than 0.04% expansion, a threshold value where cracking is usually first visible, showed evidence of cracking.

A comparative plot of all prisms tested at 23°C and 38°C at 100% relative humidity shows no correlation for either temperature (Figure 2). Overall, the samples tested at 38°C show a wider scatter of data and have larger expansions than the prisms tested at 23°C. Higher expansions would be expected for concrete prisms tested in an accelerated environment.

A comparative plot of identical concrete prisms, described in Table 4, of the same mixture compositions and age and tested under different temperature conditions (Figure 3) show that samples tested at 38°C expanded more than samples tested at 23°C. The DRI for samples tested at 38°C are identical to those at 23°C except for sample 14 (-1,-2), containing high density coarse aggregate, which has a lower DRI of 10 for the prism tested at 38°C as compared to the DRI of 40 at 23°C. Both samples 14 and 17(-1,-2) have a large spread in their expansions at different temperatures although the damage rating does not differ significantly.

A plot of expansion against DRI for concrete prisms cast using different coarse aggregate sources is shown on Figure 4. The classification of the coarse aggregate sources used in each mix design is given in Table 5. A correlation may exist between percent expansion and DRI for prisms cast with intermediate gneiss and granite gneiss aggregates (see Trend 1, Figure 4). Furthermore, a correlation may exist for prisms cast with coarse carbonate aggregates (see Trend 2, Figure 4) and for those prisms cast with greywacke aggregates, see Trend 3, Figure 4. All data points for the remaining aggregate sources are scattered with insufficient data to establish possible correlations.

Figure 1 Plot of percent expansion versus DRI for all prisms. Concrete begins to crack between 0.04 to 0.06% expansion.

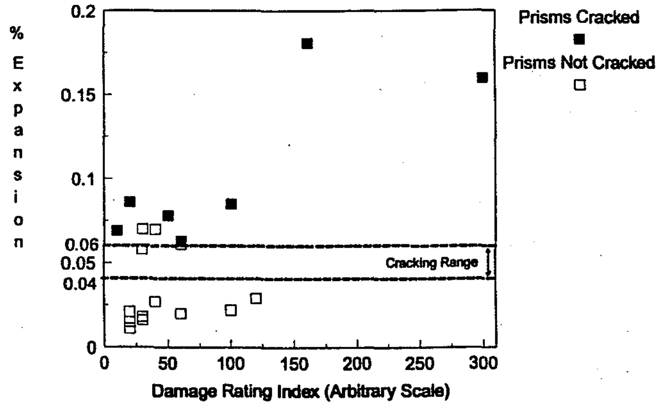


Figure 2 Plot of percent expansion versus DRI for prisms tested at 23°C and 38°C and 100% rh

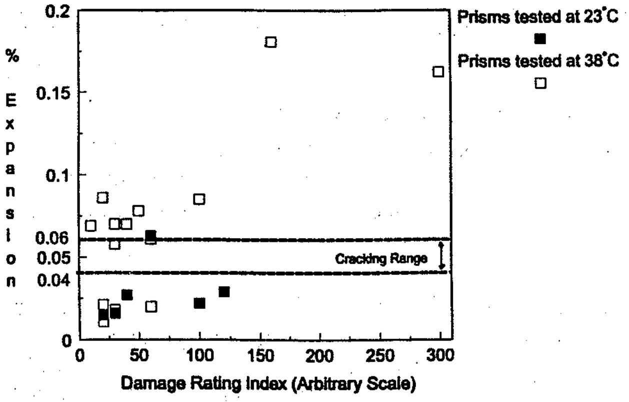


Figure 3 Plot of percent expansion versus DRI for identical prisms tested at 23°C and 38°C and 100% rh

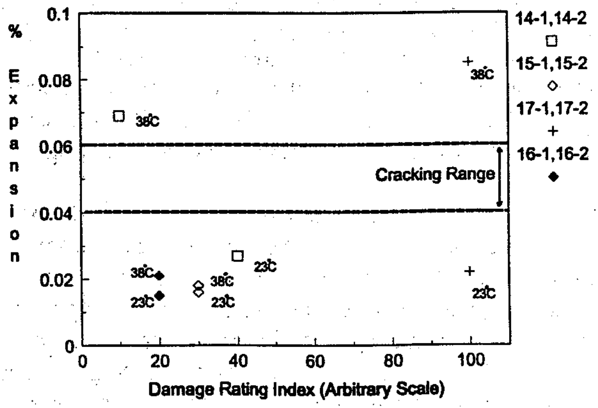
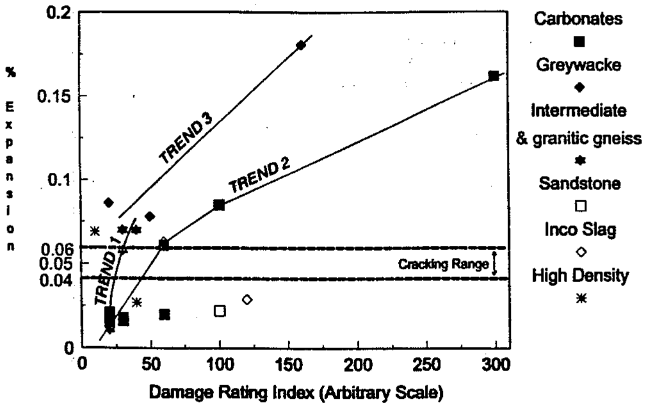


Figure 4 Plot of percent expansion versus DRI for prisms caste with different coarse aggregates. Lines have been drawn to illustrate possible trends



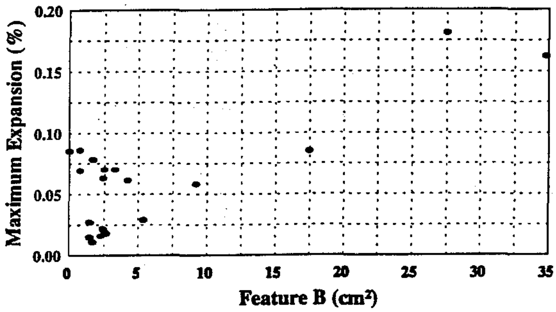
Statistical analysis of data

A multivarible statistical analysis was conducted on the data set in an attempt to determine the influence of weighting factors and determine whether a relationship existed between the linear expansion and the observed features of ASR, features A through G (Table 1). This analysis met with limited success. However, an inspection of the raw data, where each feature was plotted against maximum prism expansion, demonstrated that coarse aggregates with cracks (feature A) and coarse aggregates with cracks and alkali-silica gel (feature B) showed the best trend with maximum expansion (Figure 5). The plots for the other features showed no significant trends in the data nor contributed any valuable information concerning their effect on maximum expansion. Feature B was found to increase linearly with maximum expansion and was tested in the regression modelling procedure. Based on the limited data set, the regression equation that best modeled the maximum expansion of concrete suffering solely from ASR was:

$$\text{maximum expansion} = 0.0039 * \mathbf{B} + 0.0358$$

where **B** is the number of cracks in the coarse aggregate containing alkali-silica gel counted and normalized to 100 cm^2. However, this model should be used with caution as the representativeness and the accuracy of the model are suspect due to the small and biased data set.

Figure 5 Feature B (coarse aggregates with cracks and gel) versus maximum expansion



DISCUSSION

There are two distinct populations of data on a plot of expansion versus DRI; those prisms with a DRI of less than or equal to 120 with less than 0.09% expansion and prisms with DRI's and expansions exceeding these values. An upper limit of 0.09% expansion is very high for this study since concrete usually begins to crack between 0.04 to 0.06% expansion but this limit is the best that can be achieved using the available data. The statistical analysis confirms the conclusion that a larger sample size including more specimens with high expansion values is necessary for future studies.

The fact that no correlation exists between expansion and DRI may be attributed to either one or both of the following explanations: A - the weighting factors used in the summation of the DRI may require adjustments; B - no single relationship exits between petrographic features and expansion for all aggregate types. Of all seven features characteristic of ASR used to calculate the DRI, the number of cracks in aggregates most affect the overall calculation for DRI. Even though the aggregate cracking is assigned the lowest weighting factor (X0.25) it can often be difficult to discern the source of the cracking and thus concrete damage due to ASR can be overestimated. For example, all too often cracks within the aggregate predate concrete manufacturing and were derived from a combination of cracking due to ancient tectonic forces in the bedrock, from freeze-thaw action and quarrying operations. The weighting factor of X0.5 assigned to alkali-silica gel either lining air voids or completely infilling voids may be too high and thus overestimate the DRI. The question remains as to how much damage to the concrete is truly being contributed by alkali-silica gel formation in air voids if the cement paste surrounding these voids is not cracked due to gel formation and expansion. Therefore, the weighting factors may require adjustments. The possibility that no single relationship exits between petrographic features and expansion for all aggregate types is shown to some extent on Figure 4 where there appears to be a correlation between expansion and DRI for prisms cast with either intermediate and granitic gneiss, carbonate or greywacke coarse aggregates; different trends for different rock types. Although insufficient data exists to reinforce these trends at this time, the trends indicate that ASR will be manifested in different ways within different types of aggregates. For example, for a given level of expansion one might expect to observe less obvious signs of gel formation in a dense aggregate with well dispersed and finely disseminated reactive microcrystalline quartz, which is often the case in greywacke, than in an aggregate with the large chunks of reactive silica mineral phases such as chert. Research has shown that different trends exist for different aggregate types when expansion is plotted against a range of microcrystalline quartz percentage found within a population of a particular aggregate type (Grattan-Bellew, 1992). Different levels of alkali-silica gel production for a specific aggregate type will result in differing levels of damage and expansion to a concrete specimen thus producing the trends observed above.

Although not presented here, it is our experience that this damage rating system has demonstrated itself to be an excellent tool in evaluating the distribution of damage due to ASR in a single concrete structure.

CONCLUSIONS AND RECOMMENDATIONS

The data presented and evaluated in this report is limited. Therefore, additional damage ratings on concrete prisms are necessary to compile a larger database for comparative analysis. Concrete prism with a DRI rating of less than 120 may have experienced less than 0.09% expansion. These results can only be applied to unrestrained concrete specimens that have not been subjected to freeze-thaw damage, chemical attack, or any other mechanism of concrete deterioration typical of outdoor exposure conditions. There appears to be a limited but different correlation between expansion and DRI for prisms cast with either intermediate and granitic gneiss, carbonate or greywacke coarse aggregates indicating that any relationship between petrographic features of ASR and expansion may be aggregate dependent.

In order to facilitate further development of the damage rating system this technique should be applied to a limited number of field concrete specimens and reviewed for its applicability under field conditions. The weighting factors should be re-evaluated taking into consideration the following: the physical damage to the concrete, other than by AAR; the origin of cracks in the aggregates; if air voids filled or lined with alkali-silica gel contribute significantly to concrete deterioration; and if cracking occurring in aggregates that do not propagate into the surrounding cement paste should be ignored during the damage rating analyses.

Further studies should be initiated to determine how DRI varies with concrete expansion, aggregate type, restraint, stiffness and age of concrete. Included in these studies should be specimens cast with supplementary cementing materials, used to retard AAR.

ACKNOWLEDGEMENTS

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STUDY ON A STANDARD RAPID TEST METHOD FOR IDENTIFICATION OF ASR SUSCEPTIBILITY OF CONCRETE

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Abstract

In order to certify the durability to ASR in a given freshly mixed concrete, the final draft of a standard rapid test method for identification of susceptibility to ASR in concrete was developed and published. By the rapid test,the susceptibility can be identified two days after making specimens. For the acceleration of ASR in concrete, additional NaOH powder and a boiling procedure are applied,which is very similar to the rapid method for aggregates JIS A 1804 standardized in 1992.

This paper describes the outline of the rapid test procedures and main points of experimental results obtained during five years from 1989 through 1993 by the committee.

Keywords : *ASR,concrete,rapid test,standard test method*

Introduction

In Japan,in order to prevent premature deterioration of concrete structures due to ASR, several test methods have been standardized. These comprise two groupes:one is for the evaluation of alkali reactivity of aggregates,and the other for the future susceptibility to ASR of a given concrete mix. The former includes a chemical method (JIS A 5308 Appendix 7 which was developed by modifing the chemical method in ASTM C 289),a mortar bar method (JIS A 5308 Appendix 8 which was developed by modifing the mortar bar method in ASTM C 227) and recently a rapid method (JIS A 1804). The second group includes both long-term expansion test methods in JASS 5N T-603 and JCI AAR-3 standardized in 1991 for concrete structures in nuclear power stations by the Architectural Institute of Japan and for general concrete structures by the Japan Concrete Institute, respectively.

The rapid test method JIS A 1804 was published in 1992 as one of the standard test method series for production control of concrete designated as JIS A 1800s[1]. The rapid test method presented in this paper has been developed by modifing the rapid test method in JIS A 1804 and for the evaluation of the future susceptibility to ASR of a given concrete mix.

The final draft of a standard test method presented in this paper has been also prepared for one of JIS A 1800s. This draft was established based upon the comprehensive investigations conducted in the committee organized by the National Ready Mixed Concrete Industry Association under the guidance of Agency of Industrial Science and Technology in the Ministry of International Trade and Industry from fiscal 1989 to 1993 including laboratory test,field tests and a repeatability test [2]. This paper outlines the test procedures and main points of the experimental results.

Rapid test procedures

The flowchart of the rapid test is illustrated in Fig. 1. The outline of the rapid test method is described below.

Main apparatus used in the test are three molds for cylindrical specimens of $\Phi 100 \times 200$ mm,a reaction accelerating apparatus capable of a gauge pressure of 0.049 MPa (temperature:111 ℃) and an apparatus for measuring dynamic modulus of elasticity.

Three cylindrical specimens of $\Phi 100 \times 200$ mm are made of a given freshly mixed concrete. Prior to casting concrete in the mold,NaOH powder shall be added to the concrete. Amount of NaOH shall be adjusted so as to be 9 kg/m^3(Na_2Oeq) of additional alkali in the mix.

In order to simply measure the concrete volume, a vessel of 7ℓ for air content measurement can be used. 81.3 g of NaOH powder should be added to 7ℓ of freshly mixed concrete. Small spherical powder of NaOH (0.7 mm in diameter,TOSO-PARL (Brand name)) is convenient for the test. Just after casting concrete,the surface of specimens shall be finished by a trowel.

Then the specimens shall be cured for two days:for the first day remained in the molds in a moist cabinet at (20±3) ℃,for the second day removed from the molds and kept in water at (20±3) ℃.

After the two-day curing,in order to accelerate chemical reaction,three specimens shall be boiled in water at a gauge pressure of 0.049 MPa for two hours. Specifically mentioned,the water temperature shall be adjusted to 40 ℃ at the beginning of boiling,raised up to 111 ℃ within 30±10 minutes,maintained 111 ℃ for two hours, and then lowered to 20～40 ℃ within 30±10 minutes. After these treatment, specimens shall be taken off the vessel and dipped into water at 20 ℃ for 20 minutes before the following measurement.

In order to evaluate the susceptibility to ASR, the first resonance frequency of the specimens shall be measured just before and after boiling. Relative dynamic modulus of elasticity shall be calculated using the following formula.

$$\text{Relative dynamic modulus of elasticity} = \frac{\begin{bmatrix}\text{First resonance frequency} \\ \text{after boiling}\end{bmatrix}^2}{\begin{bmatrix}\text{First resonance frequency} \\ \text{before boiling}\end{bmatrix}^2} \times 100 \ (\%)$$

Test results can be obtained as mean value of the three results calculated down to the first decimal place.

In case of the relative dynamic modulus of elasticity of 80% and more after the boiling to that before the boiling,the concrete is evaluated"Durable to ASR (Grade A)". In case of less than 80%, the second test using 6 kg/m^3 (Na_2Oeq.) instead of 9 kg/m^3 of additional alkali in the mix shall be conducted. In case of 70% and more at the second test,the concrete is assessed"Durable to ASR (Grade B)". In case of less than 70% at the second test,the concrete is identified"Susceptible to ASR".

"Durable to ASR (Grade A)"concrete can be used for highly durable concrete structures. "Durable to ASR (Grade B)"concrete can be generally used,however,can not be used for highly durable concrete structures. "Susceptible to ASR"concrete can not be used generally,however,it can be used under extremely dry condition.

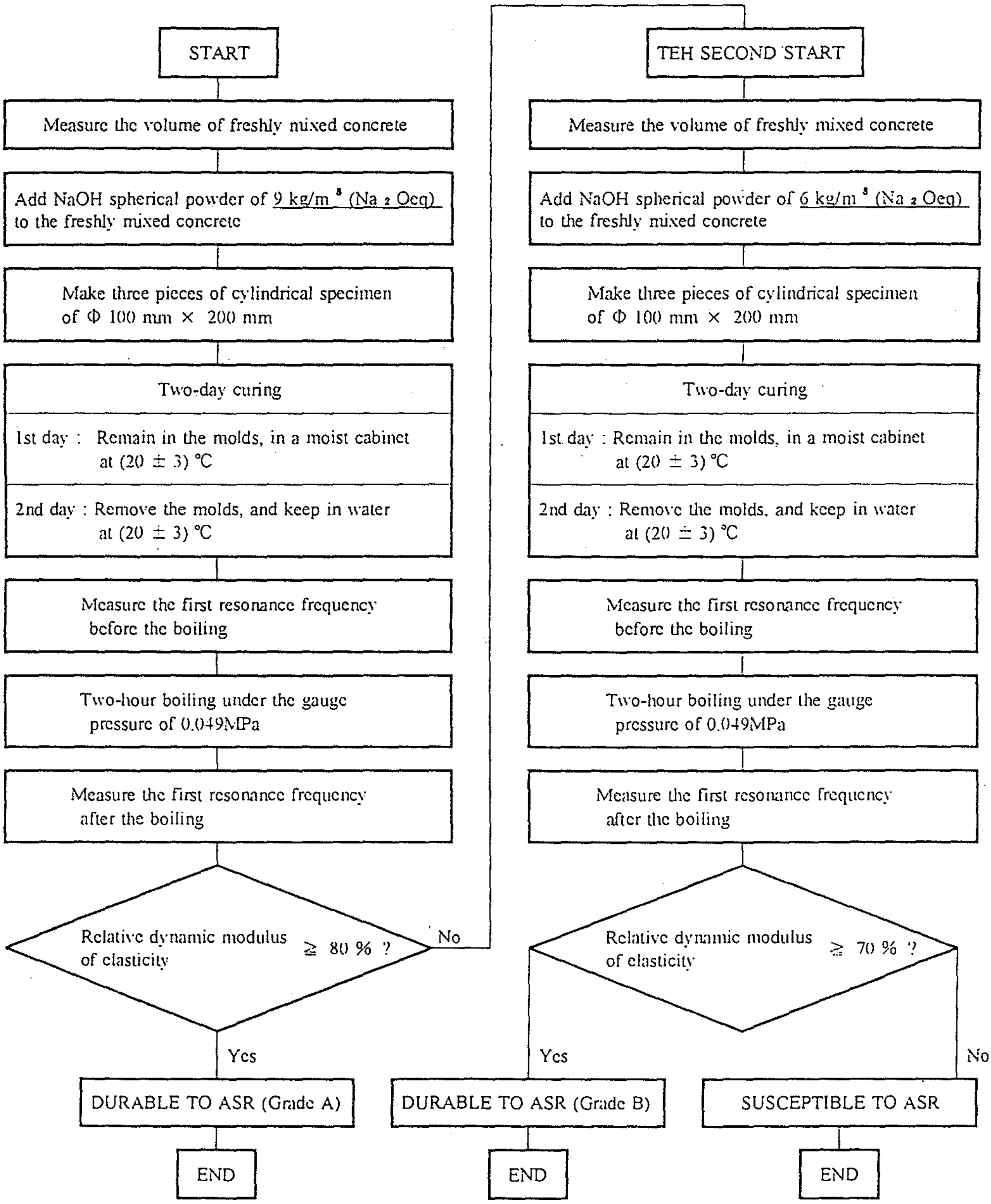


Fig.1 Flowchart of the rapid test for ASR in concrete

Experimental results

Laboratory test

Totally 206 concrete mixes have been examined in terms of the susceptibility to ASR using both some long-term expansion test in accordance with JCI AAR-3 and/or JASS 5N T-603 and the rapid test [2][3].

Expansion tests were carried out using concrete prism specimens of 75×75×400 mm. Specimens were stored in a moist condition of 40℃,100%RH,by being wrapped in wet paper and sealed in a plastic bag. In those expansion tests, additional alkali NaOH powder is also added to a given freshly mixed concrete. With regard to the additional alkali content,in JCI AAR-3,only one level of 2.4 kg/m^3 (Na_2Oeq.) is applied,however,in JASS 5N T-603,three levels of 1.2,1.8 and 2.4 kg/m^3 (Na_2Oeq.) are examined for obtaining the pessimum expansion between 0 to 3 kg/m^3 of additional alkali content by extrapolation. As for the threshold expansion for the evaluation is 0.10% at 6 month in the both expansion tests.

Many types of cement and aggregate,several kinds of aggregate combination,wide rage of unit weight of cement, consequently wide range of total alkali content in concrete were provided for the test. Fig.2 shows the relationships between the expansion test results and the rapid test ones. Based on the results,the threshold relative dynamic modulus of elasticity value of 80% was established.

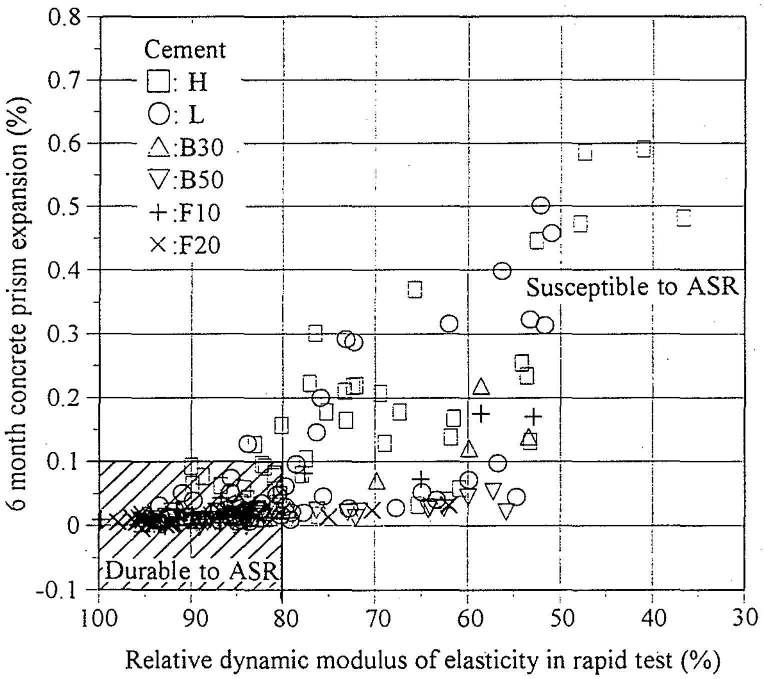


Fig. 2 Relationships between rapid test results and expansion test results

Field test

The rapid test using additional alkali of 9 kg/m^3 (Na_2Oeq.) was applied for eighteen kinds of ready mixed concrete mix actually produced in plants all over Japan. All the mixes use ordinary portland cement,and contain total alkali less than 3 kg/m^3. Long-term expansion test by JCI AAR-3 was also conducted using those mixes. According to the field tests,some concrete mixes showed extremely low values of relative dynamic modulus of elasticity by the rapid test,however,they did not show large expansion values.

Figs. 3 and 4 show the expansion test results and the relationships between additional alkali content and the rapid test results,respectively. Details of the concrete-making materials and concrete mix used in the expansion tests and the rapid tests are shown in Table 1 through 4. Fig. 3 includes the following three kinds of concrete mix. Mix No.1 using deleterious andesite gravel,which is discovered in the above mentioned field test,can be evaluated"Durable to ASR"based upon both the expansion test result and an observation field survey on existing concrete structures using the concrete mix,however showed extremely low value test results by the rapid test. Mix. No.2 can bealso evaluated"Durable to ASR"based upon the expansion test result, however showeda rapid test result very near the threshold value. Mix No.3 using crashed deleteriousandesite sand,which is considered the most susceptible to ASR in Japan,can beevaluated"Susceptible to ASR"based upon both the expansion test and the rapid test.

Judging from the results,the.second test using additional alkali of 6 kg/m^3 (Na_2O eq.) after the first test using additional alkali of 9 kg/m^3 and the threshold value 70% was established.

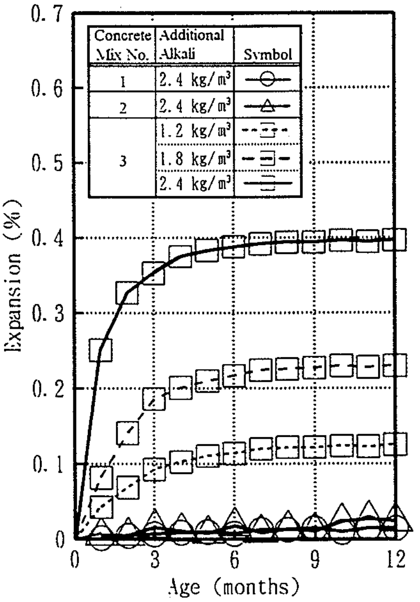


Fig. 3 Long-term expansion test results

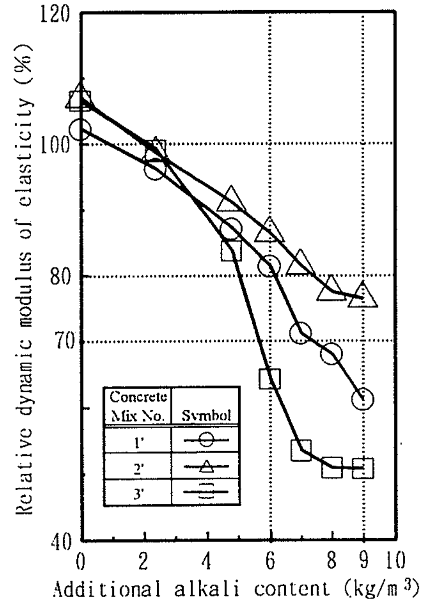
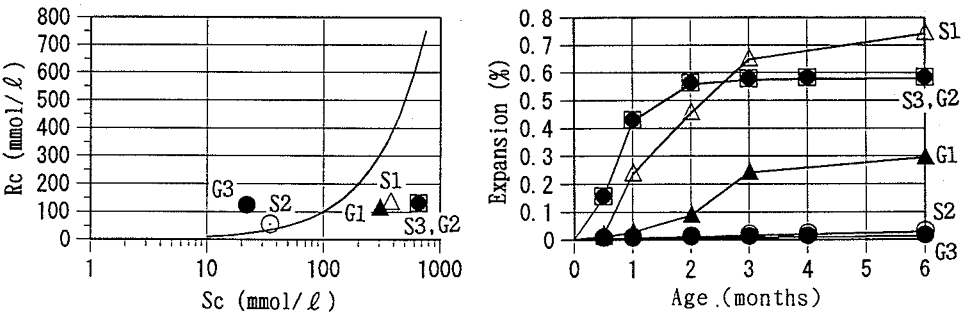


Fig. 4 Effect of additional alkali content on rapid test results

Table 1 Materials used for expansion tests

Ordinary portland cement	C_1	Total alkali(Na_2Oeq) : 0.68 %
	C_2	Total alkali(Na_2Oeq) : 0.58 %
	C_3	Total alkali(Na_2Oeq) : 0.65 %
Fine aggregate	S_1	River sand from SAI Alkali-silica reaction : Deleterious
	S_2	Marine sand from MUROKI Alkali-silica reaction : Innocuous
	S_3	Crushed sand from TESHIMA Alkali-silica reaction : Deleterious
Coarse aggregate	G_1	River gravel from SAI Alkali-silica reaction : Deleterious
	G_2	Crushed stone from TESHIMA Alkali-silica reaction : Deleterious
	G_3	Crushed sand from NAGAOYAMA Alkali-silica reaction : Innocuous
Chemical admixture	A	Air entraining water reducing agent



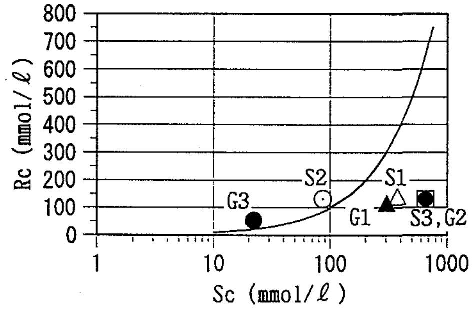
Test results by the JIS chemical method Test results by the JIS mortar-bar method

Table 2 Concrete mixes for expansion tests

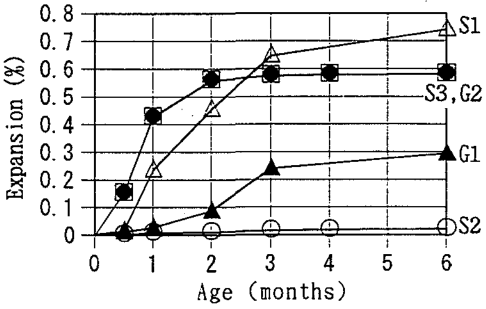
Concrete Mix No.	W/C	s/a	Unit weight (kg/m³)				Chemical admixture	Total alkali content
			Water	Cement	Aggregates			
					Fine	Coarse	A	(Na_2Oeq)
			W	C	S	G	(C×%)	(kg/m³)
1	58.0	47.1	172	C_1: 297	S_1: 851	G_1: 965	0.25	2.02
2	55.0	45.0	185	C_2: 350	S_2: 765	G_2: 910	0.25	2.03
3	52.0	45.3	191	C_2: 367	S_2: 529 S_3: 227	G_2: 981	0.25	2.39

Table 3 Materials used for rapid tests

Ordinary portland cement	C	Total alkali(Na_2Oeq) : 0.65 %
Fine aggregate	S_1	River sand from SAI Alkali-silica reaction : Deleterious
	S_2	River sand from IBE Alkali-silica reaction : Innocuous
	S_3	Crushed sand from TESHIMA Alkali-silica reaction : Deleterious
Coarse aggregate	G_1	River gravel from SAI Alkali-silica reaction : Deleterious
	G_2	Crushed stone from TESHIMA Alkali-silica reaction : Deleterious
	G_3	Crushed sand from MAKIOYAMA Alkali-silica reaction : Innocuous
Chemical admixture	A	Air entraining water reducing agent



Test results by the JIS chemical method



Test results by the JIS mortar-bar method

Table 4 Concrete mixes for rapid tests

Concrete Mix No.	W/C	s/a	Unit weight (kg/m³)				Chemical admixture A (C×%)	Total alkali content (Na_2Oeq) (kg/m³)
			Water W	Cement C	Aggregates Fine S	Aggregates Coarse G		
1’	55.0	46.0	175	C_1: 318	S_1: 812	G_1: 960	0.25	2.01
2`	55.0	46.0	189	C_2: 343	S_2: 783	G_2: 901	0.25	2.23
3`	55.0	46.0	189	C_2: 343	S_2: 548 S_3: 230	G_2: 933	0.25	2.23

After establishing the final draft of the rapid test,field tests using ready mixed concrete actually produced in plants all over Japan were carried out with cooperation of concrete testing laboratories under the National Ready Mixed Concrete Industry Association both in 1993 and 1994. Ready mixed concrete applied for the rapid tests have aggregate maximum size of 20 mm or 25 mm,slump of 80 mm through 180 mm and nominal compressive strength of 21 MPa,24 MPa or 27 MPa. All the concrete mixes contains total alkali less than 3 kg/m^3. Around 70% mixes out of 76 mixes were evaluated"Durable to ASR (Grade A)",and around 30% mixes were identified "Durable to ASR (Grade B)"in 1993. In 1994,around 65% mixes from 44 mixes were evaluated"Durable to ASR (Grade A)",and the lest"Durable to ASR (Grade B)".

Concluding remarks

In order to certify the durability to ASR in a given freshly mixed concrete,the rapid test method for identification of ASR susceptibility of concrete is a very effective measures. By the rapid test,the susceptibility can be identified two days after making specimens. In order to accelerate ASR in concrete,additional NaOH powder and a boiling procedure are applied,which is very similar to the rapid method for aggregates JIS A 1804 standardized in 1992. This final draft will be standardized as one of JIS A 1800s in the near future.

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NBRI TESTS ON AGGREGATE CONTAINING STRAINED QUARTZ

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ABSTRACT

Instances of distress to concrete structures due to alkali-silica reaction have been diagnosed in India. The reactivity of aggregate has been ascribed to the presence of strained quartz. Standard test methods developed to detect reactivity of classical reactive aggregate, failed to detect potential reactivity of such slowly reactive aggregate, containing quartz with deformed crystal lattice. This paper presents results of evaluation of a number of aggregate samples containing strained quartz by accelerated NBRI mortar-bar test. A critical evaluation of the test method is attempted.

Keywords: accelerated tests, alkali-silica reaction, salt solution, specification, temperature.

INTRODUCTION

Instances of distress to concrete structures in India due to deleterious alkali-silica reaction have been diagnosed about a decade back. The reactivity in such aggregate was ascribed to the presence of strained quartz. Standard test methods, which were developed for aggregate containing metastable silica minerals like opal, chert and chalcedony and volcanic glasses etc. as the reactive component, (hereinafter called 'classical' reactive aggregate) fail to detect potential reactivity of such slowly reactive aggregate containing strained quartz. Appropriate test methods are, therefore, sought to be developed, by modification of the ASTM C-227 mortar bar test.

Need has, simultaneously, been felt for quick and, at the same time, reliable test methods to detect potential alkali-reactivity of aggregate of all types, whether classical reactive aggregate or those containing quartz with deformed crystal lattice, secondary inclusions or microcrystalline quartz etc. The aggregate samples evaluated in the present investigation belong to the latter category. The commonly used mortar bar test as per ASTM C-227 takes about a six month period to show any dependable trend. To cut down the time of evaluation further, the accelerated mortar bar test developed by the National Building Research Institute (NBRI) in South Africa (Oberholster & Davies, 1986) has received much attention the world over. Considerable data has become available on the use of NBRI test on classical reactive aggregate, but data on aggregate containing quartz with deformed crystal lattice is rather limited.

This paper presents the results of an evaluation of a number of aggregate samples containing strained quartz by accelerated NBRI mortar bar test. The results also enable a critical evaluation of the test method and what could be the criterion of potential reactivity.

NBRI ACCELERATED MORTAR BAR TESTS

The procedure consists of preparing mortar-bar samples in the same way as ASTM C-227 i.e. by proportioning 1 part of cement to 2.25 parts of graded aggregate by mass (Oberholster & Davies, 1986). However, a fixed water to cement ratio, i.e. 0.44 for natural fine aggregate and 0.50 for coarse aggregate, is used, instead of water content determination by flow table test, as in the ASTM C-227 method. The samples, after 24 hours of normal curing, are cured in hot water at $80^{o}C$ for 24 hours. Finally, the specimens are stored in 1N NaOH solution at $80^{o}C$. Length change measurements are taken in the hot condition i.e. within 20 seconds from removing the specimens from the solution.

Results of NBRI tests reported in the literature on different aggregate samples - both classical reactive aggregate as well as slowly reactive aggregate containing deformed quartz, show that mortar-bar expansion of the same order are generally obtained in the case of classical reactive aggregate as well as slowly reactive aggregate at 12 to 14 days (Berube et al., 1992, Kerrick & Hooton, 1992). On the other hand, in ASTM C-227 mortar-bar tests at $38^{o}C$ or at $60^{o}C$, slowly reactive aggregate usually show lower orders of expansion upto six months than classical reactive aggregate (Mullick, 1994). In general, it has also been observed that a greater magnitude of expansion is obtained in the NBRI test than in the ASTM C-227 test and there is always a chance that aggregate with satisfactory field performance may be classified as expansive in the NBRI test (Grattan-Bellew, 1989, Berube et al., 1992).

While no firm criteria has yet been established for the evaluation of reactivity of aggregate by the NBRI test, it has generally been suggested (Davies & Oberholster, 1987, ASTM C 1260-94) that:

i) Expansion of less than 0.10 percent is indicative of "innocuous" aggregate;

ii) Expansion of greater than 0.20 percent to 0.25 percent is indicative of "potentially deleterious" aggregate;

iii) Expansions falling between these two limits have been considered either as 'slowly reactive' aggregate or 'inconclusive', and continuation of the test for longer periods is recommended.

EXPERIMENTAL

Twentyfive different types of natural aggregate containing strained quartz (Mullick, 1987, Mullick et al., 1992,) were taken up for investigation. These aggregate samples did not contain metastable silica minerals like opal, chert or chalcedony etc. The aggregate samples included different rock types namely quartzites, granites, greywackes, phyllites, composite samples and samples of natural sand. The angle of undulatory extinction (UE) in different rocks, as measured by the procedure of Dolar-Mantuani, 1983, varied from 10^{o} to 40^{o} and the content of quartz showing strain effect varied from 20 to 90 percent. Characteristics of granitic rocks were similar to aggregates, which have been found to result in deleterious alkali-silica reaction in concrete in service (Mullick, 1994).

ASTM C-227 mortar-bar expansion tests at 38^{o}C and 60^{o}C with ordinary portland cement of total alkali content (as Na$_2$O eq) more than 1 percent were carried out to check the potential reactivity of these aggregates as per the criteria proposed (Mullick et al., 1992). From the results of mortar bar tests at 38^{o}C as well as 60^{o}C given in Figure 1, it is seen that eleven out of 25 samples of aggregate can be classified as 'innocuous', as the expansion for these samples under the 60^{o}C regime are below the proposed limit of 0.06 percent at 180 days (Mullick, 1994). Most of them satisfy the ASTM C-227 limit at 38^{o}C.

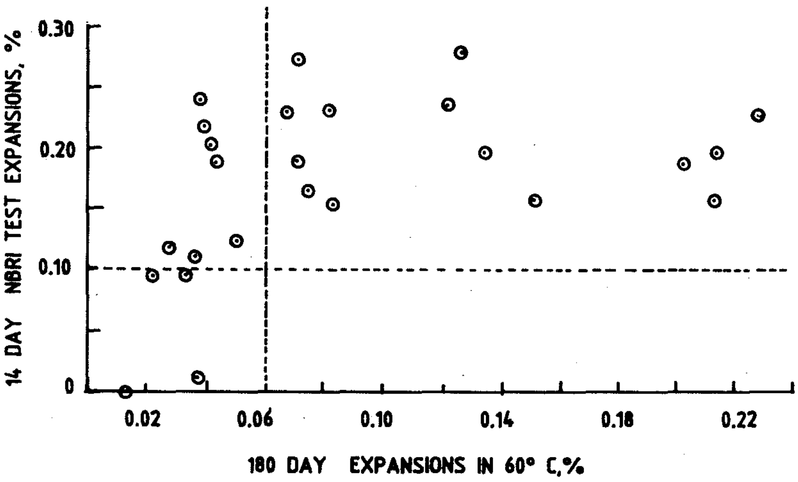

Fig. 1 Results of ASTM C 227 Mortar bar tests at 60^{o}C and NBRI tests on 25 Aggregate Samples

NBRI tests were carried out on the same aggregate samples as per the procedure described above. The same cements as in the 38^{o}C and 60^{o}C tests were used in the NBRI tests. Only such cement samples were used in the present investigation, whose expansion in the autoclave (soundness) test were of the order of 0.1 percent or less, to rule out any possibility of the effect of the unsoundness of cement on the resultant expansion. The use of glass or metal containers for storing the specimens in 1N NaOH solution at 80^{o}C is not recommended as the same get corroded by the test solution. Therefore, plastic containers with airtight covers were used.

The expansion readings were taken periodically upto the required 14 days and also continued upto 56 days in many cases.

In order to evaluate the test procedure, some additional tests were undertaken. These included measurement of total alkali content in mortar bar specimens after periods of exposure, use of cements with different alkali contents, as well as ASTM C-227 mortar bar tests carried out at 27^{o}C, 38^{o}C, 60^{o}C and 80^{o}C.

Discussion of Results

It is seen from Figure 1 that the 14 day NBRI test expansions for the 14 natural aggregate samples, which were classified as potentially reactive on the basis of mortar-bar tests at 60°C regime, are in the range of 0.15 to 0.28 percent. These expansions are comparable to those obtained by other researchers for the classical reactive aggregate. The expansions continued even after 14 days and increased to the order of 0.4 to 0.5 percent at 56 days, (Figure 2). Surface cracking was observed in the specimens at the age of 56 days. Presence of gel in the voids was observed in all the samples. The gel deposits were confined to the periphery of the samples at 14 days and progressed towards the interior after 56 days' exposure.

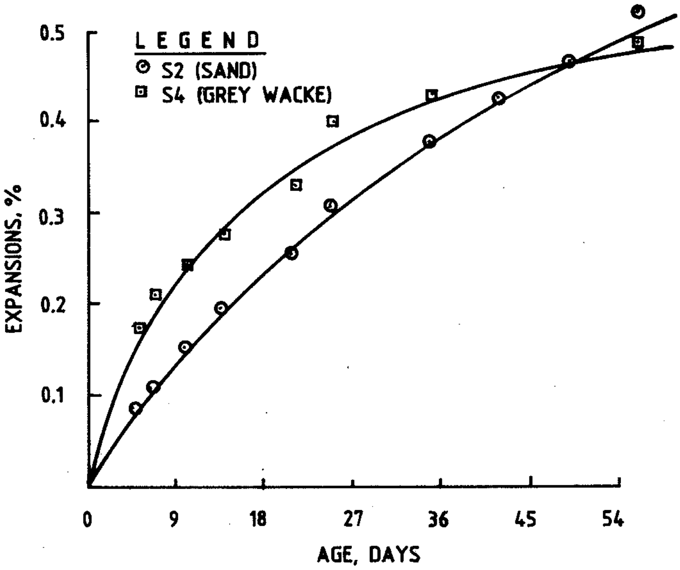

Fig. 2 Typical NBRI Expansions up to 56 days for two Aggregate Samples

No correlation between 14 days NBRI test expansions and 180 days 60°C mortar-bar expansions was apparent (Figure 1). Another eight aggregate samples, classified as 'innocuous' in the 60°C mortar-bar tests gave expansion in the NBRI test above 0.10 percent. Out of a total of 24, there were only three aggregate samples, which could be classified as innocuous in both NBRI test and 60°C mortar bar test (Figure 1).

To study the effect of inherent alkali content in the cement, NBRI tests were carried out on four aggregate samples using three cements of total alkali content 0.70, 0.76 and 1.18 percent. The 14 days expansion data are presented in Figure 3, which show that, in the presence of 1N NaOH solution, the mortar bar expansion was not influenced by the alkali content in the cement.

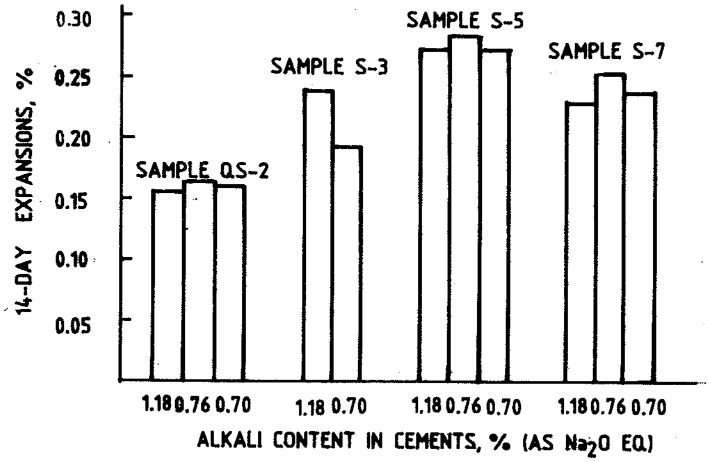

Fig. 3 14-day Expansion in NBRI test with Cements of Different Alkali Contents

Since the results indicated relatively larger expansions in NBRI tests on aggregate containing deformed quartz, which are regarded as slowly reactive, a search was made for possible reasons of acceleration of expansion. Measurement of total alkali content in mortar bar specimens showed that the amount of alkalies continued to increase with time of exposure, especially Na_2O content (Figure 4). Total alkali as Na_2O equivalent of the order of 2.73 percent by mass of mortar was measured after 100 days in the case of one sample. This was much higher than what can ever be encountered in actual constructions.

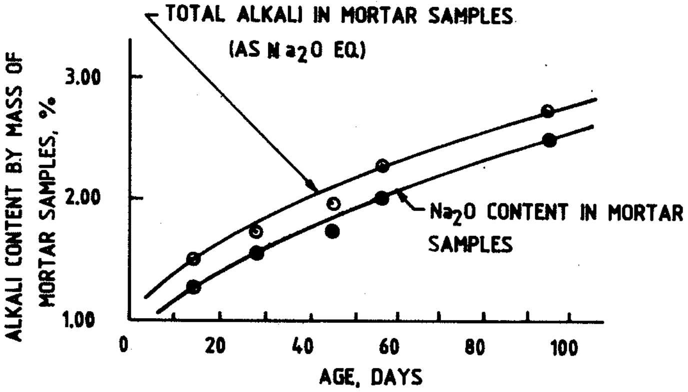

Fig. 4 Alkali Content in Mortar bar Specimens at Different Ages, S-2 Sand

Another possible reason was thermal activation i.e. effect of high temperature. Tests were conducted to study the mortar bar expansion of two aggregate samples at different temperatures. The procedure of ASTM C-227 was followed in the case of all temperatures. The results are presented in Table 1.

Table 1 ASTM C-227 Mortar-bar Expansions at Different Temperatures

Sample	Temperature Regime, °C	Expansions, %, at days						
		14	30	60	120	150	180	210
QS-4	27	0.0040	0.0052	0.0060	0.0100	0.0108	0.0108	0.0108
	38	0.0072	0.0080	0.0100	0.0140	0.0160	0.0198	0.0210
	60	0.0124	0.0144	0.0364	0.0796	0.0920	0.1086	0.1142
	80	0.0380	0.0424	0.0536	*0.0458	0.0538	0.0610	0.0694
QS-5	27	0.0036	0.0048	0.0112	0.0204	0.0228	0.0228	0.0212
	38	0.0068	0.0084	0.0132	0.0432	0.0452	0.0484	0.0500
	60	0.0104	0.0104	0.0120	0.0384	0.0676	0.0812	0.0860
	80	0.0464	0.0516	0.0644	*0.0520	0.0620	0.0720	0.0784

* Warping of specimens was noticed which increased with age.

Warping of specimens occured in tests at 80°C. Expansions increased sharply with temperature, especially above 60°C. The 14-day expansions at 80°C regime was 5 to 6 times, and at 60°C regime was about 1.5 times larger than that at 38°C (Figure 5).

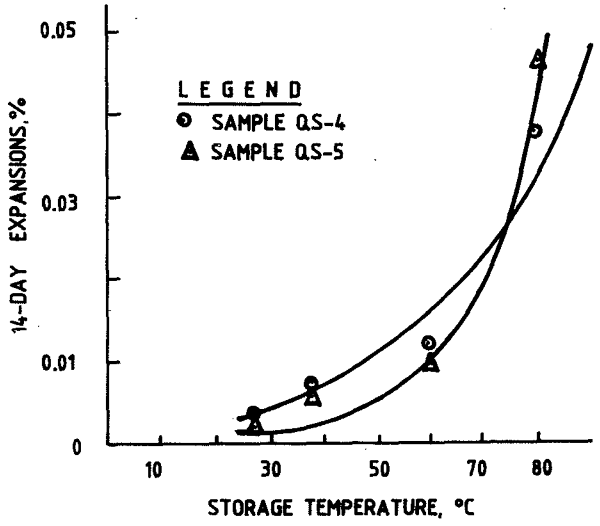

Fig. 5 Effect of Temperature on Mortar bar Expansion

It has been postulated earlier that thermal activation of mortar bar expansion due to higher temperature is more pronounced in case of slowly reactive aggregate containing deformed quartz (Mullick, 1994). The results in Figure 5 indicate a similar trend upto 80°C.

Another interesting comparison is the results of mortar bar expansion as per ASTM C-227 and NBRI test, when both are carried out at 80°C.

Results presented in Table 2 compare expansion in the case of two aggregate samples. It is seen that at comparable exposure time of 14 days , the NBRI test expansions are nearly four times greater. In this case, the acceleration is due to the presence of considerably larger amount of alkalies being available in the NBRI test.

Table 2 Comparison of Expansions in NBRI and ASTM C-227 Tests at same Temperature

Sl No	Aggregate Sample	14-day Expansion, %	
		NBRI Test*	ASTM C-227 Test*
1	QS-4	0.1524	0.0380
2	QS-5	0.1884	0.0464

*Test temperature 80°C in both cases

CONCLUSIONS

NBRI tests resulted in 14-day expansions of the order of 0.15 - 0.28 percent in the case of slowly reactive aggregate containing strained quartz, which were classified as 'potentially reactive' by ASTM C-227 mortar bar expansion tests at 60°C. Expansions in excess of 0.10 percent were obtained in the case of most aggregate samples, which were classified as 'innocuous' in the latter tests. Thus, the NBRI test did not appear to be discerning enough for the slowly reactive aggregate containing deformed quartz. As is to be expected, ASTM C-227 mortar bar test at 38°C failed to detect any potential reactivity of these aggregates. There was no one-to-one correlation between mortar-bar expansion in the NBRI test and ASTM C-227 test at 60°C. Expansions in NBRI tests continued even after 14 days. Expansions of the order of 0.4 to 0.5 percent or more were obtained at 56 days.

The total alkali content in the mortar bar specimens also increased with time of test. There was no relationship between the initial alkali content in cements and the resultant expansions. In general, NBRI tests on slowly reactive aggregate resulted in expansion of the same order of magnitude as reported in the case of classical reactive aggregate containing metastable silica minerals like opal, chert, chalcedony etc., although the former result in much lower expansion in case the of ASTM C-227 mortar bar tests at temperature upto 60°C. Possible reasons are increased thermal activation of expansion at 80°C, as well as effect of continuous intake of alkali solution in the specimens.

In view of these, the NBRI test may not be regarded as complete in itself, but could be a precursor of more detailed investigation, once an aggregate exceeds the threshold mortar-bar expansion, which is recommended to be of the order of 0.15 percent at 14 days.

ACKNOWLEDGEMENTS

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EVALUATION OF AGGREGATES FOR AAR USING ACCELERATED TEST METHODS

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ABSTRACT

There are many accelerated test methods adopted for determination of the potential reactivity of concrete aggregates . They have given satisfactory results with a few aggregates , but many of them were not reliable with aggregates from different countries . As many aggregates must be evaluated in a few days before construction , these testing methods need to be reliable, rapid and reproducible .

In the present investigation aggregates from various sources are evaluated for alkali aggregate reactivity . They will be used in the concrete of a number of dams which are under construction in Iran . Chemical methods and accelerated mortar bar tests were used for determining the alkali - reactivity of the aggregates . The concrete prism methods will be carried out for the confirmation of the accelerated test results .

Test results show that some of the accelerated test methods can be reliable for the types of concrete aggregates used in this investigation .

Keywords : Alakli - aggregate reactivity, Accelerated tests, Mortar bar .

INTRODUCTION

Alkali - aggregates reactivity (AAR) in concrete is an undesirable chemical reaction that may occur between soluble alkalies in the pore fluid and certain types of aggregates . AAR often develops as a slow progressively deteriorating process , with the first visible signs taking anywhere from a few months to many years to develop.

Except for a few “text book” cases , notable signs , such as map - cracking , oozing gel , and popouts , aren’t evident. The apparent symptoms , most often excessive or abnormal structural distortions and cracking, are not unique to AAR . Taken individually , these symptoms may be mistakenly attributed to other causes of structural problems , such as foundation deficiencies, inadequate reinforcement , temperature , or other effects .

AAR produces concrete expansion and can generally lead to a loss of strength , stiffness (cracking) , and generates undesirable deformations and disturbances in the equilibrium of internal forces .

A few concrete dams and other structures in Iran are suffering from deteriorations induced by alkali - aggregates reaction (AAR) that impair the durability , serviceability , and might also affect , in long term , the safety of the installation .

In this study the results of two rapid chemical screening test(CSA A 23.2-26A and ASTM C 289) and one rapid Mortar Bar Method (CSA A 23.2-25A) with petrographic examination on a number of aggregates is presented (1,2). The CSA A23.2-26A test is used for the evaluation of carbonate aggregates

The aim of this investigation is to study the behavior of some aggregates that will be used or have been used in the concrete in a number of dams built or under construction in Iran .

TEST PROGRAM

Materials

1- Aggregates
In this study 14 types of aggregates obtained from 4 dams have been tested. The dams which have been selected for this investigation are:

1. Shahid Rajai (with 5 aggregate types)
2. Maroon (with 2 aggregate types)
3. Karkheh (with 3 aggregate types)
4. Ahar (with 4 aggregate types)

The petrography and the physical properties of the aggregates used in this investigation are given in Table 1.

TABLE 1 :Petrography and physical properties of the coarse and fine aggregates .

Aggregates	Rajai	Maroon	Karkheh	Ahar
Type	Quarried carbonate Rock	Gravel glacial deposit	Carbonate Rock	Carbonate Rock
Rock Type	limestone (more calcite and less Dolomite)	limestone (Calcite Dolomite Quartz)	Siliceous limestone (quartz and feldspar calcite)	Siliceous limestone (calcite. quartz. feldspar)

2- Portland Cement
The physical properties and the chemical composition of the cement used is given in Table 2 .

TABLE 2 : Chemical Analysis of the Cement

cement	Insoluble residue	SiO_2%	CaO%	Al_2O_3%	Fe_2O_3%	MgO%	So_3%	free calcium oxide	loss of ignition	Na_2O%	K_2O %
Tehran	0.5%	21.2%	62.5%	5%	3.1	3.15	2.85	0.6	1.4	0.7	0.88

TEST PROCEDURES

Petrography

Lists of potentially reactive minerals and corresponding host rock types are available from a number of publications (3,4). The pertrographic examination of aggregates in thin sections under the optical microscope usually allows recognition of potentially reactive mineral phases of rocks .

Further microscopic examination of thin section of concrete samples will be carried out at a later stage for the confirmation of alkali carbonate or alkali silica reactions.

ASTM C 289 Chemical Test

The ASTM C 289 chemical method is one of the most widely used test for evaluating the potential reactivity of silica-bearing aggregates. Its use is mainly because it requires only small quantities of material and results can be obtained within a few days (1) .

CSA A 23.2-26A Chemical Test

This is a modified chemical test which is in the process of being adopted in the Canadian Standards for AAR, covers the evaluation of potential alkali-carbonate reactivity of quarried carbonate rocks. In this test method, a representative sample of the aggregates to be tested is reduced in particle size, small jaw crusher and a disk pulverizer, to pass a 160 μ m sieve. The material is then carefully homogenized and sent for chemical analysis for CaO , MgO and AL_2O_3 (2).

CSA A 23.2-25A Mortar Bar Test

Mortar bar for this test are made in accordance with CSA A23.2-25 A and immersed after demoulding in water at 23 C with the containers immediately placed in an oven and stored at 80 C. The next day, the zero reading is taken, then the bars immersed in a 1N NaOH solution and reading are taken each working day (2).

TEST RESULTS

Alkali - Aggregate Reactivity Testing

The results of ASTM C 289 chemical test for Rajai dam are given in Fig 1. This figure gives the plots of Sc (Dissolved silica mM/l) versus Rc (Reduction in alkalinity mM/l) for series of aggregates . The results of chemical analysis of aggregates are shown in Tables 3 to 6 . related plots are given in Figs 2 and 3 . In each cases the amount of Al_2O_3 versus the CaO: In each case the amount of AL_2O_3 versus the CaO : MgO ratio are ploted in Figs 2 and 3 . The results of accelerated mortar bar test method are given in Figs 4 to 5.

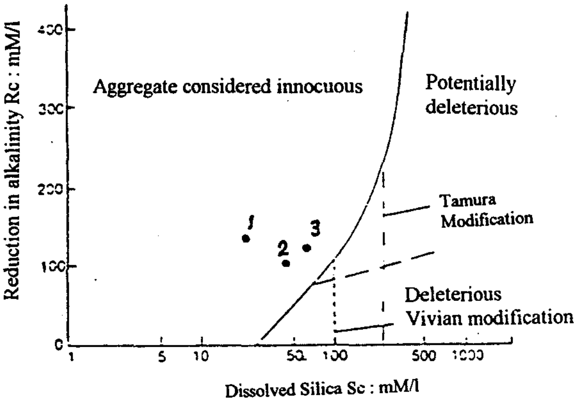

Fig1 : Results of ASTM C 289 chemical test for the Rajai dam aggregates.

* P.E = Potentially Expansive N.E = None Expansive

TABLE 3 : Chemical analysis of the Shahid Rajai aggregates

S.NO	L.I	SiO_2%	Fe_2O_3%	Al_2O_3%	MgO%	CaO%	So_3%	$C\bar{L}$%	$\frac{CaO}{MgO}$	Result
1	30.2	25.7	2.75	3.75	1.98	35.75	0.1	0.05	18.66	P.E
2	29	29.6	2.75	3.5	2.2	33	0.12	0.03	15	P.E
3	32.5	24.5	1.9	4.6	1.2	34.75	0.1	0.04	28.96	P.E
4	30.8	23.1	1.75	2.75	1.8	40.3	0.1	0.01	22.39	P.E
5	31.9	23.5	3	4.5	2.98	34	0.14	0.03	11.4	P.E

TABLE 4 : Chemical analysis of the Maroon aggregates

S.NO	L.I	SiO_2%	Fe_2O_3%	Al_2O_3%	MgO%	CaO%	So_3%	$C\bar{L}$%	$\frac{CaO}{MgO}$	Result
1	39.1	6.8	0.5	3	1.7	49	0.075	0.03	28.8	P.E
2	40.8	8.5	1.25	1.5	1.8	45.25	0.11	0.02	25.14	NON.E

TABLE 5 : Chemical analysis of the Karkheh aggregates

S.NO	L.I	SiO_2%	Fe_2O_3%	Al_2O_3%	MgO%	CaO%	So_3%	$C\bar{L}$%	$\frac{CaO}{MgO}$	Result
1	14	57	0.75	2.75	1.9	23	0.085	0.02	12.10	P.E
2	29.7	32	1.5	1.25	1.2	34.5	0.11	0.04	28.75	NON.E
3	28.1	31.3	0.9	2.35	1.9	35.75	0.09	0.03	18.5	P.E

TABLE 6 : Chemical analysis of the Ahar aggregates

S.NO	L.I	SiO_2%	Fe_2O_3%	Al_2O_3%	MgO%	CaO%	So_3%	$C\bar{L}$%	$\frac{CaO}{MgO}$	Result
1	8.7	56.4	5	13.25	3.17	12	0.08	0.02	3.8	P.E
2	17.7	39.8	5	13	3.08	21.75	0.09	0.016	7	P.E
1	6.2	54.1	8.5	19.5	2.89	8.85	0.075	0.018	2.96	P.E
2	9.7	49.1	6.75	19.25	3	11.75	0.074	0.018	3.92	P.E

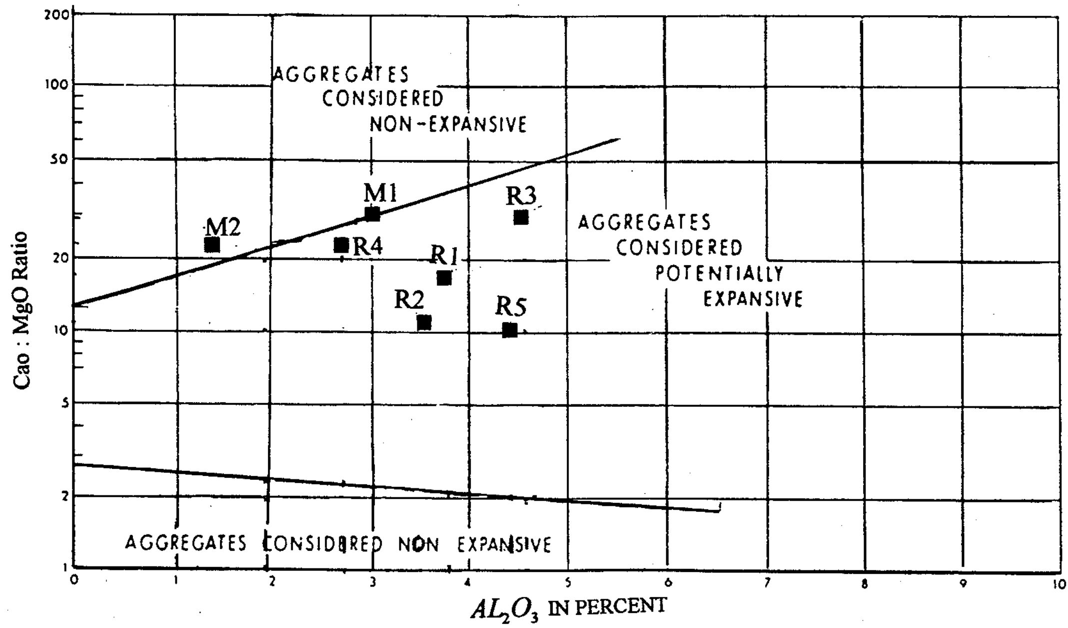

Fig2 : Results of the CSA A 23.2-26A chemical test

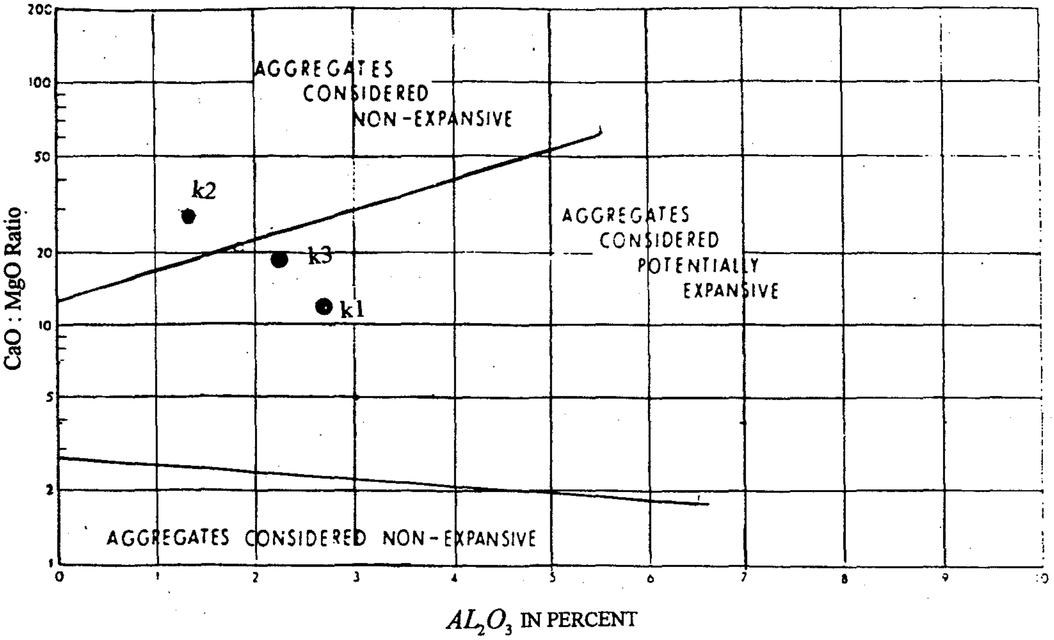

Fig3 : Results of the CSA A 23.2-26A chemical test

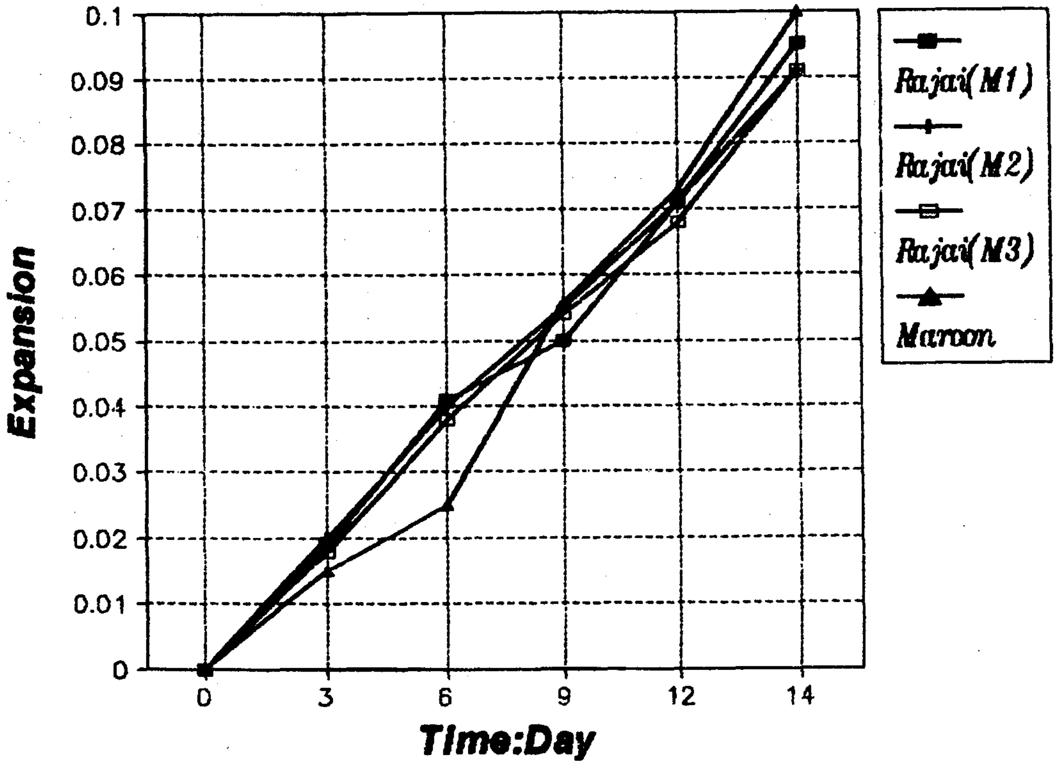

Fig4 : Results of the CSA accelerated Mortar bar testing of 1) shaid rajaii ; 2) Maroon aggregates According to CSA A23.2-25A.

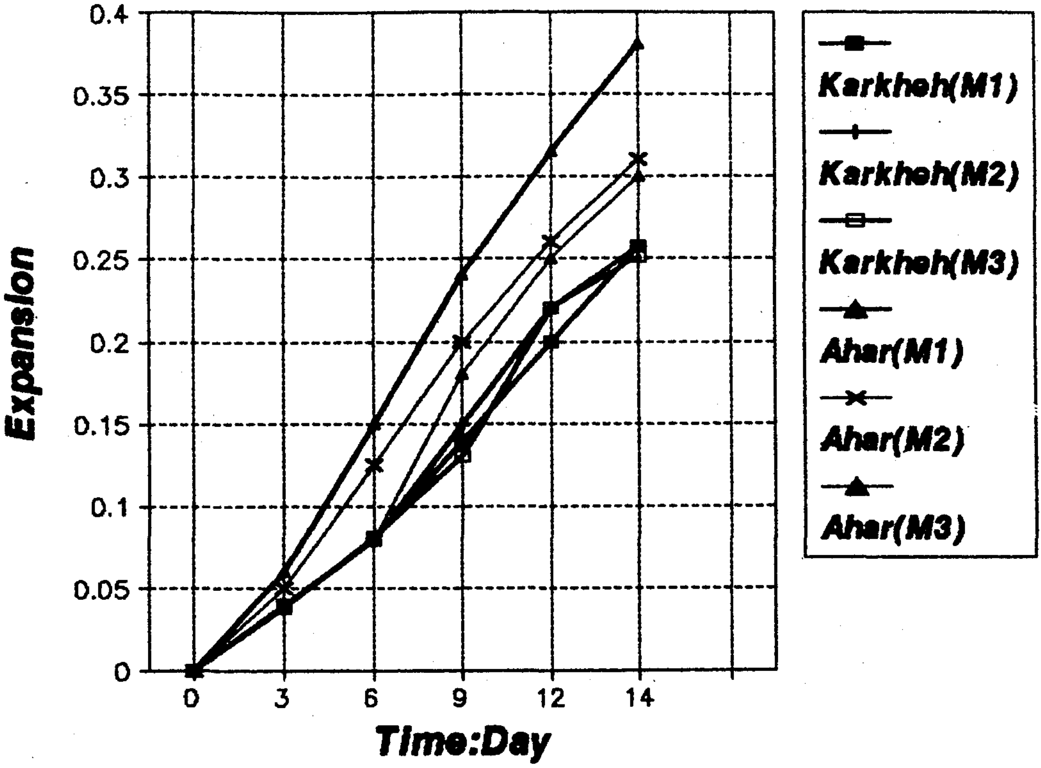

Fig5 :Results of the CSA accelerated Mortar bar testing of 3. Karkheh › 4. Ahar aggregates According to CSA A23.2-25A.

DISCUSSION

Petrographic examination :

Shahid Rajai dam : According to the test , the aggregate originate from a grey limestone deposit . The material is made up of fine particles which differs from massed to crystalline formations . They are composed of mostly calcite and lesser dolomite .

Maroon dam : The same properties as above have been obtained for Maroon aggregates and the existance of dolomite and calcite minerals in the rocks indicate the potential alkali-aggregate reactivity in the dam(ACR).

Karkheh dam : According to x-ray diffraction analysis the aggregates used in construction were made up of some quartz and lesser feldspar.

Ahar dam:The aggregate is composed of siliceous limestones, with mineral references components of (60% $CaCO_3$, 30-35% quartz and 10-12% feldspar and calcite) .

ASTM C 289 Chemical Test

From the results obtained for different aggregates it can be seen that this test is not applicable for the carbonate rocks and therefore this test was carried out for comparison purposes. The results of the tests show that the Rajai aggregates can be considered innocuous (see Fig 1) .

CSA A 23.2-26A Chemical Method

In this study each series of aggregates was tested and the results show that most of aggregates can be considered " potentially expansive " . These results shoud be confirmed either by satisfactory service record or concrete prism test.

CSA A 23.2-25A Mortar Bar Method

Results of the CSA accelerated mortar bar test showed that the aggregates from Rajai and Maroon expanded less than 0.1% at 14 days , which was the proposed expansion limit .
The results of other aggregtes such as Karkheh and Ahar showed that expansion of mortar bars were greater than 0.1% at 14 days . Comparison of the results of mortar bar tests with the results of chemical tests and petrographic examination reveals that these aggregates contain reactive silica and reactive carbonate phases . The concrete prism test should be carried out to confirm these results .

CONCLUSION

1- Most of the aggregates used for dams in Iran are carbonate aggregates. Therefore the chemical tests for alkali silica reactivity are not suitable tests for these aggregates.
2- The results of a modified chemical test show that most of the aggregates investigated were reactive aggregates , Long term concrete prism tests are needed to confirm the results of the chemical test .
3- The aggregates with high reactive silica contents showed higher expansion in both CSA A23.2-25A and CSA A23.2-26A tests revealing the fact that the ASR and ACR tests are necessary tests for the aggregate investigated in this study .
4- The results of the accelerated mortar bar test show that some aggregates were very reactive . The expansion of the bars was about three times the permissible value .

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IMMERSION TEST TO IDENTIFY CEMENT ALKALI LEVELS AND POZZOLANS TO PREVENT ASR

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ABSTRACT

ASTM C 227 (mortar-bar), ASTM C 289 (quick chemical), and ASTM C 1260 (rapid immersion) tests are either too lenient or too severe, and do not permit determining the minimum requirements for safe cement alkali levels or pozzolans for specific field installations. Experimental work in the Strategic Highway Research Program (SHRP), conducted in the United States during 1988 to 1993, indicated that the C 1260 procedure, adapted from that developed by Oberholster and Davies in South Africa, lends itself to identifying these requirements using certain modifications. This includes adjusting the molarity of the sodium hydroxide immersion solution to correspond to specific cement alkali levels and water-cement ratios, and making adjustments in the failure criterion. Based on a comprehensive literature review, an equation was developed to calculate the appropriate solution molarity for a given cement alkali level. Based on field performance, a projected adjustment of the failure criterion was made. This use of the procedure is still under refinement and evaluation. Test data are provided to illustrate the application of this approach to preventing expansive alkali-silica reactivity in concrete.
Keywords: alkali-silica reactivity test, cement alkali level, fly ash

INTRODUCTION

A primary objective of tests to identify susceptibility to expansive alkali-silica reactivity (ASR) is the selection of innocuous cement-aggregate combinations for field use. Predictions based on current long-standing ASTM tests have been found, in many field installations, to be erroneous, particularly for so-called slowly reacting aggregates and the use of low alkali portland cements (alkali contents less than 0.60% as equivalent Na_2O). In general, these tests have been too lenient. The development of the rapid immersion test at the National Building Research Institute (NBRI) in South Africa (Oberholster and Davies, 1986) has become widely accepted as a rapid and more appropriate test, and has been adopted in slightly modified form as ASTM C 1260, "Standard Test for the Rapid Identification of Alkali-Reactive Aggregates (Mortar Bar Method)." As it now stands, the test appears to be too severe in that it may identify an aggregate to be potentially deleteriously reactive while, in field structures, it has performed satisfactorily for many years even with the use of high alkali cements. With some modifications, it appears that the procedure could identify safe cement alkali levels more in accordance with field performance, and also identify the requirements of mineral admixtures to prevent deleterious ASR. Details of these modifications, and experimental results are presented below.

EXPERIMENT BASIS

It is generally agreed that higher cement alkali levels lead to earlier and more severe ASR and attendant distress due to expansion in a given exposure environment than lower alkali levels. The importance of cement alkali level lies in the resulting alkalinity of the pore solution in the concrete. In a review of the published literature, a linear regression equation was developed that related hydroxyl ion solution molarity to cement alkali level for different water-cement ratios (Helmuth, 1993), as follows:

$$[OH^-] = 0.339\frac{Na_2O}{w/c} + 0.022 \pm 0.06 \text{ moles/liter} \qquad (1)$$

where:

$[OH^-]$ corresponds to NaOH molarity
Na_2O = (equivalent) percent Na_2O of the cement.
w/c = water-cement ratio of the concrete

Using this equation, Figure 1 illustrates in graphic form relationships among the three variables. The NaOH molarity values plotted were calculated using the scatter in data on the high side (+0.06 moles/L in the equation) of the average. This provides somewhat conservative maximum cement alkali contents that would be required for safe levels of expansion.

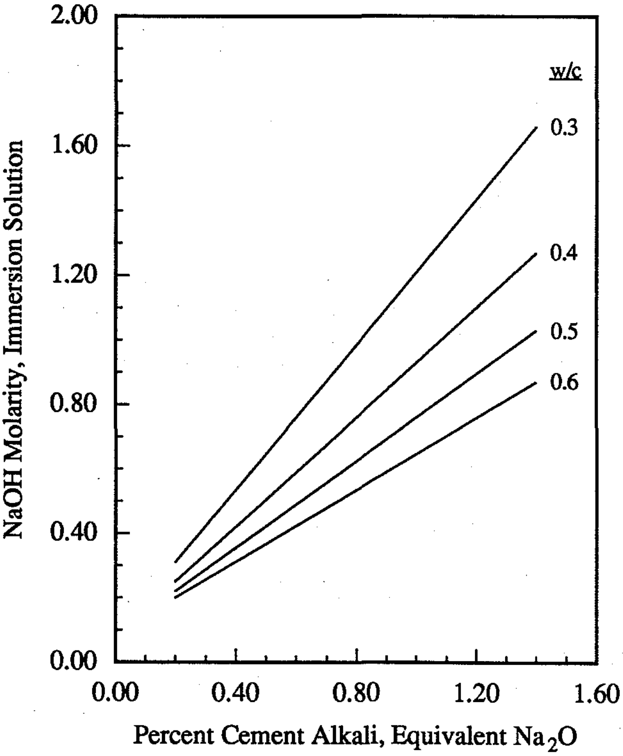

Figure 1 Relationship between cement alkali level and NaOH concentration of immersion solution for different w/c ratios

For example, for a w/c of 0.50, a 1.0 M NaOH solution corresponds to a cement alkali level of about 1.40%. This is the alkali concentration used in the NBRI and ASTM C 1260 test procedures. Since few cements, probably none in the United States, possess alkali levels of this magnitude, it is evident that the test solution is of greater alkalinity than is liable to be encountered in field concretes at 0.50 w/c. This in turn relates to the overly severe environment in the test procedure compared with the field and the tendency to find otherwise innocuous aggregates to be potentially deleteriously reactive. That is, field performance for the given aggregate is based on pore solutions of lower alkalinity because cement alkali levels do not reach 1.40% equivalent Na_2O. Thus, application of the test to determine more realistically the maximum safe cement alkali level can be approximated by adjusting the molarity of the immersion solution as indicated in the equation.

Another consideration, in field practice, is that the water-cement ratio may be of a lower value, such as 0.40, which is more characteristic of a structural concrete and corresponds to a cement alkali level of about 1.10%. However, a fixed water-cement ratio of 0.50 is essential in the immersion test because ASR depends upon diffusion of NaOH solution into the test specimens, and permeability of the mortar bar specimens under this test condition is strongly dependent upon water-cement ratio. Additionally, the question of cement content may be raised. The immersion test mortar bars are made at a fixed aggregate-cement ratio of 2.25:1.00, which is probably equivalent to a cement content in excess of 450 kg/m^3 in concrete. At such high cement contents in concrete, the additional mitigating factors of increased strength and creep on ASR expansion potential should also be considered.

The evaluation of mineral admixtures using this immersion test also was first carried out by NBRI (Davies and Oberholster, 1987) and later by CSIRO (Shayan, 1990) and in the Strategic Highway Research Program (Stark, Morgan, Okamoto, and Diamond, 1993). In these investigations, it was concluded that the procedure would be useful in evaluating mineral admixtures, specifically fly ash and ground granulated blast furnace slag, for capability to prevent expansive ASR. Test results are presented in this paper to support this conclusion.

IDENTIFYING SAFE CEMENT ALKALI LEVELS

It is evident that, to identify maximum safe cement alkali levels, the 1.0 M NaOH solution concentration must be reduced to correspond to specific cement alkali levels at a given w/c. This need is due to the fact that the NaOH concentration is raised up to one molar to accelerate the reaction and to properly classify the slowly reacting aggregates. In determining a safe alkali level the test would be attempting to identify a specific cement alkali level for a specific aggregate regardless of relative susceptibility to the reaction. For example, it has been found that the 0.080% test criterion is the minimum expansion obtained with slowly reactive aggregates and the maximum obtained for innocuous aggregates at a 1.40% equivalent Na_2O of the cement (Stark, Morgan, Okamoto, and Diamond, 1993). Also, since field observations for the aggregates under study displayed evidence of expansive ASR at cement alkali levels less than 0.95% equivalent Na_2O, a failure criterion would have to be lower at 14 days for an immersion solution of 0.67M NaOH, which corresponds to 0.95% equivalent Na_2O of the cement, per the linear regression equation.

A number of aggregates were evaluated according to this approach, using four solution molarities to represent a range of cement alkali levels. All mortar bars were made using an ASTM Type I cement (8% C_3A) with 0.18% equivalent Na_2O, and a w/c of 0.50. Although alkali content of the mortar bar cement is generally believed to have little, if any, measurable effect on expansion, a very low alkali cement was used to further assure that virtually no alkali in addition to that provided by the immersion solution could bias the test results. Aside from the use of the 0.50 w/c and the adjusted immersion solutions, mortar bar casting and test storage were carried out in accordance with ASTM C 1260.

Aggregate composition, cement alkali level for deleterious ASR in field concrete, cement alkali level equivalent in the immersion solution, and 14-day expansion values for C 1260 testing conditions (1.0 M NaOH solution at 80° C) are given in Table 1. All results are averages for four companion mortar bars. These data reveal a progressive reduction in expansion with reduction in solution molarity (cement alkali equivalent), as would be expected. Even for aggregates not associated with ASR, expansion decreased with cement alkali equivalent but, in these cases, from an already innocuous expansion level. It should be noted that the cement alkali equivalents of the immersion solutions were calculated on the basis of the worst case for w/c of 0.50.

To determine the safe cement alkali level for an aggregate by this procedure, the expansion criterion must decrease with reduction in NaOH solution concentration as discussed previously. A plot of results from the present tests is given in Figure 2. The projected failure criterion (dashed line) is shown as being 0.08% at 1.0 M concentration. The criterion is subsequently shown to decrease at a rate dictated by the coincidence of the

Table 1 Results of tests to identify maximum safe cement alkali levels

Aggregate rock type	% cement alkali reported for deleterious ASR	Cement alkali equivalent, %, in immersion solution	14-day expansion, percent
Crushed gabbro	No ASR reported	1.40	0.026
		1.00	0.014
		0.75	0.011
		0.50	0.002
		0.24	0.002
Crushed dolomite	No ASR reported	1.40	0.023
		1.00	0.014
		0.75	0.011
		0.50	0.002
		0.25	0.006
Crushed granite gneiss	0.85 to 0.95	1.40	0.086
		1.00	0.042
		0.75	0.009
		0.50	0.004
		0.25	0.006
Mixed siliceous reactive chert and sandstone	0.95 to 1.05	1.40	0.272
		1.00	0.214
		0.75	0.040
		0.50	0.004
		0.24	0.005
Crushed metabasalt	0.80 to 0.90	1.40	0.096
		1.00	0.060
		0.75	0.032
		0.50	0.016
		0.25	0.010

curves for the gabbro, dolomite, and granite gneiss and the fact that the line falls below the field failure indicated for the granite gneiss. This convergence point should insure lack of ASR because both the gabbro and dolomite have known satisfactory service records. Therefore, data plotted for 1.40% cement alkali are results for 1.0 M NaOH concentration, as normally used. With reduction in NaOH concentration in the test, safe expansions are reduced to a maximum of 0.030 to 0.040% at 14 days. Any point below this line for an aggregate would represent a safe cement alkali level, while points above this criterion represent potential excessive expansion. Figure 2 indicates, for example, that any cement alkali level up to 1.40% is safe for the innocuous gabbro and dolomite aggregates. For the granite gneiss, which reacted deleteriously in the field at 0.85% to 0.95% alkali, the safe level would be about 0.70% alkali.

At the present time, further data are being developed to better define maximum safe cement alkali levels by this method before this approach can be used commercially.

IDENTIFYING MINERAL ADMIXTURES FOR PREVENTING EXPANSIVE ASR

As stated earlier, the rapid immersion test procedure appears to be adaptable to evaluating mineral admixtures for preventing expansion due to ASR. Studies were carried out in the present program to substantiate those findings.

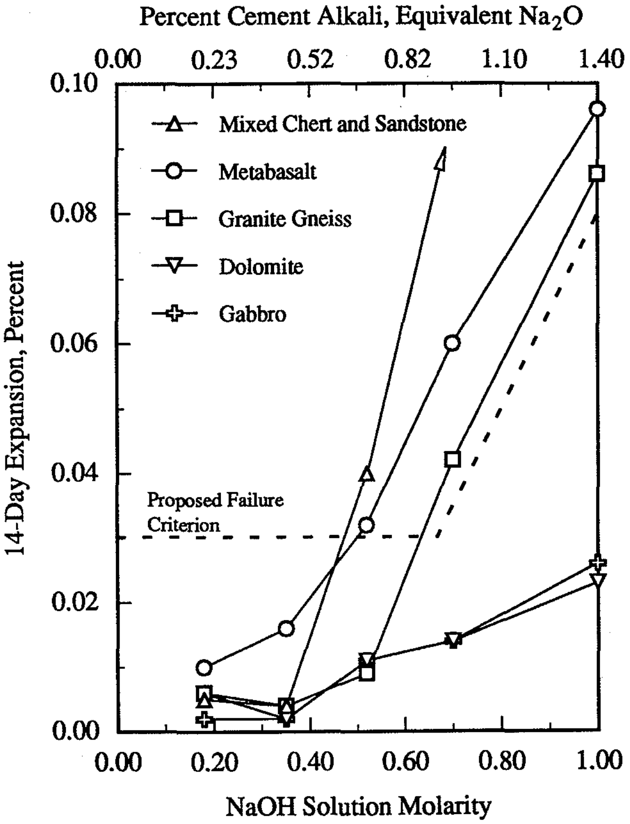

Figure 2 Projected failure criterion to determine safe cement alkali level for deleterious aggregates, using the rapid immersion test procedure with mortar bars at 0.50 water-cement ratio.

In one investigation, three fly ashes were evaluated in field pavement test sections and in the rapid immersion test as 20 to 25% mass replacements of an ASTM Type II portland cement with 0.50 to 0.60% equivalent Na_2O. One was an ASTM Class F fly ash with approximately 5% CaO. A second was a high-lime ASTM Class C fly ash with approximately 25% CaO, while a third was a 50:50 blend of the same high-lime fly ash and a second low-lime Class F fly ash from a different source. A highly reactive natural sand and gravel from a single source was used in both the pavement and rapid immersion tests.

Observation of the pavement after three years of service revealed the early development of longitudinal and map cracking characteristic of that due to ASR in pavements. The only test sections displaying such distress were those containing the high-lime Class C fly ash and the control sections with no fly ash. The sections containing the low-lime Class F fly ash or the fly ash blend were in excellent condition.

Results of the rapid immersion tests paralleled those noted in the test pavement. That is, control mixtures with no fly ash, and the mixture containing the high-lime Class C fly ash developed by far the greatest expansions. Mixtures containing the fly ash blend or the Class F fly ash developed little expansion, with the latter remaining below the test criterion of 0.08% at 14 days. These results thus indicate similar relative performance for the test pavement sections and the rapid immersion tests.

A second series of rapid immersion tests was run using known reactive aggregate from three sources together with two Class F fly ashes and one Class C fly ash. Characteristics of these fly ashes and the ASTM Type I portland cement used are given in Table 2.

Table 2 Composition of fly ashes and portland cement

	Percent			
Analyte	Fly Ash No. 1	Fly Ash No. 2	Fly Ash No. 3	Portland Cement
SiO_2	51.40	48.71	33.00	21.22
Al_2O_3	18.50	19.98	18.90	4.86
Fe_2O_3	16.10	15.26	5.97	3.39
Total	86.00	83.95	57.87	—
CaO	4.49	5.67	27.00	65.25
MgO	1.06	1.31	5.28	0.52
S (sulfide)	1.43	0.67	2.60	2.38(SO_3)
Total Na_2O	0.84	0.73	1.98	0.02
Total K_2O	2.34	2.53	0.39	0.25
Total Equivalent as Na_2O	2.38	2.39	2.25	0.18

The fly ashes were used in proportions up to 60% replacement of cement by mass. The quarried volcanics (metabasalt) aggregate with greatest expansion with no fly ash is from the same source as used in the test pavement. The source showing the least expansion without fly ash is a quarried granite gneiss, while the aggregate with an intermediate expansion is a chert-quartzite gravel. The cement used in this series contained 0.19% alkali as equivalent Na_2O.

Results in this test series are given in Table 3 where 14-day expansions are given for different cement replacement levels for the three fly ashes used with these aggregates. The data indicate that the two Class F fly ashes reduced expansions to below the failure criterion when used at the 20 to 30% cement replacement level, while the Class C fly ash failed to do so even when used at up to the 60% cement replacement level. Also, it is seen that the required proportion of fly ash to reduce expansions to below the test criterion varied somewhat with aggregate source.

Most importantly, the data show that reductions in expansion to below 0.10% were achieved only by the two ASTM Class F fly ashes. In an unpublished report, the writer obtained similar results in which Class F fly ashes at 20 to 30% levels of replacement for cement reduced expansions to below the test criterion, while Class C high-lime fly ashes could not achieve this level of reduced expansions.

The writer believes that differences in performance are related to lime-silica ratios of the calcium silicate hydration products of the cement and fly ash (Bhatty, 1985).

CONCLUSIONS

Results reported herein suggest that, with further refinement, the NBRI rapid immersion test can be used not only to determine whether an aggregate is potentially deleteriously reactive, but also to identify maximum safe cement alkali levels. Also, it appears that the procedure may be able to identify the capability of mineral admixtures to prevent excessive expansion due to ASR with respect to the proportion of fly ash required to do so with a particular aggregate and cement alkali level. Such use appears to require adjustments in either or both the failure criterion and the NaOH concentration of the immersion solution.

Table 3 Results of rapid immersion tests to evaluate fly ashes

Fly ash		14-day expansion (%) Reactive Aggregate Component		
Source, ASTM class	Cement repl. level (%)	Volcanics	Granite gneiss	Chert, quartzite
None	0	0.867	0.309	0.409
No. 1, Class F	10	0.489	0.231	0.146
	20	0.054	0.064	0.000
	30	-0.001	0.008	-0.006
No. 2, Class F	10	0.623	0.247	0.232
	20	0.102	0.111	0.028
	30	—	0.040	0.014
No. 3, Class C	30	0.771	0.231	0.310
	45	0.510	0.178	0.230
	60	0.535	0.170	0.125

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Multi-Laboratory Study of the Accelerated Mortar Bar Test for Alkali-Silica Reaction

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ABSTRACT

The accelerated mortar bar test, developed by NBRI in South Africa, has been adopted in North America for the rapid identification of potentially alkali-silica reactive aggregates. The purpose of the study, involving 46 laboratories, was to obtain data to develop a multi-laboratory precision statement for the test. The aggregate used in this study was Spratt siliceous limestone from a quarry in Ottawa, Ontario. At an age in solution of 14 days, the mean expansion was found to be 0.364% (coefficient of variation 13%) when laboratories used the same cement, and 0.417% (coefficient of variation 15%) when laboratories used their own cement. Autoclave expansion, and the alkali, sulphate and MgO contents of cement were not related to mortar bar expansion. A relationship was found between MgO content and autoclave expansion.

Keywords: mortar bar test, alkali-silica reaction, precision, cement, chemistry.

INTRODUCTION

Alkali-silica reactions between silica from aggregates and alkalies from cement can cause damaging expansion and cracking of concrete. These reactions need three things to occur: concrete that is sufficiently moist, reactive aggregate and sufficient alkalies in the concrete to start and support the reaction.

One of the most important tests to detect alkali-silica reactive aggregates is the mortar bar expansion test. This test was developed in the early 1940's by Stanton [1943] in California and was adopted by ASTM as a tentative test method in the early 1950's (ASTM C227). The test involves making cement mortar bars with a suspect aggregate and storing the bars at 38°C over water for a year or more. Excessive expansion of the bars indicates a potentially deleterious, expansive aggregate cement combination.

The test has often been found to be impractical because of the length of time before results are obtained. Concrete is often placed and in service before results are available showing compliance of the aggregates with the specification. In the late 1980's, it was also found that wide variations in the test could be caused by leaching of the alkalies out of the mortar bars during storage [Rogers and Hooton 1991]. This was traced to the type of container used for storing the mortar bars. Different storage containers, meeting the specification, could make a test furnish results either meeting or failing the specification requirements.

In the early 1980's, Oberholster and Davies [1986], at NBRI, developed a means of accelerating the expansion of the mortar bars by storage in hot NaOH solution. It is worth noting however, that Stanton [1943] was the first to experiment with storage of bars in hot sodium hydroxide solution. In the NBRI test, the bars are de-moulded after one day and placed in water which is then heated to 80°C. After 24 hours, the bars are measured for length while at 80°C and immediately placed in a 1N NaOH solution preheated to 80°C. They are stored at 80°C in the solution and expansion is measured by quickly removing the bars from the solution, measuring length and returning the bars to the solution. Normally, expansion after 14 days storage in solution has been found to be a convenient time period to determine if the aggregate is deleteriously expansive. In some cases, other times (12 days, 28 days) have been adopted for setting specification limits.

Draft test procedures for this test were adopted by CSA in 1988 and by ASTM in 1989. This test was formally adopted by both CSA and ASTM in 1994 (CSA A23.2-

25A, ASTM C1260). The history of the adoption in North America will be found in a paper by Hooton and Rogers [1992].

A requirement of any satisfactory engineering materials test is that the test have a known and low multi-laboratory variation. Two multi-laboratory studies of the variability of the test have been conducted. Davies and Oberholster [1987], in a study within one laboratory with three different technicians, found a coefficient of variation of 10.4% (mean expansion 0.34%). Hooton [1990], in a study using the Spratt aggregate (used in this study) and nine laboratories, found an average multi-laboratory coefficient of variation of 22.5%. When he excluded three inexperienced labs, the variation was reduced to 9.4% (mean expansion 0.42%). These studies have been done with limited numbers of laboratories, and so do not give a true picture of likely multi-laboratory variation. Generally, to get stable precision estimates, the number of laboratories should be at least 30. A large number of laboratories are now doing this test in North America, and the CSA and ASTM sub-committees responsible for the test procedure decided to conduct a multi-laboratory study. Another reason for doing the study was to qualify a stockpile of alkali-silica reactive aggregate that could be used in the future as a reference material for control purposes within individual laboratories.

LABORATORY TESTING

Experimental Design

To ensure the maximum number of laboratories participated, it was decided to keep the study as simple as possible while still collecting data on multi-laboratory variation. Normally, multi-laboratory studies should be conducted on at least three different materials and two replicates of each material should be tested [ASTM C802]. It was decided not to attempt such an ambitious program. The test procedure used by the participants was that published by CSA. This procedure is identical in all technical respects to ASTM C 1260 except that the w/c ratio specified by CSA and used in this study was 0.50 rather than 0.47 specified by ASTM and there are different cement requirements. The design called for each laboratory to receive 6 kg of aggregate, enough to do at least two mortar mixes. In addition, each laboratory received 1 kg of cement from the organizing laboratory. The program called for each laboratory to make two mortar mixes, one with the standard cement and one with the cement normally used by that laboratory. This design would allow the estimation of multi-laboratory variation but would not allow estimates of within laboratory variation. Such a design would also show how cement properties affected test variability. Previous ruggedness testing had shown that cement properties, within broad limits, did not significantly influence expansion in the test. This data had been obtained with small numbers of different cements and there was speculation that more rigorously specifying the cement or the use of a standard cement would significantly reduce between laboratory variation.

Aggregate

The aggregate selected for the study was a siliceous limestone from the Spratt quarry near Ottawa, Ontario. This horizontally bedded limestone contained 3-4% microscopic chalcedony and black chert. The rock consisted of calcite and small amounts of dolomite with an insoluble residue of 10% consisting of silica, illite and pyrite. The principal reactive component is finely disseminated silica not visible with normal optical methods. The aggregate, one of the most expansive alkali-silica aggregates in Canada, has been used in previous studies of alkali-reaction test methods [Rogers and Hooton 1991]. For the purposes of this study, a new 100 tonne stockpile of 5-20 mm stone was established in Toronto. On delivery, Four samples (40kg each) were taken at random from the stockpile and tested in the accelerated mortar bar test. The results showed that the materials gave a range of expansion after 14 days in solution of 0.335-0.344% (average 0.339%). Following this testing, a sample of about 400 kg, representative of the stockpile, was oven dried and sieved. Each sieve fraction was placed in separate sample containers. Sixty samples of 6 kg each were made by combining aggregate of each sieve size according to the gradation of the material in the stockpile. These samples were stored in canvas bags.

Cement

The coordinating laboratory had a stockpile of about 2 tonnes of high alkali cement. Specimens of this cement were prepared by sequentially placing about 200 g of cement taken from one plastic barrel with a small scoop in each of 50 plastic bags with a self sealing closure. Additional 200 g aliquots were added until each bag contained about 1 kg of cement. The bags were sealed and placed inside another self sealing plastic bag.

Testing

Each laboratory received a specimen of cement, a sample of aggregate, a copy of the test procedure and forms for recording expansion data, equipment details and laboratory conditions. Each laboratory was asked to ship a 1 kg sample of the cement used to the coordinating laboratory. The coordinating laboratory tested each sample in the autoclave expansion test (ASTM C151) together with 11 specimens of a laboratory control cement. The Lafarge Canada, Montreal laboratory tested chemical composition using XRF (ASTM C114). The laboratories were asked to record mortar bar expansion at ages of 7, 13, 14, 15, 21 and 28 days in NaOH solution. They were asked to record average (3 bars) expansion to 0.0001%. This is more accurate than required by the test procedure, but was requested in order to avoid rounding errors influencing the estimate of precision.

RESULTS AND DISCUSSION

Statistical analysis

Expansion data was received from 46 laboratories. Data from five laboratories was excluded from the final analysis. Laboratory 10, which gave the highest expansion, was rejected because they noted that they mixed the mortar by hand. This probably resulted in inadequate mixing and poor consistency of the paste. Laboratory 19, which gave relatively low expansion, was rejected because they were not using mortar bars of the correct length (210 mm instead of 285 mm). Laboratory 27 was rejected because they reported very high expansion at 7 days and very little subsequent expansion between 7 and 28 days. The laboratory gave no reason and did not reply to enquiries about the problem. Laboratory 35 was rejected because they had not previously conducted the test and reported problems with their measuring device. Laboratory 36 was rejected because they used a RILEM mortar bar (40 x 40 x 160 mm) and obtained relatively low expansion. These were believed to be sound reasons to reject data. With the exception of Laboratory 27, no data was rejected just because 'it looked wrong' or because it exceeded three standard deviations of the mean.

Following the removal of outlying data, standard deviation and coefficient of variation was calculated at the various times of storage. Figure 1 shows the coefficient of variation at various ages before and after removal of outliers. Figure 2 shows the variation in the form of a scatter diagram. The scatter diagram shows that there were a number of laboratories that tended to get either consistently high results or consistently low results, in addition to those already rejected. These laboratories are ones which show some bias in their testing. There was however no obvious physical reason to reject their data and it has been included in the analysis.

It is possible to prepare a precision statement as follows: For mortars giving average expansions after 14 days in solution of more than 0.3%, the multi-laboratory coefficient of variation (1s% of ASTM C 670) has been found to be 14.9%. Therefore, the results of two properly conducted tests in different laboratories on specimens of a sample of aggregate should not differ by more than 42% (d2s% of ASTM C670) of the mean expansion.

Table 1. Summary of multi-laboratory test information before and after removal of outlying data.

standard cement, all data included						**n = 46**
age	7 day, %	13 day, %	14 day, %	15 day, %	21 day, %	28 day, %
mean exp.	0.2326	0.3477	0.3653	0.3860	0.4943	0.6533
max exp.	0.5563	0.5652	0.5663	0.5827	0.8233	0.9767
min exp.	0.1455	0.2155	0.2310	0.2435	0.3406	0.4570
Std. Dev.	0.0624	0.0663	0.0658	0.0714	0.0899	0.1096
C. of V.	26.8%	19.1%	18.0%	18.5%	18.2%	16.8%
standard cement, five laboratories excluded						**n = 41**
age	7 day, %	13 day, %	14 day, %	15 day, %	21 day, %	28 day, %
mean exp.	0.2290	0.3459	0.3641	0.3850	0.4919	0.6549
max. exp.	0.3493	0.4887	0.5003	0.5190	0.6837	0.8804
min. exp.	0.1455	0.2673	0.2936	0.3088	0.3693	0.4752
Std. Dev.	0.0369	0.0477	0.0483	0.0526	0.0725	0.0968
C. of V.	16.1%	13.8%	13.3%	13.7%	14.7%	14.8%
individual laboratories cement, all data included						**n = 46**
age	7 day, %	13 day, %	14 day, %	15 day, %	21 day, %	28 day, %
mean exp.	0.2703	0.3990	0.4166	0.4367	0.5368	0.6830
max exp.	0.5646	0.5990	0.6367	0.7027	0.9647	1.2970
min exp.	0.1413	0.2426	0.2546	0.2768	0.3810	0.4736
Std. Dev.	0.0698	0.0763	0.0779	0.0844	0.1070	0.1471
C. of V.	25.8%	19.1%	18.7%	19.3%	19.9%	21.6%
individual laboratories cement, five laboratories excluded						**n = 41**
age	7 day, %	13 day, %	14 day, %	15 day, %	21 day, %	28 day, %
mean exp.	0.2676	0.3981	0.4159	0.4356	0.5318	0.6756
max. exp.	0.3787	0.5360	0.5493	0.5740	0.7783	0.9773
min. exp.	0.1663	0.2912	0.3037	0.3166	0.3810	0.4736
Std. Dev.	0.0495	0.0602	0.0621	0.0660	0.0862	0.1138
C. of V.	18.5%	15.1%	14.9%	15.2%	16.2%	16.9%

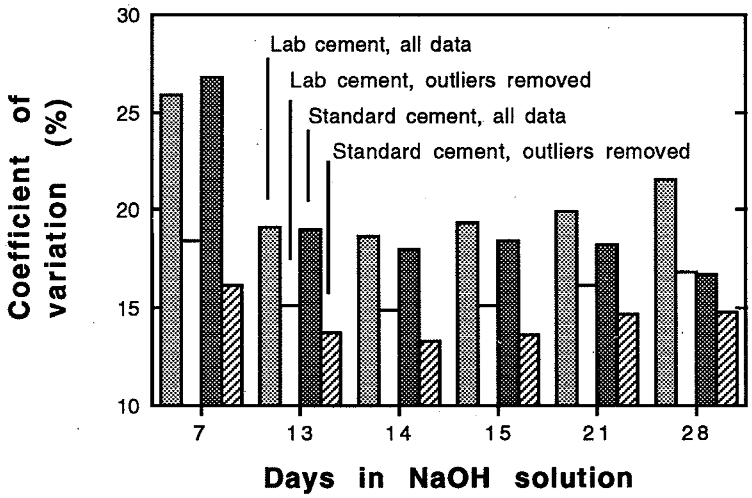

Fig. 1 Multi-laboratory coefficient of variation for accelerated mortar bar test at various ages.

Cement composition

To what extent does the composition of the cement effect the expansion measured by a laboratory? The data shows that with the cements studied, the chemistry of the cement

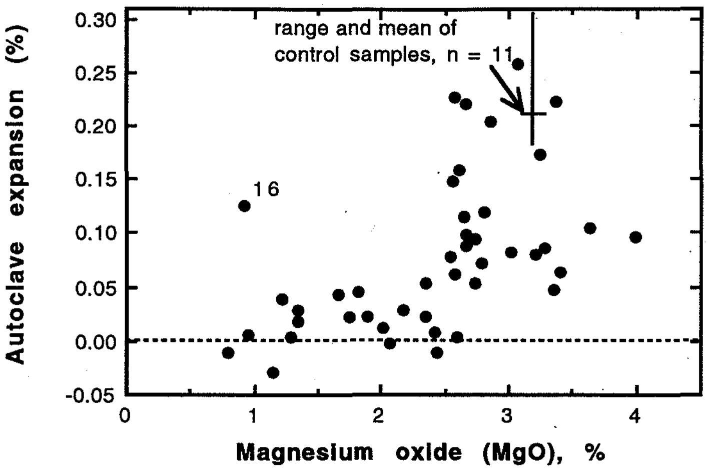

Fig. 4 Cement autoclave expansion, ASTM C151, compared with MgO content of the cement.

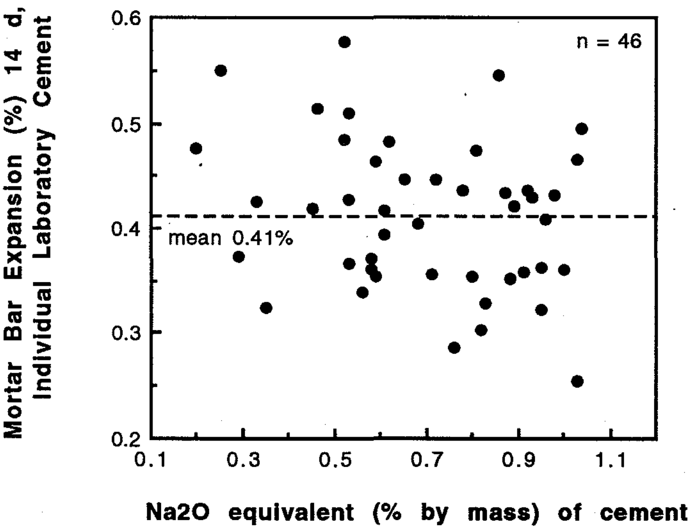

Fig. 5 Mortar bar expansion at 14 days in solution compared with alkali content of the cement.

Figure 4 does show that there is a relationship between MgO content and autoclave expansion in the ASTM C151 test. This is logical. With the exception of cement used by Laboratory 16, there is a threshold of MgO content at about 2.5% above which autoclave expansion may be significant and below which it is negligible. Lea [1971] noted that cement could accommodate about 1.5-2% MgO both in the glass phase of clinker and in solid solution with other cement minerals, and not cause a problem. Above this level of MgO, periclase would form and depending on crystal size, determined by cooling rate, cause unsoundness.

Figure 5 shows that there is no consistent relationship between alkali content of the cement and expansion. This confirms the observations of Davies and Oberholster [1987] and Hooton [1990]. Figure 6 shows that the sulphate in the cement (measured as total SO_3) is not related to expansion of the mortar bars. It can be concluded that the

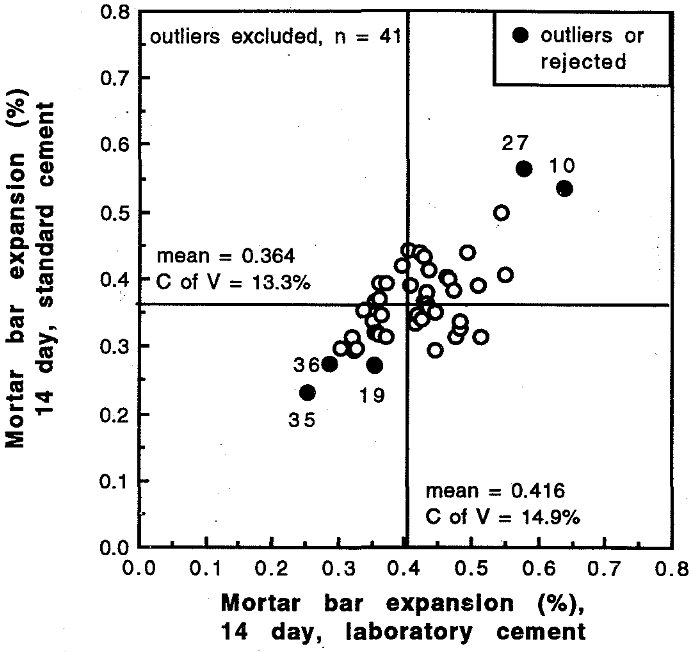

Fig. 2 Scatter diagram showing multi-laboratory variation of accelerated mortar bar test made with individual laboratories cement and standard cement after 14 days in solution.

had little systematic effect on expansion. The standard cement gave average expansion results of 0.364% at 14 days that were about 12% less than the average expansion when different cements were used. The variation was also slightly, but consistently, lower using the same cement compared with different cements as shown in Figure 1.

Chemical analysis of the cements used by the individual laboratories was compared with mortar bar expansion. Figure 3 shows the relationship between MgO content and expansion of mortar bars at 28 days. No obvious correlation is seen. It had been suspected that cements with high periclase content, measured by MgO might contribute to expansion, especially in view of the high temperatures of storage in the test (80°C).

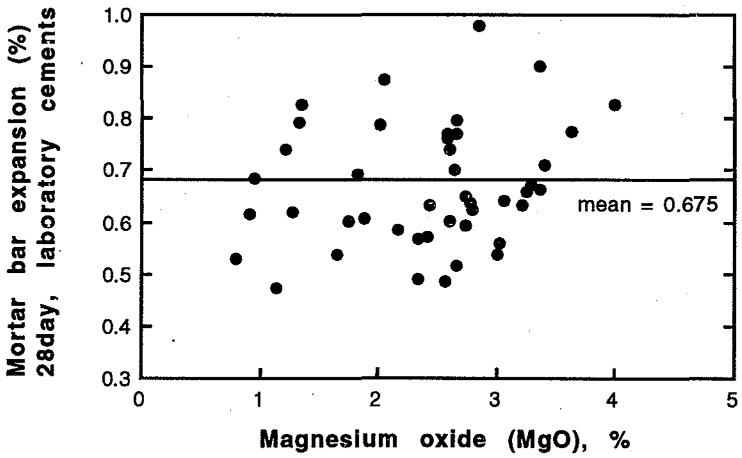

Fig. 3 Mortar bar expansion at 28 days in solution compared with MgO content of the cement.

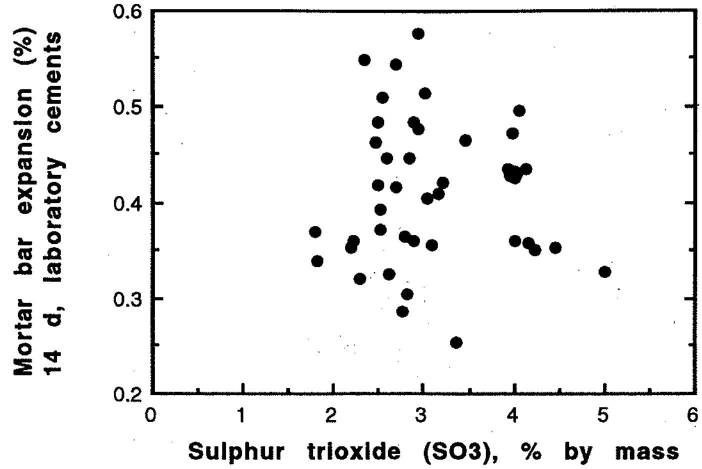

Fig. 6 Mortar bar expansion at 14 days in solution compared with the SO_3 content of the cement.

chemistry of the cements used in this study did not have a major influence on expansion of the mortar bars. It is possible that variation caused by cement was masked by test variability. A specific cement may have an influence on the expansion in this test. In the future it would be good practice to qualify a cement with the aggregate used in this study, before embarking on an aggregate testing program.

CONCLUSIONS AND RECOMMENDATIONS

In a study involving 46 laboratories, it was found that the multi-laboratory coefficient of variation after 14 days in solution was 13.3% when the same cement was used and 14.9% when each laboratory chose a different cement. Precision can be stated as follows: For mortars giving average expansions after 14 days in solution of more than 0.3%, the multi-laboratory coefficient of variation (1s% of ASTM C670) has been found to be 14.9%. Therefore, the results of two properly conducted tests in different laboratories on specimens of a sample of aggregate should not differ by more than 42% (d2s% of ASTM C670) of the mean expansion.

The use of a standard cement did reduce multi-laboratory variation by a small amount (11%). It is probably not worthwhile to establish a standard cement to ensure slightly better precision. At present the CSA version of the test method (A23.2-25A, 1994) calls for a cement with a total alkali content of 0.9 ± 0.10%. The ASTM version (C1260-94) calls for a cement with a autoclave expansion of less than 0.20%. In future, consideration should be given to allowing the use of any normal Portland cement provided it can be qualified using the aggregate used in this study. Mortar bars would have to give expansions between 0.329% and 0.504% which is the d2s% applied to the mean expansion of 0.417% at 14 days found when various cements were used.

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EXPANSION CHARACTERISTICS OF AAR IN CONCRETE BY AUTOCLAVE METHOD

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ABSTRACT

We selected the autoclave method to accelerate alkali aggregate reactions in concrete, and studied the influences of treatment pressure, treatment time, and alkali content on the alkali aggregate reactions in concrete. As a result, it was found that these factors exhibit a pessimum value at the point of maximum expansion. We determined that when alkali aggregate reactions in concrete are accelerated by the autoclave curing method, an alkali content of a certain or greater level is required.

Through this experiment, we determined that expansion of concrete containing 2.5-3.0% alkali and treated in an autoclave at a 0.2 MPa for 4-8 hours is almost equal to that of concrete at the age of 6 months which has been accelerated at 40°C and a relative humidity of 100%.

Key words: Acceleration testing, alkali aggregate reactions (AAR), autoclave, pessimum value, residual expansion

INTRODUCTION

Alkali aggregate reactions (AAR) take place over a long time and are one source of damage to structural members of concrete which seriously affect the durability and load resistance of concrete structures.

The chemical method, mortar bar method, and concrete method have been established and proposed as methods for AAR. Among them, the concrete method is considered to be effective in judging AAR because it enables used concrete to be examined and the AAR can be judged with greater precision. However, even though the concrete method is an acceleration AAR method, it has the drawback of requiring at least half a year before results can be gathered.

In this study we considered the use of an autoclave to cure concrete at high temperatures and pressures as our testing method for AAR in early stage concrete. Firstly, we wished to understand the expansion characteristics of AAR in concrete in an autoclave. In addition, we studied the residual expansion when concrete was cured under conditions of 40°C and R.H. of 100% following its removal from the autoclave.

OUTLINE OF THE EXPERIMENT

Table 1 shows the material used in this experiment and the test conditions.

The following aggregates were used: non-reactive aggregate, bronzite andesite fine aggregate, and non-reactive fine aggregate for comparison. The bronzite andesite has caused damage in actual concrete structures and its reactivity has been determined using the JCI chemical method, mortar bar method and concrete method. First-class reagent NaOH was used to adjust the alkali content in the cement to an equivalent Na_2O content of 1.0, 1.5, 2.0, 2.5, 3.0, and 4.0%.

The treatment pressures and times used in the autoclave were set at 5 levels in the range from 0.10 to 0.30 MPa and in the range from 1 to 8 hours. The length changes in the specimens immediately after the autoclave treatment were measured. The residual expansion was measured every half month until the age of 3 months following the autoclave treatment while the specimens were stored in a curing tank at 40°C and a R.H. of 100%, and then measured every month after the age of 3 months. In addition, specimens for comparison at each alkali content were cured in a curing tank at 40°C and R. H. of 100% and measured for length change. This measurement was made after the concrete specimens were taken out of the curing tank and moved to a constant temperature room and the specimen temperature had decreased to 20°C.

Table 1 Materials and testing conditions

Cement		Ordinary portland cement (Na_2O eq. 0.33%)
Coarse aggregate		Non-reactive (Sp.Gr.=2.70, W.A.=0.65%, FM=6.64)
Fine aggregate		Reactive (Sp.Gr.=2.64, W.A.=1.48%, FM=2.61) Rc=67.5, Sc=301, Sc/Rc=4.46 Non-reactive (Sp.Gr.=2.66, W.A.=1.15%, FM=2.81)
Type of alkali added		NaOH
Total alkali content (Na_2O eq. %)		1.0, 1.5, 2.0, 2.5, 3.0, 4.0,
Mix proportion		W/C=0.54, s/a=43%
Dimensions of specimen		7.5 x 7.5 x 40.0 (cm)
Autoclave	Pressure (MPa)	0.10, 0.15, 0.20, 0.25, 0.30
	Period (Hour)	1, 2, 4, 6, 8,

TEST RESULTS AND CONSIDERATION

Expansion characteristics immediately after autoclave treatment

Influence of alkali content and treatment time

We studied the influence of alkali content and autoclave treatment time by on the expansion caused by AAR in concrete. We set the treatment pressure of the autoclave at 0.2 MPa.

Figure 1 shows the relationship between expansion and alkali content over time under a treatment pressure of 0.2 MPa. This figure indicates that when the alkali content is 2.5% or less, expansion increases as treatment time increases. However, when the alkali content exceeds 2.5%, the expansion rate of concrete treated for 6 and 8 hours decreases. The greatest expansion occurs when the alkali content is 3.0% and the treatment time is 4 hours.

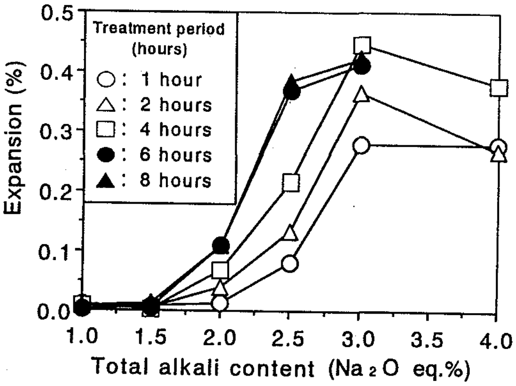

Fig-1 Relationship between expansion immediately after autoclave treatment and alkali content (Treatment pressure: 0.2 MPa)

We considered that a 3.0% alkali content is a pessimum value when the treatment pressure is 0.2 MPa.

The expansion volume due to AAR is controlled by the volume, viscosity, and stiffness of the gel produced by the reactions. The volume of the gel increases as the surface area of the reactive aggregate and alkali content increase. In addition, the viscosity and stiffness of the gel varies with its chemical composition. As the alkali content per reactive aggregate increases, the production volume of the gel increases, accompanying the increase in expansion. However, when there is a large amount of alkali, the reaction rapidly proceeds, but the ratio of SiO_2/Na_2O in the gel produced decreases and the gel softens at an early stage, therefore the expansion pressure decreases [T. Ming-Shu 1983, C. Chatterji 1978]. When the volume of reactive aggregate is equal, the rate of expansion increases as the alkali content increases to a certain value. However, if the alkali content exceeds a certain value, expansion pressure decreases due to softening of the gel, and expansion decreases. It is considered that this value is the pessimum value for alkali content, as shown in Figure 1.

Based on these observations, when the treatment pressure of the autoclave is fixed at 0.2 MPa, the expansion reaches its maximum value at an alkali content of 3.0% and a treatment time of 4 hours.

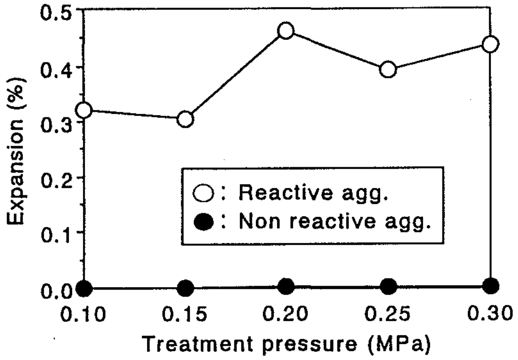

Fig-2 Relationship between expansion immediately after autoclave treatment and treatment pressure (Alkali content: 3.0%, treatment period: 4 hours)

Influence of treatment pressure

We studied the influence of treatment pressure on AAR in concrete with an alkali content of 3.0%, and a treatment time of 4 hours. Figure 2 shows the relationship between the expansion and treatment pressure for concrete using reactive fine aggregate and for concrete using only non-reactive aggregates at a treatment time of 4 hours. This figure shows that maximum expansion appears when the treatment pressure is 0.2 MPa. Concrete containing only non-reactive aggregate does not exhibit any expansion at any treatment pressure. Therefore, the expansion in concrete containing reactive fine aggregate is considered to be due to AAR.

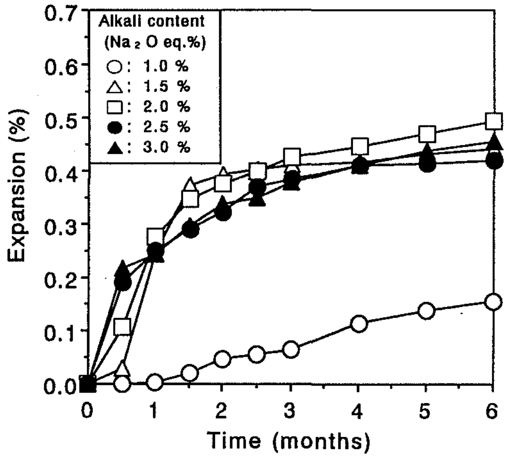

Fig-3 Expansion over time (Control specimen)

Residual expansion

Expansion characteristics of the control specimen

Figure 3 shows the change in expansion over time for concrete specimens containing various alkali contents without autoclave treatment which were subsequently stored in an accelerating environment at 40°C

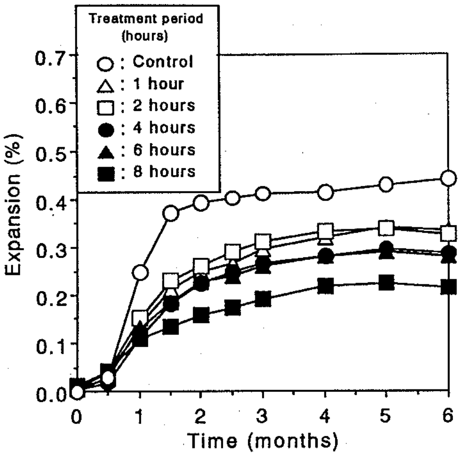

Fig-4 Expansion over time (Alkali content: 1.5%, treatment pressure: 0.2 MPa)

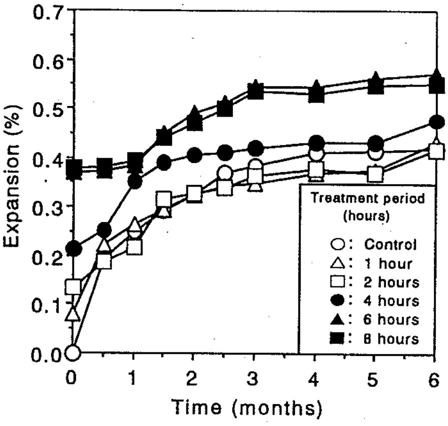

Fig-5 Expansion over time (Alkali content: 2.5%, treatment pressure: 0.2 MPa)

and 100% R.H. The figure illustrates that the specimens with a 1.0% alkali content started expanding at the age of 1 month. After that, the rate of expansion slowly increased to 0.15% at the age of 6 months. The expansion rate of the specimens with a 1.5–3.0% alkali content rapidly increased at an early age, and after that, slowly increased up to the age of 6 months. The specimens with a 2.0% alkali content showed the greatest expansion. As mentioned in *Influence of alkali content and treatment time*, it is considered that when a large amount of alkali exists, the ratio of SiO_2/Na_2O increases, the gel softens, the expansion pressure decreases, therefore, expansion is maximized at an alkali content of 2.0%; that is the pessimum value.

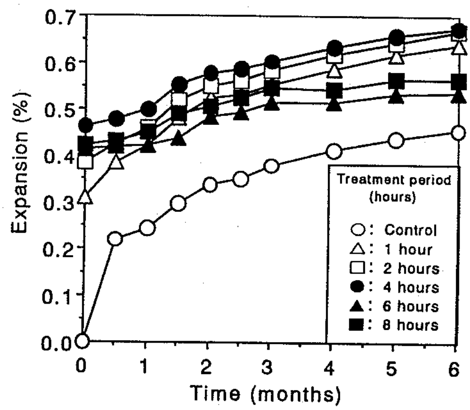

Fig-6 Expansion over time (Alkali content: 3.0%, treatment pressure: 0.2 MPa)

Residual expansion in concrete treated by autoclave curing

Figures 4 to 6 show the change in expansion over time of specimens with 0.5, 2.5, and 3.0% alkali contents respectively at a treatment pressure of 0.2 MPa, non–treatment and treatment time are indicated as parameters.

As for the specimens with a 1.5% alkali content, the expansion of the specimens treated by autoclave curing are smaller than that of the control specimens at any treatment time. In particular, the longer the treatment time, the smaller the expansion. In addition, the residual expansion of the specimens with a 2.5% alkali content treated for a long time exhibited a greater expansion than that of the control specimens. This tendency was more pronounced when the alkali content was 3.0%. All specimens treated by autoclave curing exhibited higher expansion than the control specimens.

That is, when alkali content is low in comparison to reactive aggregate content, the extent of final expansion of the specimens treated by autoclave curing decreases, but when sufficient alkali content is provided, greater expansion occurs than for specimens not treated by autoclave curing.

It is considered that the gel in the specimens treated by autoclave curing has a greater calcium content than that of the specimen not by autoclave curing, that is, a lot of high-calcium-based gel is produced. [S. Nishibayashi 1987, 1988]

When the alkali content is low, the alkali silicate gel produced by alkali aggregate reactions obtains calcium ions from an early age, therefore, the proportion of high-calcium content gel (exhibiting low expansion) increases. We think this is the reason why the expansion decreases after autoclave treatment. On the other hand, when the alkali content is high, alkali aggregate reactions occur rapidly and the gel volume increases at an early age, so expansion after autoclave treatment increases. In addition, as the treatment time is lengthened, more alkali ions are consumed during autoclave treatment and more calcium ions elude, and the rate of increase in the expansion after autoclave treatment declines.

Influence of treatment pressure on residual expansion

Figure 7 indicates that maximum expansion appears at the treatment pressure of 0.2 MPa immediately after autoclave treatment. After that, the expansion of the specimens treated by autoclave curing is greater than that of the control specimens. In addition, the trend of the increase in expansion is roughly the same level, regardless of the treatment pressure.

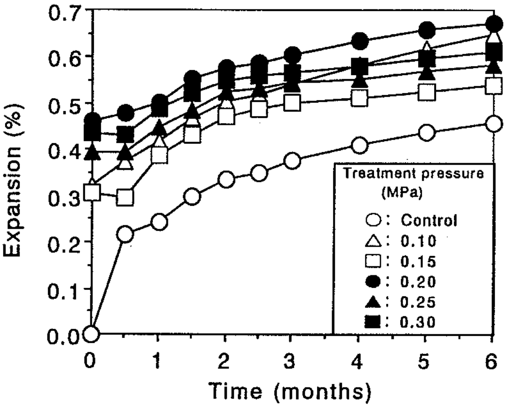

Fig-7 Expansion over time (Alkali content: 2.5%, treatment period: 4 hours)

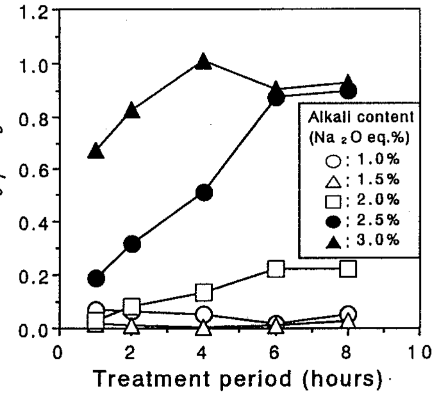

Fig-8 Relationship between $\varepsilon_0 / \varepsilon_6$ and treatment period (Treatment pressure: 0.2 MPa)

ε_0 : Expansion immediately after autoclave,

ε_6 : Expansion at 6 months

Expansion development immediately after autoclave treatment

Figures 8, 9, and 10 show the relationship between the ratio of $(\varepsilon_6/\varepsilon_0)$ between the expansion rate (ε_6) of the control specim at the age of 6 months (ε_6) and the expansion rate immediately after autoclave treatment (ε_0), and the treatment time, treatment pressure, and additional alkali content.

Based on Figure 8, showing the relationship between $(\varepsilon_6/\varepsilon_0)$ ratio and treatment time, it is determined that when the alkali content is 2.5%, 50% of the expansion of a 6-month control specimens developed at a treatment time of 4 hours, and 90%

? erreur ?

of the expansion of the same control specimens was developed at 6–8 hours of treatment. In addition, when the alkali content is 3.0%, 70% expansion was developed at a treatment time of 1 hour, and 90% expansion at 2 hours.

Based on the relationship between ($\varepsilon_6/\varepsilon_0$) in Figure 9, it is determined that when the alkali content is 3.0%,
the same expansion as that of the control specimens at the age of 6 months was developed at a treatment pressure of 0.2MPa, and at other treatment pressures, more than 50% of the expansion of the control specimens at the age of 6 months was obtained.

Fig. 10 shows the relationship between alkali content and ($\varepsilon_6/\varepsilon_0$). When the alkali content is 2.5%, the degree of expansion development varies with treatment time; the shorter the treatment time, the less the expansion development (0.5 at 4 hours, 0.9 at 6 and 8 hours). When the alkali content is 3.0%, more than 0.6 times the expansion development was determined at any treatment time. At the treatment time of 4 hours the same expansion development as that of the 6-month old control specimens can be expected.

Based on these results, it was determined that when alkali content is 3.0%, the same expansion as that of the specimens at the age of 6 months by ordinary acceleration testing at 40°C and R.H. of 100% was obtained by autoclave treatment at a pressure of 0.2 MPa and a treatment time of 4 hours.

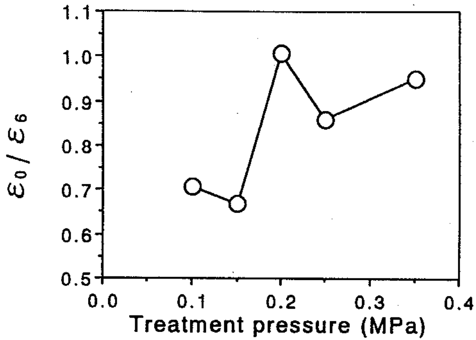

Fig-9 Relationship between ε_0 / ε_6 and treatment pressure (Treatment period: 4 hours, alkali content: 3.0%, time: 6 months)

ε_0 : Expansion immediately after autoclave,

ε_6 : Expansion at 6 months

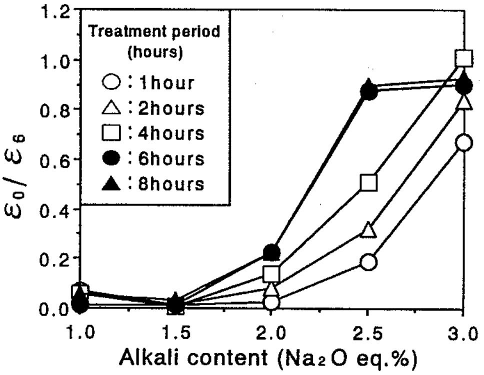

Fig-10 Relationship between ε_0/ ε_6 and alkali content (Treatment pressure : 0.2 MPa)

ε_0 : Expansion immediately after autoclave,

ε_6 : Expansion at 6 months

CONCLUSION

In this study we selected autoclave treatment to accelerate AAR in concrete and studied the characteristics of AAR expansion in concrete under high-temperature / high-pressure conditions and the residual expansion characteristics. The following results were obtained:

1) In order to accelerate AAR by autoclave treatment, we have to consider alkali content, treatment pressure, and treatment time as factors to maximize the expansion of concrete. We determined pessimum values for alkali content, treatment pressure,

and treatment time.

2) When concrete with a lower alkali content is treated by autoclave curing, the rate of expansion immediately after autoclave treatment decreases. The rate of residual expansion decreases compared to concrete not subjected to autoclave treatment. Therefore, when autoclave curing is adopted for acceleration of AAR, the alkali content should be higher than 2.5%.

3) In this experiment, the same expansion as found in specimens at the age of 6 months after accelerated testing at 40°C and a 100% R.H was obtained under an autoclave treatment pressure of 0.2 MPa, a 4-hour treatment time, and a 3.0% alkali content.

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PETROGRAPHY AND ALKALI-REACTIVITY OF SOME VOLCANIC AGGREGATES FROM ICELAND

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ABSTRACT

Petrographic examinations were made of volcanic aggregates from Iceland, based on microscopy, XRD and EPMA analysis of reactive minerals, to interpret early IBRI data of potential reactivity according to ASTM C227 mortar bar and C289 chemical tests. Basaltic rocks were found non-reactive when consisting of fresh basalt glass, but they can be reactive when they contain rhyolitic interstitial glass, and are even highly reactive when altered to contain secondary opal, besides cristobalite and rhyolitic glass. It was also revealed that volcanic rocks oversaturated with silica contain cristobalite and/or tridymite, and likely present pessimum phenomena. These relationships between the mineralogy and potential reactivity of Icelandic volcanic rocks are remarkably similar to those reported from volcanic aggregates in New Zealand and Japan.

Keywords: Alkali-reactivity, petrography, reactive minerals, volcanic rocks

INTRODUCTION

Concrete aggregates in Iceland are mostly basaltic rocks which contain both alkali-reactive and non-reactive types (Gudmundsson & Asgeirsson1975, Helgason 1981). However, petrographic methods to distinguish these aggregates with full mineralogical explanations, have not been well established (Thaulow & Olafsson 1983), unlike volcanic aggregates in the circum-Pacific region such as New Zealand and Japan (Katayama & Kaneshige 1986, Katayama et al. 1989).

In order to identify reactive minerals in these Icelandic volcanic rocks, detailed petrographic examinations were made by means of XRD and EPMA, and correlated with results of ASTM C227 & C289 tests obtained earlier at the Icelandic Building Research Institute (IBRI). This paper shows a possibility of mineralogically interpreting the reactivity of volcanic rocks from Iceland, like those done in the circum-Pacific region.

GEOLOGICAL SETTING

Iceland consists essentially of basaltic rocks belonging to younger geologic ages since the Miocene Tertiary time. Subglacial eruptions during Pleistocene Quaternary produced hyaloclastites, dominated by water-granulated basalt glass called sideromelane (Saemundsson 1979). Sand and gravel deposits used for concrete aggregates in Iceland are rich in these glassy basalts.

These rocks have undergone extensive burial diagenesis intermediate between weathering and regional metamorphism, which alters sideromelane through palagonite into an assemblage of secondary minerals, smectite-zeolite-opal (e.g. Jakobsson 1979). Since this process is common to Japanese volcanic rocks of the same geologic ages (Katayama & Kaneshige 1986), Icelandic volcanic rocks examined were classified here into three alteration zones: the unaltered, the slightly altered and the smectite zones, as defined in Japan (***Table 1***).

PETROGRAPHY OF VOLCANIC ROCKS

Several rock types representing western and southwestern Iceland, such as basalt, andesite, dacite, both deleterious and non-reactive basaltic sands, were examined petrographically to identify reactive minerals in these rocks. Summarized results are given below concerning the content of silica minerals, determined by microscopy (***Table 2***) and a phosphoric acid treatment coupled with XRD analysis (***Fig.1***), as well as chemical compositions of interstitial glasses and silica minerals analyzed by EPMA (***Tables 3, 4***).

Olivine tholeiite: This is almost a fresh basalt undersaturated with silica, and none of the silica minerals was detected. EPMA analysis revealed that the rock contains less than 5% of rhyolitic interstitial glass with SiO_2 >80% (***Fig.2***). Small amounts of iddingsite is found within olivine phenocrysts.

Tholeiite: This is a typical tholeiite oversaturated with silica, which is present as cristobalite (***Fig.3***) and quartz in the groundmass. It is a well-crystallized basalt containing a trace amount of rhyolitic interstitial glass, along with accompanying smectite of a nontronite variety that replaces the glass (***Table 3***). Secondary calcite occurs in veins.

Basaltic andesite: This rock contains nearly 10% of rhyolitic interstitial glass with SiO_2 of 77%, with a trace amount of cristobalite. The glass has been slightly altered to a disseminated smectite (nontronite).

Table 1 Geologic origin of Icelandic volcanic rocks

Geologic age	Rock type	Origin	Alteration zone	Locality
Pleistocene	Olivine tholeiite	Lava flow	Unaltered zone	Lake Leirvogsvatn
Pleistocene	Basaltic sand	Spit*	Unaltered zone /Slightly altered zone	Raudamelur
Pliocene	Tholeiite	Dike	Slightly altered zone	Mt.Brekkufjall
Pliocene	Basaltic andesite	Lava flow	Slightly altered zone	Mt.Raudahnukur
Pliocene /Pleistocene	Basaltic sand	Glacio-marine*	Smectite zone /Unaltered zone	Hvalfjordur
Tertiary	Dacite	Plug	Smectite zone	Stora Borg

* Hyaloclastite dominates in the Pleistocene parent rock

Table 2 Petrography of some Icelandic volcanic rocks

	Chemical			XRD of residue[3]			Microscopy[4]								
	SiO_2[1]	Q[2]	Res[3]	Silica minerals			Silica minerals					Glass			
				cr	tr	qz	cr	tr	op	cd	qz	sd	pg	rh	sm
Olivine tholeiite	48.0	0	0.0	0.0	0.0	0.0	0	0	0	0	0	0	0	3	0
Tholeiite	50.5	6	4.0	2.1	0.0	1.9	1	0	0	0	<1	0	0	2	1
Basaltic andesite	52.6	6	0.1	0.1	0.0	0.0	<1	0	0	0	0	0	0	9	2
Dacite	63.5	17	8.1	0.8	3.5	3.8	1	3	0	0	5	0	0	20*	5
Raudamelur sand			0.0	0.0	0.0	0.0	0	0	0	0	0	15	3	1	0
Hvalfjordur sand			1.4	0.1	0.0	1.3	<1	0	<1	<1	<1	3	1	1	6

[1] After Fridleifsson(1973) and Franzson(1978)
[2] Normative quartz as oversaturated silica
[3] Residual silica, extracted by phosphoric acid treatment
[4] cr: cristobalite, tr: tridymite, op: opal, cd: chalcedony, qz: quartz
sd: sideromelane, pg: palagonite, rh: rhyolitic glass, sm: smectite
* : devitrified to a cryptocrystalline groundmass

Dacite: This is an altered rock containing smectite. About a half of the oversaturated silica is present as silica minerals tridymite, cristobalite and quartz (*Table 2*), but the other half is hidden in a devitrified glass with a crypto- to microcrystalline texture. Tridymite occurs as wedge-shaped crystals, while cristobalite forms imbricated round patches in the groundmass. Some of these primary silica minerals have been inverted to secondary quartz leaving pseudomorphs of their original outlines (*Fig.4*). Secondary quartz also occurs as irregular-shaped, sponge-like patches in the devitrified groundmass, formed through recrystallization of interstitial glass. The mineral assemblage of smectite-tridymite-cristobalite suggests that the rock has undergone a moderate degree of alteration, comparable to the smectite zone defined in Japan.

Table 3 Volcanic glasses in Icelandic volcanic rocks

	Olivine tholeiite	Tholeiite		Basaltic andesite
	Rhy glass	Rhy glass	Nontronite*	Rhy glass
An. no:	98	207	209	116
SiO_2	80.75	79.75	43.54	77.19
TiO_2	0.53	0.40	0.19	0.84
Al_2O_3	10.38	9.68	4.32	11.55
FeO	1.57	1.79	30.81+	2.35
MnO	0.00	0.03	0.21	0.04
MgO	0.02	0.17	6.66	0.02
CaO	1.47	0.27	1.34	0.68
Na_2O	3.55	2.93	0.14	1.11
K_2O	0.68	4.13	0.35	3.65
Total	98.95	99.15	87.56	97.43

* Altered from glass, + as Fe_2O_3

Fig. 1 XRD patterns of silica minerals extracted

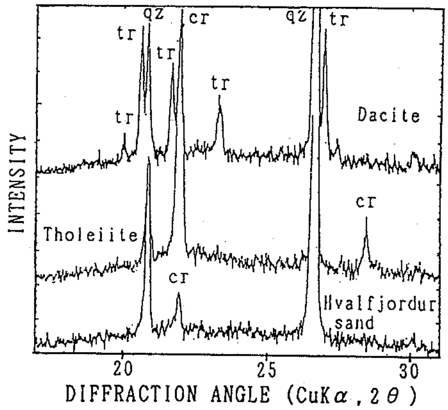


Fig.2 Rhyolitic interstitial glass in olivine tholeiite

Fig.3 Interstitial cristobalite in tholeiite

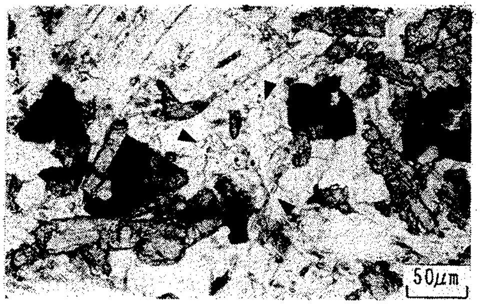


Fig.4 Pseudomorphic quartz after tridymite in dacite

Fig.5 Palagonite rim to sideromelane in Raudamelur sand

Raudamelur sand: This sand, roughly a fresh basaltic hyaloclastite, has been used as a concrete aggregate for over two decades with good results. EPMA analysis revealed that a full range of interstitial glasses from basaltic to rhyolitic compositions are present in the basalt grains, depending on their crystallinity (***Table 4***). The sand with a grain size 0.6-1.2mm, consists of fresh basalt (55%), basalt glass (sideromelane, 30%), altered basalt glass (palagonite, 10%), plagioclase, olivine and other rock fragments. Sideromelane represents average bulk compositions of fresh basalt, while a palagonite rim to sideromelane indicates an initial alteration of this glass (***Fig.5***).

Hvalfjordur sand: This was a sea-dredged aggregate used as a sole sand source for concrete in Reykjavik from the 1960's to 1970's, usually unwashed. Extensive damage to concrete houses was caused by this aggregate (Olafsson 1989). It is an altered basalt containing smectite, and the fraction of 0.6-1.2mm consists of altered basalt (80%), fresh basalt (10%), rhyolite, andesite, basalt glass (sideromelane), secondary minerals and shell fragments.

Vesicles of altered basalt are often filled with smectite (nontronite), forming amygdale textures, or occasionally lined with secondary opal (***Fig.6***) or chalcedony. In the altered basalt, cristobalite occurs as rounded patches, surrounded by smectite that replaces interstitial glass (***Fig.7***), and zeolite (alkali-heulandite, ***Table 4***) may also be present. The mineral assemblage of opal-cristobalite-smectite-zeolite is common to moderately altered volcanic rocks from Miocene to Pliocene Tertiary in Japan, defined as the smectite zone (Katayama & Kaneshige 1986). Secondary opal and smectite are also common to a deleterious Tertiary basalt from Queensland, Australia (Shayan & Quick 1988).

Table 4 EPMA compositions of mineral phases in the basaltic sands

	Raudamelur sand					Hvalfjordur sand						
	Fresh basalts				Alt.bas	Fresh basalts			Altered basalts			
	Unaltered zone				Slt.alt.z.	Unaltered zone			Smectite zone			
	Bas glass	And glass	Dac glass	Rhy glass	Pala-gonite	Bas glass	And glass	Rhy glass	Cristo-balite	Opal	Non-tron.	Heu-land.
An.no:	140*	133	138	128	202	65*	61	96	78	62**	81***	72
SiO_2	50.09	54.42	67.35	70.53	33.09	49.82	57.11	80.06	97.09	95.7	50.44	63.96
TiO_2	1.54	2.67	0.42	2.12	2.81	1.08	2.49	0.74	0.09	1.3	0.23	0.03
Al_2O_3	13.15	10.53	10.24	11.71	26.94	13.88	9.30	9.98	2.00	0.3	8.65	18.07
FeO	11.20	11.59	12.36	6.93	19.28	11.13	16.19	1.51	0.07	2.2	21.67+	0.24
MnO	0.19	0.18	0.00	0.17	0.20	0.23	0.43	0.02	0.00	0.0	0.02	0.01
MgO	8.08	3.05	0.33	0.03	0.45	8.58	3.26	0.00	0.00	0.3	3.49	0.09
CaO	12.17	13.69	3.36	4.83	0.94	13.11	7.96	0.25	0.07	0.1	0.63	0.96
Na_2O	1.99	0.79	1.94	2.25	0.10	1.89	1.28	1.28	0.36	0.0	0.38	2.38
K_2O	0.37	0.53	1.84	0.72	0.16	0.21	1.41	4.46	0.02	0.2	2.82	4.39
Total	98.78	97.44	97.83	99.30	83.97	99.92	99.41	98.30	99.70	100.0	88.32	90.11

* Sideromelane, ** total normalized to 100%, *** altered from glass, + as Fe_2O_3

Fig.6 Opal in altered basalt in Hvalfjordur sand

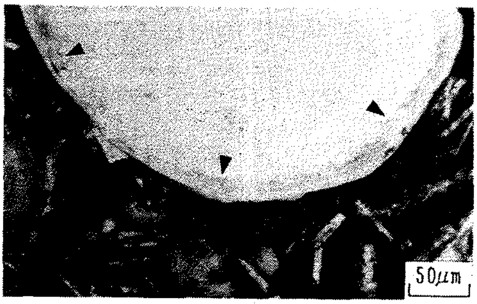
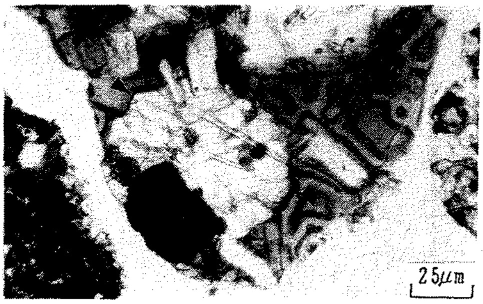


Fig.7 Cristobalite surrounded by smectite in Hvalfjordur sand



POTENTIAL REACTIVITY OF ICELANDIC VOLCANIC ROCKS

The results of the ASTM C289 and ASTM C227 tests of Icelandic volcanic rocks performed at IBRI (e.g. Helgason 1981), are shown in ***Table 5*** and ***Figs.8 & 9***.

Chemical test: Rocks containing rhyolitic glass (olivine tholeiite & basaltic andesite) with low dissolved silica (Sc) & low reduction in alkalinity (Rc), classify as deleterious, whereas rocks with silica minerals cristobalite and tridymite (dacite, tholeiite & Hvalfjordur sand) gave higher Sc & Rc in the potentially deleterious field, or near the boundary. These tendencies are similar to volcanic rocks of the same geologic ages in Japan & New Zealand (Katayama et al. 1989). Experiences show that Icelandic deleterious volcanic rocks are often misjudged as innocuous in this test, due to unreliably high values of Rc, so that only values of Sc > 100 mmol/l have been used as a deleterious indication (Thaulow & Olafsson1983). Smectite in altered rocks is probably the cause of misjudgement of marginally reactive rocks as innocuous, because this mineral raises Rc values by cation exchange (Morino et al. 1987).

Mortar bar test: Olivine tholeiite tested as innocuous in the ASTM C227 at 26 weeks, and was mixed as 'inert' aggregate to examine pessimum phenomena of other rock types (***Table 5***). Rocks containing cristobalite and/or tridymite (dacite & tholeiite) exhibited a marked pessimum proportion at 20% mixing, whereas basaltic andesite containing mainly rhyolitic interstitial glass, did not show such a marked pessimum (***Figs.8,9***). Tridymite and cristobalite are considered responsible for the pessimum phenomena of some Icelandic volcanic rocks, which is similar to the experiences in Japan and New Zealand (Katayama et al. 1989). The olivine tholeiite is marginally reactive at one year (0.095% /52 weeks), if judged by a criterion (0.1%/1 year) by Mielenz et al (1947) that divides the innocuous & deleterious fields in the ASTM C289. As discussed later, the reactivity of this rock is attributable to the rhyolitic glass.

Table 5 Potential reactivity of Icelandic volcanic rocks

Rock	Reactive	ASTM C289 1)			ASTM C227 (26w) 1)				ASTM C227 (52w) 1)			
	phase	Sc	Rc(mmol/l)		10%	20%	30%	100%	10%	20%	30%	100%
Olivine tholeiite	Rhy.gl	86	18	D	-	-	-	0.08 I	-	-	-	0.10(D)
Raudamelur sand	(Bas.gl)	96	91	PD	-	-	-	-	-	-	-	0.04 I
Basaltic andesite	Rhy.gl, cr	167	151	D	-	0.14	-	0.07 D	-	0.20	-	0.12 D
Hvalfjordur sand	Cr, opal	204	179	PD	-	-	-	-	-	-	-	0.3-0.5D
Tholeiite	Cr	268	272	(I)	0.17	0.20	0.19	0.02 D	0.21	0.27	0.26	0.04 D
Dacite	Tr, cr	340	320	PD	0.33	0.43	0.43	0.01 D	0.38	0.56	0.54	0.03 D

1) IBRI data, using high-alkali cement; gl:glass, cr:cristobalite, tr:tridymite D:deleterious, PD:potentially deleterious, I:innocuous

Fig.8 Pessimum phenomena of volcanic rocks (26 weeks)

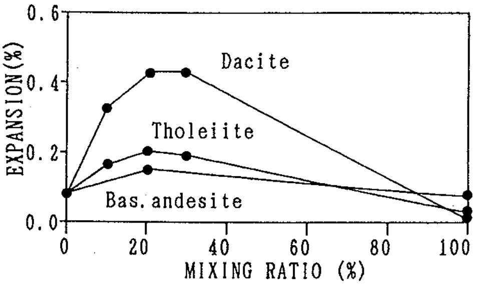
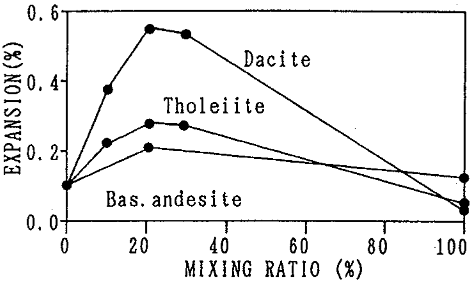


Fig.9 Pessimum phenomena of volcanic rocks (52 weeks)



REVIEW OF POTENTIAL REACTIVITY OF REACTIVE MINERALS

A literature review was made on the potential reactivity of silica minerals and volcanic glasses (***Table 6***), to support interpretations of the results of the ASTM chemical test and ASTM mortar bar test of Icelandic rocks.

Silica minerals: References show that silica minerals have marked pessimum proportions, which tend to decrease with increasing the potential reactivity, i.e. opal, cristobalite, tridymite and chalcedony present pessimum proportions from less than 5% to 20% in the mortar bar test, and occupy the potentially deleterious field in the chemical test. Cryptocrystalline quartz and microcrystalline quartz are less reactive and their pessimum ranges from about 50% to 80%, or possibly 100%. The boundary line between the potentially deleterious field and the deleterious field in the ASTM C289 was presumably drawn at a pessimum proportion of 50%, based on the data by Mielenz et al. (1947) and Chaiken and Halstead (1959).

Volcanic glasses: Early American data suggest that obsidians, a natural rhyolitic glass, have pessimum proportions (Mielenz et.al.1947). However, it is necessary to check the presence of cristobalite and tridymite in these glassy rocks by a XRD analysis, because these minerals are often hidden in the obsidians and likely missed under the microscope. Nowadays, volcanic glasses are not considered to exhibit pessimum phenomena unless silica minerals such as cristobalite and tridymite are present (e.g.CCANZ 1991). Fresh volcanic glasses become particularly reactive when their SiO_2 content exceeds 65%, i.e. dacitic composition (Katayama et al.1989, Katayama 1992).

For example, deleterious Egmont andesite in New Zealand containing dacitic glass (SiO_2 65.4%, Katayama et al.1989) and a glassy bronzite andesite in Japan containing rhyolitic glass (SiO_2 78.0%), did not show a pessimum (StJohn 1988, Yoshioka et.al.1985). By contrast, less glassy bronzite andesite from Japan, with glass plus cristobalite, showed a pessimum proportion between 20 and 40% (Yoshioka et al.1985). Likewise, an altered basalt from Queensland, Australia, containing dacitic glass (SiO_2 67.3%) plus secondary opal and smectite (nontronite), showed a pessimum (Shayan & Quick1988). Thus, rhyolitic glass in the Icelandic rocks is not believed to show a pessimum phenomenon.

Table6 Alkali reactivity of silica minerals and volcanic glasses

Reactive mineral	ASTM C289 chemical test			ASTM C227 mortar bar test		
	Sc(mmol/l)	Result	Material	Pessimum	Material	Result
Silica minerals						
Opal	>1000	PD	Opal(1, 2)	< 5%	Opal(1), opaline chert(3)	D
Cristobalite	500-1000	PD	Synthetic(4)	<10%	Syn.(5), andesite(6)***	D
Tridymite	500-1000	PD	Andesite(4)*	<10%	Syn.(5), andesite(6)***	D
Chalcedony	500-1000	PD	(1, 2, 7)	20%	Chalcedony(1, 7)	D
Cryptocrystal.qz	150-300	PD/D	Chert(2, 8)**	50%	Chert(8, 9)**	D
Microcrystal. qz	70-150	D	Chert(2)	>80%	Miscellaneous(10)****	D
Volcanic glasses						
Rhyolitic(SiO_2>70%)	70-90	D	Synthetic(4)	100%	Andesite(gl. $SiO_2$78.0%)(11)	D
Dacitic (62-70%)	70-90	D	Synthetic(4)	100%	Andesite(gl. $SiO_2$65.4%)(12)	D
Andesitic (52-62%)	30-50	I/D	Synthetic(4)		Syn. gl($SiO_2$61.8%)(13)	(D)
					Basalt(gl. $SiO_2$52.1%)(4)	I
Basaltic (<52%)	15-30	I	Synthetic(4)		Glassy basalt(1)	I

* Mixed with cristobalite, ** mixed with chalcedony
*** JIS A 5308 mortar bar test, ****CSA A 23.2-25A accelerated mortar bar test
(1)Mielenz et al.(1947), (2)Katayama & Futagawa(1989), (3)Stanton et al.(1942)
(4)Katayama et al.(1989), (5)Gaskin et al.(1955), (6)Ishida et al. (1988)
(7)Andriolo & Sgaraboza (1986), (8)Morino (1989), (9)Goldman & Klein (1959)
(10)Grattan-Bellew (1992), (11)Yoshioka et al.(1985), (12)StJohn (1988)
(13)Stanton (1942), D:deleterious, PD:potentially deleterious, I:innocuous

CONCLUDING REMARKS

From the foregoing considerations on the geology, petrology and potential reactivity of Icelandic volcanic rocks, along with the information from reviewed references, the following points can be summarized.

- Icelandic volcanic rocks have undergone stratigraphic alterations due to burial diagenesis, in a similar mode to that of Japanese volcanic rocks of the same geologic ages (Neogene Tertiary & Quaternary). The rocks examined can be classified into the unaltered, the slightly altered, and the smectite zones from upper to lower horizons.
- Deleterious basaltic sand came from altered strata of the Neogene Tertiary, belonging to the smectite zone. The high reactivity of this sand is due to the presence of secondary opal and chalcedony, combined with primary cristobalite and rhyolitic glass. Smectite in the altered rocks may give an unreliable result in the chemical test, causing high Rc and a misjudgement of potentially deleterious rocks as innocuous.
- Basaltic rock is usually non-reactive when it is glassy and contains fresh basaltic glass, but may show a potential reactivity when it is highly crystalline and contains rhyolitic interstitial glass as a residual melt. Rocks containing rhyolitic glass tend to test as deleterious in the chemical test, which suggests little possibility for these rocks to show a pessimum phenomenon in the mortar bar test.
- Volcanic rocks oversaturated with silica often contain the reactive silica minerals cristobalite and tridymite. They show high values of Sc & Rc in the chemical test, and exhibit a marked pessimum phenomenon. All these tendencies are common to volcanic aggregates of the same geologic ages in the circum-Pacific region, such as New Zealand and Japan.

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BLAST FURNACE SLAG'S POTENTIAL ALKALI REACTIVITY AND LONG TERM SERVICE

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ABSTRACT

Air-cooled iron Blast Furnace Slag Aggregate from the works of Broken Hill Proprietary (BHP) Port Kembla has been used in concrete since the early 1950's.

The suitability of this aggregate for use in concrete has been assessed in accordance with the appropriate Australian Standard AS 2758.1 - 1985 Part 1. This assessment included consideration of the potential reactivity of the aggregate to alkalis.

Of recent interest is the accelerated test procedure developed by the CSIRO (Commonwealth Scientific and Industrial Research Organisation of Australia) for the assessment of potential alkali aggregate reaction.

This paper reports the results of testing for potential alkali reactivity using the Australian Standard tests and the CSIRO test of Port Kembla Blast Furnace slag aggregate and the inspection of the performance of concrete made using this aggregate over a period of nearly 40 years.

Keywords: Alkali Reaction, Blast Furnace Slag.

INTRODUCTION

"Satisfactory" concrete can be made from a wide range of aggregates obtained from either natural resources or as by-products of manufacturing processes. The measure of being "satisfactory", until recent years, has been to comply with a specified compressive strength at 28 days.

More recently the focus has been on how the concrete performs in service, that is, its serviceability or durability.

These durability considerations have included such aspects as shrinkage and susceptibility to cracking, ability to prevent corrosion of the reinforcement and possible spalling of the concrete, and the likelihood of alkali aggregate reaction with the subsequent deterioration of the concrete from this action.

This last mentioned phenomena has assumed increasing prominence in very recent times as its occurrence seemingly has become more widespread.

The concern for the likelihood of the occurrence of alkali aggregate reaction understandably has concentrated on concrete made using natural aggregates rather than artificial aggregates because of the greater use of the former materials. However with the move to conserve natural resources and hence to increase the use of aggregates obtained as by-products of manufacturing processes, the possibility of alkali aggregate reaction from the use of artificial aggregates should be considered.

A significant potential source of aggregate for the major construction area of Sydney (Australia), population of approximately five million is blast furnace slag from the Port Kembla Steelworks of BHP, some 80 kms south of Sydney. Approximately 1.2 million tonnes of blast furnace slag is produced at Port Kembla each year. Of this, a certain amount is granulated for use in cement manufacture leaving an appreciable amount of material that is air-cooled and available for use as a concrete aggregate. In keeping with the concern of possible alkali aggregate reaction, it is appropriate to examine air cooled blast furnace slag aggregate as a potential contributor to this problem.

THE FORMATION OF AIR COOLED BLAST FURNACE SLAG

Blast furnace slag is made in the production of iron in a blast furnace. The molten slag being the lighter of the two materials floats on the top of the molten iron. The iron is tapped off to be used in the production of steel while the molten slag is run off and cooled.

In the case of Port Kembla blast furnace slag, the molten slag is either run off into pits and allowed to air cool or cooled quickly with a high volume of water. This latter process causes the slag to form granules that are suitable for the manufacture of slag cement.

The air cooled slag is dug from the cooling pits and transferred to a storage area.

The air cooled slag is then made into concrete aggregate by processing through a crushing and screening plant.

COMPOSITION AND MINERALOGY OF PORT KEMBLA BLAST FURNACE SLAG AGGREGATE

Slag is composed essentially of lime, alumina and silica with variable amounts of magnesia and small amounts of sulphur, alkalis, titania etc.

As will be seen from the mineralogy these compounds do not occur in the slag as single entities but, for the Port Kembla slag, combined mainly as calcium alumino silicate with also other calcium silicates and calcium sulphide.

The chemical composition of the currently produced Port Kembla blast furnace slag aggregate is shown in Table 1 (Australian Steel Mill Services, 1994)

Table 1 Chemical composition of Port Kembla blast furnace slag aggregate

Constituent	Percentage
CaO	41.1 - 41.6
SiO_2	33.7 - 34.3
Al_2O_3	14.1 - 14.7
MgO	6.5 - 7.1
MnO	0.39 - 0.48
S	0.7 - 0.8

It is emphasised, with particular reference to the silica content, that these compounds do not exist in blast furnace slag as "free" or single compounds.
It has been established that a petrological examination is necessary if a considered judgement on the presence of reactive silica is to be made (Nurse *et al.* 1951).

Bauman (1959), speaking of blast furnace slags in general, stated that very little, if any, free silica existed in slag. Concerning Port Kembla blast furnace slag, Hensel (1995) reported no free silica in slag from this source.

TESTING FOR POTENTIAL ALKALI REACTIVITY

Australian Standard 2758.1 - 1985 (Standards Australia 1985) refers to three test procedures that could be used to assess the potential alkali reactivity of an aggregate:

- ASTM 295 (American Society for Testing and Materials 1994) a method of petrographic examination for analysis of types of minerals present.
- Australian Standard 1141 Section 39 (Standards Australia 1974a), a chemical test relating the reaction of the aggregate with sodium hydroxide in terms of amount of silica dissolved and the reduction in alkalinity of the solution - expressed in terms of the potential of the aggregate to react with alkali.
- Australian Standard 1141 Section 38 (Standards Australia 1974b) where reactivity of the aggregate is assessed from the expansion of mortar bars made using the aggregate, the bars being cured for up to six months.

Dissatisfaction with these methods has led to the development of other tests including two Australian test methods -

- the CSIRO accelerated mortar bar method (Shayan *et al.* 1988)
- the Queensland Department of Transport accelerated cured concrete prisms (Carse *et al.* 1990).

The Port Kembla air cooled blast furnace slag aggregate is regularly tested for potential alkali reactivity using the test method of AS 1141 Section 39 and by petrographic examination.

Testing of the Port Kembla slag aggregate to Section 39 gives typical results for reduction in alkalinity and silica concentration of 65.0 and 3.0 millimoles/litre respectively. These results classify the Port Kembla slag aggregate as "innocuous" as far as potential alkali reactivity is concerned.

The petrographic examination typically shows the Port Kembla blast furnace slag aggregate to be predominantly melilite with no free silica. (Table 2).

Table 2 Typical petrographic description of Port Kembla blast furnace slag aggregate

Constituent	Percentage
Melilite	55
Larmite	4
Opaque oxides	Less than 1
Unidentified Ca Silicates	1
Glass	1
Interstitial Material	13
Voids	25
Carbonate	Less than 1
Others	-

The Port Kembla blast furnace slag aggregate has also been tested for potential alkali reactivity using the CSIRO accelerated mortar bar method. Testing to this method has shown the Port Kembla material to be non-reactive.

EXAMINATION OF OLD CONCRETE CONTAINING PORT KEMBLA BLAST FURNACE SLAG AGGREGATE

In April 1995 a survey was made of a number of concrete structures within the steelworks of Broken Hill Proprietary (BHP) at Port Kembla to detect the occurrence of alkali aggregate reaction in the concrete.

The structures inspected were specially selected for the following considerations:

- Concrete age in excess of 30 years.
- Exposure conditions conducive to alkali aggregate reaction.
- No supplementary cementitious materials used in the concrete.

At the time of selection of the structures it was appreciated that other factors such as the cement content of the concrete and the alkali content of the cement used can have an effect on the occurrence of alkali aggregate reaction. However it was decided to defer consideration of the possible influence of these factors until the final analysis.

Supplementary cementitious material was first used in concrete for the BHP Port Kembla works, then operating as Australian Iron and Steel Pty Ltd, in 1966. The material used was ground granulated blast furnace slag (Visek. 1969). Although flyash was available in the area in 1965 it was not used in concrete supplied to Australian Iron and Steel Pty Ltd at Port Kembla until 1967 some time after the introduction of the use of ground granulated blase furnace slag in concrete supplied to that company. (Visek .1969)

The survey was restricted to concrete placed before 1966 to ensure that no supplementary cementitious materials had been used in the concrete.

DETAILS OF CONCRETE MIX USED

The typical mix design for the concrete supplied to all of the structures inspected was as follows:

Type A Ordinary Portland Cement	290 kg/cu.m
20mm air cooled blast furnace slag aggregate	1185 kg/cu.m
6mm air cooled blast furnace slag aggregate	145 kg/cu.m
Dune sand	565 kg/cu.m
Water (approximate)	170 litres/cu.m

RESULTS OF THE INSPECTIONS OF OLD CONCRETE STRUCTURES AT BHP STEELWORKS, PORT KEMBLA

Number 1 salt water lift pump station

This structure was built in 1958. It is in tidal seawater with parts of the structure in the splash zone. Seawater temperatures range from 13 to 22°C throughout the year.

No supplementary cementitious materials were used in the concrete.

From the records that are available it would seem that cement contents used in these times were not high. (Murrie *et al.* 1968)

A thorough visual examination of this structure showed no cracking or distress that could be attributed to the occurrence of alkali-aggregate reaction. In fact, this structure is notable for the lack of cracking of the concrete.

Number 2 - 3500mm plate mill scale pits

The concrete in this structure at the time of inspection was 32 years old. The concrete is exposed to wetting and drying in a water environment. In addition the operation of a clam shell bucket removing mill scale from the bottom of the pit would provide mechanical impact as well as abrasion and attrition from the mill scale particles in the water.

Steel sections have been set into the surface of the concrete as a means of protection from the impact of the clam shell. However it is believed that such inclusions of steel could aggravate deterioration of the concrete by differential expansion and contraction due to thermal effects. The concrete in this situation would be subjected to fairly high stresses from a number of different influences.

These stresses, it is suggested, would aggravate any potential for alkali aggregate reaction.

When this concrete was examined, some 32 years after placement it showed no sign of cracking or other distress.

Number 3 - Coolant basement of cold rolled steel mill

The concrete for this structure was placed in 1961. The conditions in the basement are humid, warm to hot still air with the presence of high electrical activity due to the presence of continuously operating electrical motors.

The concrete was closely examined for signs of alkali aggregate deterioration. No deterioration of the concrete that could be attributed to alkali aggregate reaction was detected.

Number 4 - Walls, and beams of the Oil Cellars and De-scaling Pump-house

This structure was built in 1963. The adverse conditions are heat and humidity. No deterioration of the concrete was observed when inspected on 6 April 1995.

Discussion

The conditions to which the old concrete has been subjected and the length of time the concrete has experienced these conditions are considered to be conducive for the occurrence of alkali aggregate reactivity.

The examination of the various old concrete structures however has shown no signs that this reaction has occurred.

Other factors influencing the occurrence of AAR have been considered. It has been reported that no supplementary cementitious materials were available for use in the concrete for these structures (Visek 1969). Also the cement content of the concrete would not have been particularly high (Murrie *et al.* 1968). The typical mix design of the concrete confirms this. Concerning the available alkalis from the cement used, it would seem that the alkali content of the cement supplied by Southern Portland Cement during the time these structures were built was fairly low (Na_2O 0.04%, K_2O 0.20%) (Schott 1995).

The most definite and influential factor that has been established in examining the lack of evidence of alkali aggregate reaction is the innocuous nature of the blast furnace slag aggregate. It is maintained that this is the single most important factor in the non-occurence of alkali aggregate reaction in the concrete structures examined.

Conclusion

Concrete structures ranging in age from 32 to 37 years subjected to conditions considered conducive to alkali aggregate reactivity were examined and found to show no signs of this phenomenon.

From reports of the various materials used in the concretes, particularly mineralogical examinations and potential alkali reactivity testing of the blast furnace slag aggregate, the most important factor in the non-occurrence of alkali aggregate reaction seems to be that the aggregate is not reactive with alkalis.

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Structural Effects

of

Alkali-Aggregate Reaction

A REVIEW OF THE INSTITUTION OF STRUCTURAL ENGINEERS REPORT "STRUCTURAL EFFECTS OF ALKALI-SILICA REACTION (1992)"

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ABSTRACT

The publication of the Institution of Structural Engineers' Report "Structural Effects of Alkali-Silica Reaction" (Doran 1992) coincided with the 9th International Conference on Alkali Aggregate Reaction. Since the publication a three year research programme, undertaken at The University of Birmingham, has been completed. Much of the research carried out related to the "Research Needs" described in the Institution of Structural Engineers' Report. During the research programme over 200 concrete specimens affected by Alkali Silica Reaction (ASR) were tested. The results of these tests are discussed in relation to the Institution of Structural Engineers Report. It is shown that the report overestimates deterioration in relationship to expansion and reasons are given for this. The paper also discusses the relationship between expansion and restraint and methods of predicting expansion are tested. It is shown that a better estimation of expansion can be obtained from crack widths when the crack angle is also considered.
Keywords: Alkali-silica, Concrete properties, Crack widths, Expansion, Restraint.

INTRODUCTION

The Institution of Structural Engineers' (IStructE) Report "Structural Effects of Alkali-Silica Reaction" (Doran 1992) listed areas where it believed additional information was required to aid in the assessment of structures affected by ASR. Experimental work was carried out at The University of Birmingham to investigate: the changes in the characteristics of the concrete as ASR develops and how these changes should be considered in the appraisal of a structure, the phenomena of laboratory scaling and acceleration and their effects on the reaction, and the effects of restraint on expansion. The results of this experimental work are discussed in relation to the relevant chapters of the IStructE Report.

EXPERIMENTAL PROCEDURE

The experimental work involved the testing of Alkali Silica reactive specimens with various sizes, expanded at various rates. The rate of expansion was varied by modification of the conditioning temperature of the water the specimens were stored in. Temperatures of 38, 30, and 20°C, giving approximate expansion rates of 0.1 0.06 and 0.01 mm/m/day. The specimens were reinforced with steel ratios ranging from zero to 2% and expanded under applied stresses ranging from 4 N/mm^2 tension to 7 N/mm^2 compression. The specimens were monitored until their expansion was complete and then tested to failure (Jones 1994). These failure tests were carried out to ascertain the compressive strength, elastic modulus and tensile strength of both specimens and cores removed from the specimens. All the specimens tested were manufactured from a concrete mix with a Na_2O equivalent of 7 kg/m^3, obtained by the addition of sodium and potassium hydroxides, and a Thames Valley reactive aggregate.

RELATIONSHIP BETWEEN EXPANSION AND CONCRETE PROPERTIES

Chapter 4 of the IStructE report discusses the "Physical effects on concrete of ASR expansion". A table is given which relates the properties of Cube compressive strength, Uniaxial compressive strength, Elastic modulus and Tensile strength, as a ratio of their 28 day value, to expansion. This table represented the lower bound to the then available data. The properties of uniaxial compressive strength and elastic modulus are likely to be the most significant for assessment purposes. The values given are discussed in relation to the research described here.

Uniaxial Compressive Strength

Figure 1 presents the crushing strengths of 100 mm x 200 mm long cylinders expanded under different applied stresses to induce different final expansions. On this figure the relationship between uniaxial compressive strength and expansion given by the IStructE report is also plotted along with a new relationship proposed by the Authors later in this paper.

The relationship between uniaxial compressive strength and expansion given by the IStructE report underestimates the compressive strength. This is not surprising given that the relationship was based on a lower bound to a large number of data sources, however, the research described here has highlighted reasons for this underestimation:

- The crushing strength of ASR affected control specimens was found to be more sensitive to length to width ratio than normal concrete. This was found to be due to the restraining effect of the non-reacted surface layer of the ASR control specimens in the direction perpendicular to loading. It was found that a length to width ratio of at least 3 was required to give the true uniaxial strength (Jones 1994) and this agrees with the value stated by Clayton et al (1990).
- The data plotted in the IStructE report included those from cores. These cores did not have the non-reacted skin of the control specimens and would not have experienced the benefit of restraint which it applied. The concrete in a reacted structure will generally be under some degree of restraint perpendicular to the direction of loading in the form of the surrounding concrete, reinforcement or applied loads. It has also been shown (Jones *et al* 1994) that cores deteriorate during the cutting process and continue to deteriorate during the time between cutting and testing. For these reasons the strength found from cores removed from an affected area of a structure will underestimate the true compressive strength of the in situ concrete.

It is suggested that the proposed relationship shown on Figure 1 is adopted when comparing compressive strength to expansion. This relationship uses the same factors as those proposed by the IStructE report; however, these factors are applied to an estimated undeteriorated strength instead of the 28 day strength. In Figure 1 the estimated undeteriorated strength was obtained by extrapolating the data back to the zero expansion line and taking a reasonable lower bound. In reality it could be found by testing unreacted areas of the structure. It should be noted that in all cases the 28 day strength quoted is the actual 28 day strength which in itself may be significantly greater than the value used for design purposes.

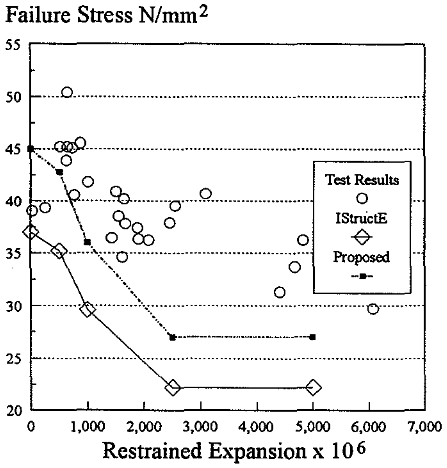


Figure 1 Compressive strength of concrete compared to expansion.

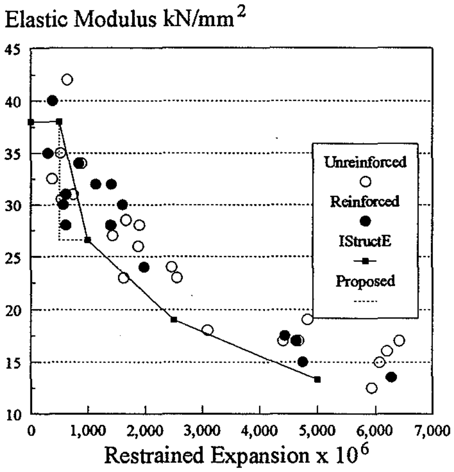


Figure 2 Elastic modulus of concrete compared to expansion.

Elastic Modulus

Figure 2 compares the elastic modulus of the cylinders discussed above, to their restrained expansion. Again the relationship given by the IStructE report is plotted. The correlation between the data and the relationship given is good except at low expansions where the relationship overestimates the elastic modulus of some specimens. It should be noted that there is much scatter in the results at these low expansions. It is unlikely that the expansion estimation of real structures will be accurate to 500 microstrain. Hence, it is proposed that estimated expansions in the range 500 to 1000 microstrain should be rounded up to 1000 microstrain to give the relationship shown in Figure 2. Thus, a member would either be classed as having negligible expansion and zero deterioration, or an expansion of 1000 microstrain or above with an appropriate loss of stiffness.

EFFECTS OF ASR ON CONCRETE STRUCTURES

Chapter 5 of the IStructE report discusses "The effects of ASR on Concrete Structures". Whilst much of the research carried out confirmed the information given in the report this section highlights areas where the research has added to the understanding.

Influence of Restraints

Reinforcement

When ASR affected reinforced concrete expands the reinforcement is placed in tension. This places the concrete in compression which restrains further expansion. The IStructE report calculated the stress developed in the concrete for various specimens with various reinforcement ratios. It was concluded that the stress generated increased with reinforcement ratio and this was confirmed by the testing carried out. The IStructE report also proposed that the stress generated was related to the expansion rate. In the tests described here the stress generated was found to be unrelated to expansion rate (Jones 1994).

In the experimental work reported here the concrete stress generated by the tensioning of reinforcement was found to be highly dependent on the orientation of the reinforcement in relation to the casting direction of the member. Figure 3 presents the upper and lower bounds of the stresses found by other researchers (Hobbs 1988), (Cope *et al* 1993) and (Kobayashi 1986) along with the results found by the Authors for 100 mm x 200 mm long cylinders and 100 mm x 200 mm x 2000 mm beams. It can be seen that the cylinders generally developed stresses close to the upperbound of those found by others, whilst the beams developed stresses towards the lower bound. The cylinders were cast (and reinforced) in the vertical position, whilst the beams were cast in the horizontal position. From this it can be concluded that the stresses developed, by the stressing of reinforcement, in the direction of casting are greater than those generated in the direction perpendicular this.

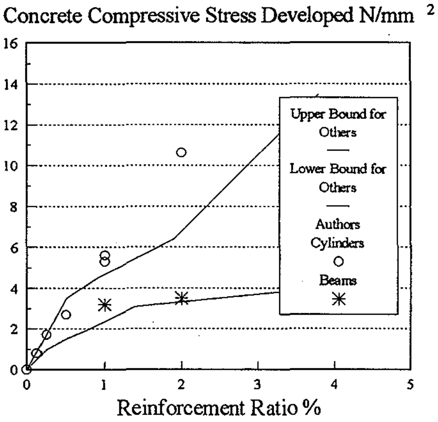


Figure 3 Stress developed in concrete compared to reinforcement ratio

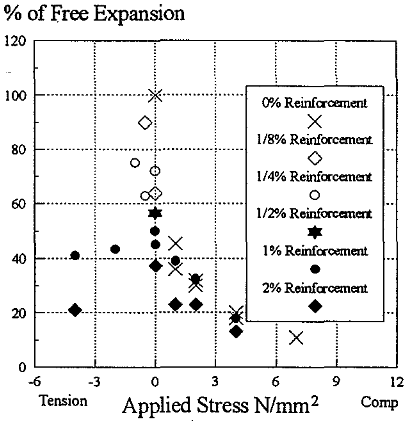


Figure 4 ASR expansion compared to applied stress for specimens with different reinforcement ratios

Applied Stress

Applied compressive stress has been shown to reduce expansion, however, Figure 8 of the IStructE report shows a significant variation in the relationship found by two separate researchers (Clayton *et al* 1990) and (Ng 1991). The research described here showed that this was due to size effects caused by the non-reactive skin. For smaller specimens the non-reactive skin was a greater proportion of the overall cross section than for larger specimen. Thus although the applied stress was calculated on the basis of overall cross section it was in fact restraining only the reactive heart concrete. This caused the actual stress on the reactive core of the smaller specimens to be greater than that on the larger specimens. This in turn caused the smaller specimens to expand less.

Tests were carried out to ascertain the compressive stress required to prevent any expansion. Even under an applied compressive stress of 7 N/mm^2 an expansion of 500 microstrain was recorded and it was estimated that to prevent any expansion a stress of 10 N/mm^2 was required. This compares well with that predicted by theory (Diamond 1989).

Tests were also carried out to ascertain the effects of applied tensile stresses on the expansion of reinforced specimens. It was shown that tensile stress increased the overall expansion of specimens. However, when non-ASR strains such as elastic and creep strains were considered applied tensile stresses did not increase ASR expansion. Indeed for some of the more heavily reinforced specimens the ASR expansion appeared to reduce under increasing applied tensile stress. This was due to some of the expansion potential being used to overcome the initial elastic and creep strain.

Combined

To demonstrate the effects of applied compressive or tensile stresses and reinforcement ratio Figure 4 was drawn. This plots the expansion of the cylinders tested as a percentage of the free (unreinforced under zero applied stress) expansion against applied stress for various reinforcement ratios. The expansion plotted has had the non-ASR strains deducted. This non-ASR strain was taken to be the strain experienced at 50 days, which was before ASR expansion started in the free expansion specimens.

Expansion Perpendicular to Restraint

The IStructE report questioned the effect of applied stress on the ASR expansion perpendicular to the direction of stress. In the tests reported here neither applied stress nor reinforcement ratio had a significant effect on the expansion perpendicular to the direction of stress/reinforcement. This is significant as it implies that when estimating expansion, the effects of stress perpendicular to the direction being considered can be ignored.

Effects on Member Strength

Section 5.3 of the IStructE report describes the effects of ASR on the properties of concrete members. Generally the report recommends that members can be assessed in a similar manner to unaffected members provided that the deteriorated concrete properties are used. The report also recommend that the effects of prestress, due to ASR, can be considered. This was researched further in the tests described here.

Reinforced cylinders, which had expanded due to ASR, were loaded, via their reinforcement, in tension. It was found that the prestress caused by ASR did increase the stiffness of the cylinders. For reinforcement ratios of up to 1/2% the behaviour of the specimens could be predicted by calculating the tension in the reinforcement, considering the surface expansions, and analysing the specimens using normal prestress theory. For the 1% and 2% reinforced specimens normal prestress theory over estimated the benefits. This was indicated by an earlier than predicted cracking load. It was concluded that this was due to the distribution of stresses along the specimens. At certain points along the reinforcement bar the reaction caused yielding and there would have been high compressive stresses in the surrounding concrete. Remote from these areas the concrete was under less stress. Cracking occurred through the concrete sections with the lowest compressive stress.

The 2% reinforced specimens cracked at about 50% of the load predicted by prestress theory and for this reason it is proposed that only 50% of the prestress calculated from surface strains is used in analysis. It was found that applied tensile stresses during the conditioning of the specimens increased this effective prestress. In addition, in a member with more than one bar, there will tend to be an averaging of prestress along the specimen. This will increase the ratio of effective prestress to prestress calculated from surface strains. The behaviour of specimens loaded in tension was not significantly affected by the expansion rate indicating that the prestress did not deteriorate, significantly, with time.

ASSESSMENT OF EXPANSION

From the previous discussion it can be seen that the current expansion is central to the assessment of the current structural properties of an affected structural member. The estimation of expansion is dealt with in section 7.3 of the IStructE report. The IStructE report proposed the use of the summation of crack widths to estimate expansion and this was investigated in the research discussed here. In addition a mathematical expansion model proposed by May *et al* (1991) was extended and compared to the results obtained from the test specimens

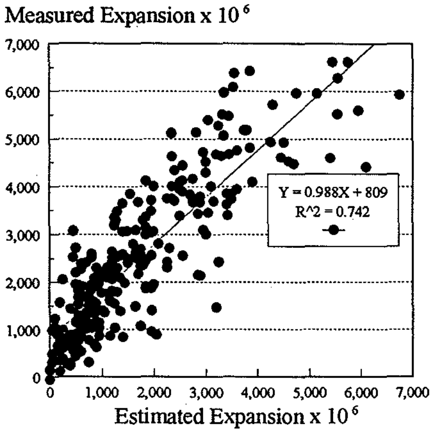
Use of Crack Widths

The IStructE report proposed that the crack widths along at least five lines no less than 1m long and 250 mm apart be summed and divided by the total length of all the lines to give an estimation of the expansive strain. It was not possible to have this number and length of line on the cylinders tested and the crack widths were measured along four lines 100 mm long and running along the length of the specimens. The estimated expansions compared with the actual expansions are shown in Figure 5.

The Authors proposed (Jones and Clark 1994) that the expansion demonstrated by the opening of a crack actually occurs perpendicular to the direction of that crack. Therefore the expansion in the direction of a line intersecting the crack at an angle is the width of that crack multiplied by the sine of the angle. The expansion estimated using this method is compared to the measured expansion in Figure 6. It can be seen that the correlation between estimated expansion and measured expansion is better in Figure 6 than Figure 5. However it can also be seen that the gradient of the best fit line increases from approximately 1 in Figure 5 to 1.2 in Figure 6. This is due to the inherent inaccuracies in the measurement of crack widths which have been discussed elsewhere (Jones and Clark 1994). It should also be noted that in both Figures the intercept is at approximately 800 microstrain. This represents the average expansion required before crack widths are measurable. It is proposed that when estimating the ASR expansion of a member, from crack widths, the following relationship be used:

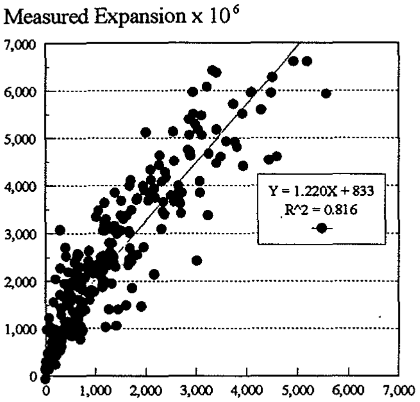
$$Expansion = \frac{1.2\sum crack\ widths \times \sin crack\ angle}{Length\ of\ reference\ line} + 800\ microstrain$$

It is likely that both the 1.2 and the 800 factor are slightly conservative for real members due to size effects.



Each point represents the average of one sample
i.e. four 100mm lengths

Figure 5 Expansion estimated from the sum of crack widths compared to measured expansion.



Each point represents the average of one sample
i.e. four 100mm lengths

Figure 6 Expansion estimated from the sum of crack width x sine of crack angle compared to measured expansion.

Use of Mathematical Models

A mathematical model for estimating the ASR expansion of reinforced members, proposed by May *et al* (1991), was extended by the Authors (Jones and Clark 1996) to include the effects of an applied conditioning stress. The model required a knowledge of the original Elastic modulus for both the concrete and the reinforcement, the free expansion of the concrete and the stress required to prevent any expansion (critical stress). The model predicted upper and lower bounds to the ASR expansion and showed fair correlation with the experimental results. However the amount of information required to calibrate the model probably restricts its use in the field.

CONCLUSIONS

1. The IStructE report overestimates the deterioration in compressive strength with ASR expansion. However it adequately predicts deterioration in elastic modulus.
2. The effects of restraint in both reducing expansion and developing prestress are far more dependent on the orientation of the restraint, in relation to the casting direction, than expansion rate.
3. The applied stress required to prevent ASR expansion is dependent on specimen size and may be as high as 10 N/mm^2.
4. Applied tensile stresses increase overall expansion but have little effect on ASR expansion.
5. Applied stresses have no significant effect on ASR expansion perpendicular to their direction of application.
6. Prestress is developed in reinforced concrete expanding due to ASR. However this prestress may be only 50% of that calculated from consideration of surface strains.
7. Estimation of ASR expansion from crack widths can be improved by the consideration of the crack angle.

ACKNOWLEDGEMENTS

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ALKALI-SILICA REACTIONS IN DAMAGED CONCRETE
Static and dynamic tests
Material investigations

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ABSTRACT

The purpose of this investigation was to link observations in the field, results found in the laboratory and results from earlier investigations concerning the effects of frost and ASR on the static and dynamic strengths of structural members. The Road Directorate decided to carry out laboratory tests on a damaged bridge deck, and in June 1993 four concrete beams were cut out from the deck of a road bridge, constructed in 1977. The investigations were carried out in close cooperation between the Road Directorate and the Civil Engineering Department of the Danish Engineering Academy, Physics and Materials Science Section. The structure showed severe damage due to frost and ASR, and it was therefore decided to demolish the bridge deck and reconstruct it in 1993.

The project comprised the following tests:

- Evaluation of earlier investigations of the bridge deck
- Dynamic and static laboratory tests in bending on large structural beams
- Compressive strengths of cores
- Evaluation of the AS-reactivity of the aggregate
- Evaluation of the influence of ASR on watersorption
- Evaluation of damage - thin sections and polished sections

Keywords: Alkali-silica reactions, beam, bridge,cracking, durability, dynamics, frost, reinforced concrete, strength, test.

INTRODUCTION

In 1984 - 1989 the Ministry of Transportation, (the Road Directorate) carried out research and examinations of reinforced concrete beams with alkali-susceptible aggregates (Ministry of Transp. 1990). Axial sections on cores and thin sections has been one of the methods for assessing the condition of a structure. However, it is difficult from laboratory research alone to estimate the extent to which damage implies loss of load- carrying capacity. In order to link results in the laboratory and the theory of structural design, the Road Directorate decided to carry out investigations on damaged reinforced concrete beams from a structural member damaged by ASR (Ministry of Transp. Rp. 12, 1994).

BACKGROUND

This paper describes some of the static and dynamic tests and materials research, which have been carried out on a bridge deck from a bridge owned by the Road Directorate. The bridge deck was demolished and reconstructed in the summer of 1993.The abutments were repaired with in-situ cast concrete.

BRIDGE DESIGN

The bridge was built on 8 July 1977 ($42m^3$ concrete) as an in-situ cast single-span two-hinged frame-tunnel, and founded on solid bottom with compressive reinforcement between the frame foundations. Data for the reinforced concrete - see table 1. Vacuum-dewatered concrete was cast on the bridge deck. The test beams used in the investigation were cut out of the bridge deck with a watercooled diamond saw.

Table 1. Bridge-deck concrete.

Specifications [kg/m^3]	**Compression strength** σ_{ck} **= 30 MPa**
Cement PC(A/L/S)	340
Water (V/C $\leq$ 0.55)	128
Fine agg. 0-8mm (alkali-susceptible)	710
Coarse agg. 8-32mm (non-susceptible)	1106
Aerocon & Sparcoplast	36 ml / m^3 & 360 ml / m^3
Reinforcement is Tentorsteel $\sigma_{yk} \geq$ 560MPa and mild steel $\sigma_{yk} \geq$ 240MPa.	

MATERIAL TESTS AND RESULTS

In this paper only some of the measured material properties will be presented.

COMPRESSIVE STRENGTH

At a given maturity, a lower strength can be expected for drilled cores than for normally cast cylinders. The relation between drilled cores and cast cylinders can be seen in table 2.

Table 2. Results from compression tests 1977 and 1993.

Date	Type	Strength [MPa]	Correction	Strength $\sigma_{c,k}$ [MPa]
July 1977	cast	31.5	0.8	25.2
Sept. 1977	drilled	23.7	1.0	23.7
Nov. 1993	drilled	21.2[1]	1.0	21.2
[1] 8 drilled cylinders, 2 test results were cancelled. Empirical deviation = 2.7 MPa				
$\sigma_{c,cast} = \mathbf{1.25} \cdot \sigma_{c,drilled} \Rightarrow \sigma_{c,drilled} = \mathbf{0.8} \cdot \sigma_{c,cast}$				

Sorption properties of concrete

Deleterious ASR in concrete occurs when alkali-ions from the cement dissolve in water and produce alkalihydroxyl, which reacts with alkali-silica-reactive aggregate. The result can be alkali-silica-gel, which can absorb very large quantities of water. The absorbed water can expand and produce cracks in the concrete. According to (Anders Nielsen 1993) the chemical reaction will have two effects. The amount of evaporable water will be reduced as the ASR-gel contains some non-evaporable water. The ASR-gel will contribute to the hygroscopic binding capacity of the concrete, and therefore the watersorption curve will be higher than that for a concrete without ASR-gel. The results show that the investigated concrete contains ASR-gel, because the watersorption curve is on a higher level than that of the reference concrete. Chemical shrinkage

Chemical shrinkage was measured on reused sand (0-4mm) from acid-treated concrete specimens. A mean value of 0.37ml dissolved silica/kg sand showed that more unreacted alkali-silica exists. Therefore the concrete has the potential for further reactions and expansions. This agrees with observations in thin sections, where unreacted aggregates without cracks were observed; thin sections from 1985 compared with thin sections from 1993 showed that the increase of alkali-silica reactions has been low, possibly because the alkali-ions from the cement have been used up. New reactions can be started by adding alkali-ions; as an example it can be mentioned that NaCl is used in large amounts for deicing in Denmark

STATIC TESTS AND RESULTS

A sketch of the reinforcement in the test beams is shown in figure 1.

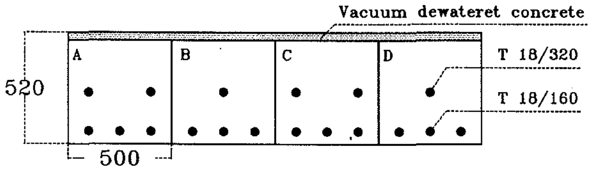

Figure 1. The reinforcement in the cross-section of the testbeams (mm)

The calculated moments at failure ($M_{v,calculated}$) for the test beams: (A: 319kNm, B: 262kNm, C: 223kNm & D: 210kNm). The moments differ due to different areas of reinforcement (A_s) and effective heights (h_{eff}). In the tests an estimated value of the failure-moment M_u = 220kNm was used, and the calculated failure-load ($P_{failure}$) excludes dead-load (P_g = 6.2kN/m), as $P_{failure}$ is the load on the beam at failure.

Test set-up and results

Figure 2 show the set-up used in the static- and the dynamic tests and the arrangement of measuring equipment. The beams were tested in 10 steps and for each load-level (10%, 20%... of estimated failure-load) the load, deformations, bending and cracks were measured.

The following measurements were made (at approx. ½ hour intervals):

- Load measured with tension and compression cells (25ts-Eilersen)
- Deflections measured with three dial-gauges (0.01 mm)

- Longitudinal deformations at the top and bottom of the beams with Demec mechanical strain gauge (Instr. nr. 482; accuracy ± $3\cdot 10^{-6}$, $1\cdot 10^{-5}$ strain is one division)
- Observation and registration of cracks with a crack-gauge (0.05 mm)
- Photogrammetric measurement of deflections and strain (sensitivity approx. 0.5 mm)

Measured and calculated parameters:

1. Crack-width: w [mm]
2. Strain:
   $\varepsilon = (l_n + l_{n+1})\cdot 1\cdot 10^{-5}$, where l_n and l_{n+1} are measurings at the n'te load level and the (n+1)'te load level respectively

The results of the strain-measurement will not be shown in this paper.

3. Curvature κ [mm^{-1}]
4. Bending u [mm]

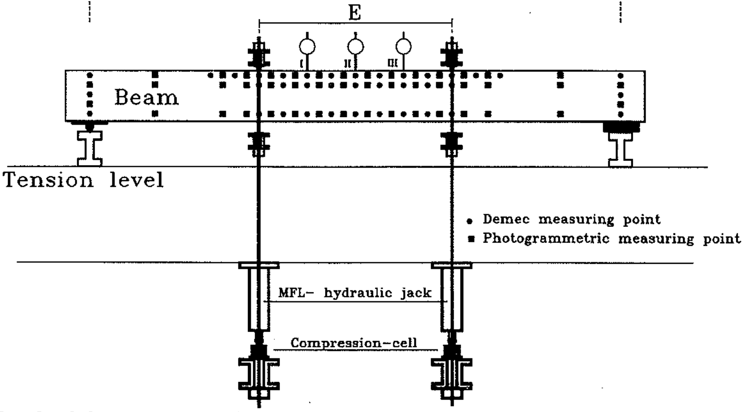

Figure 2. Sketch of the test rig for dynamic/static tests and measuring points on beam. U= 4800mm (A & B); U= 5460mm (C & D); E = 2000mm (A, B, C & D)

ad. 1& 3 Crack-width and Curvatures

Table 3 shows corresponding values of calculated and measured crack-widths for beams A and B. The curvatures (5/) κ [mm^{-1}] of a beam can be calculated, and the curvature (κ) is used for the calculation of the bending (u):

$$\kappa = 1/R = \sigma_s /[E_{sk} (h_{eff} - h_c)] \quad \& \quad u = 1/10 \cdot \kappa \cdot l^2 \text{, where}$$

σ_s	Tensile stress in the reinforcement [MPa]
E_{sk}	Modulus of elasticity of the reinforcement [MPa]
h_{eff}	Effective height of the cross-section of the beam
h_c	Height of the compression-zone [mm]
l	Span of the beam [mm]

There is fairly good agreement between the calculated (DS411) and the measured crack widths but some divergence on the curvatures. In general all the measurements are conservative compared to the calculated results.

Ad. 4. Bending u[mm]

Table 4 shows the calculated and measured bending for beams A and B at each load level. The results from the photogrammetric method are not presented in this paper.

Table 3. Crack-width and bending for beams A & B.

Crack-width w[mm]				**Bending $\kappa \cdot 10^{-7}$ [mm]**		
Level	Calculated	Measured		Calculated*	Measured curvature κ**, at point II.	
		Beam A	Beam B		Beam A	Beam B
1	0.11	none	none	10.8	2.3	0.9
2	0.23	none	none	21.5	3.3	2.2
3	0.34	none	< 0.05	32.3	4.1	3.3
4	0.46	none	0.10	43.1	5.9	5.7
.	.	.	.	.	.	.
9	1.02	0.10- 0.90	0.10-0.75	96.9	27.1	26.1
10	1.14	> 0.90	> 0.75	107.8	29.4	-
DS 411: $w = 5\cdot10^{-5}\cdot\sigma_s\cdot a_w^{-1/4}$; σ_s is tensile stress in the reinforcement and a_w is a crack parameter				κ* $= \sigma_s /[E_{sk}(h_{eff} - h_c)]$ κ_{1-2}** $= \varepsilon_1 - \varepsilon_2 / 2\cdot h_{1-2}$		

The agreement between calculated and measured bending (table 4) is rather good. At higher load levels the measured deflections are higher than the calculated. The photogrammetric measurement has the same order of magnitude.

Table 4. Bending of Beams A and B by different measuring methods

Bending [mm] Center-line of beams (point II).					
Load	**Calculated** $u_{max} = 1/ 10\cdot\kappa_{max} \cdot l^2$	**'Measured curvature''** u_{max}		**Dial-gauge** u_{max}	
		A	B	A	B
1	2.49	0.54	0.21	1.01	0.81
2	4.95	1.30	0.71	1.56	1.67
3	7.44	2.25	1.48	2.09	4.24
4	9.93	3.61	2.71	2.79	5.44
.	.	.	.	.	.
9	22.33	24.32	20.96	9.94	18.17
10	24.84	31.07	-	12.25	-

DYNAMIC TESTS AND RESULTS

Loads from traffic will reveal the dynamics of a bridge. Two tests (beams C and D) were carried out to illustrate how dynamic loads influence a damaged bridge construction. Failure from a dynamic load is also called fatigue fracture from a large number of load-oscillations (10^5 to 10^8 cycles). It is therefore important to investigate variations of stress, because oscillations of stress can cause microscopic fractures in the structure even if the stress is lower than the stress which would cause fracture at a static load. A dynamic fracture differs from a static fracture because repeated oscillations of loads give irreparable microscopic cracks in the concrete. The dynamic tests were of the following type:

- Dislocated oscillations-influence, where the max. and min. tensions have the same sign but are non-zero. The used test rig is seen in figure 4.

Loading

Because of the ASR-cracks in the test beams it was expected that a dynamic load would be critical. The first dislocated oscillations were therefore carried out at a level of 25-50 kN on each yoke at a low frequency, and continued with various levels and frequencies (table 5). The loads on the test beams correspond to the loads from lorries with a total weight of 9.000 kg , 12.000 kg and > 25.000 kg.

Development of fracture for beams C and D

The first cracks were observed after approx. 2 hours (25-50 kN). In general the cracks increased in the first few hours after changing the load and the frequency, and thereafter stabilized. When the tests were stopped many of the cracks reached the upper level of the reinforcement. Table 6 shows the dislocated oscillations used and the swings obtained compared to numbers of truckaxles of 4o kN

Table 5. Loading and numbers of oscillations for beams C and D

Load [kN]	**Beam C**			**Beam D**		
	Frequency $f_C[s^{-1}]$	Time [s]	Oscillations N_{iC}	Frequency $f_D[s^{-1}]$	Time [s]	Oscillations N_{iD}
25-50	0.041	353.280	14.532	0.118	1.200	142
25-50	0.076	160.860	12.168	0.204	600	122
25-50	0.171	166.140	28.352	0.365	86.700	31.646
25-50	0.345	869.040	299.668	0.435	7.860	3.419
25-50				0.608	514.800	312.998
38-64	0.401	173.160	69.542			
50-75	0.383	108.300	41.494			
25-50	0.383	325.260	124.621			
50-75	0.437	367.740	160.585	0.591	515.400	304.601
62-77	0.980	254.940	249.941			
75-100				0.653	621.240	405.670
	N_{iC}		**1.000.903**	N_{iD}		**1.058.598**

For a bridge deck similar to the deck in this investigation, axle-loads were registered in a traffic census. They have been recalculated to a standard axle load of 40 kN which gives an annual average 24hour-traffic of (365· 5714) = 2.085.610 "40kN-axles".

Dividing the number of oscillations ($\Sigma N_{4000\ kg}$) by the annual average 24-hour-traffic gives the following values of n for beam C and D:

n_C = 4.810.776 / 2.085.610 = 2.3 years
n_D = 13.136.188 / 2.085.610 = 6.3 years

Table 6. Load (P_i), passages (N_i), and 4000 kg axles for Beams C and D.

Load P_i [kN]	$P_{4000\ kg}$ [kN]	**Beam C**		**Beam D**	
		$N_{i,C}$	$N_{4000\ kg}$	$N_{i,D}$	$N_{4000\ kg}$
25	40	239.672,5	36.571	174.163,5	26.575
38		34.771	28.321		
50		340.712	831.816	326.464	797.031
62		124.970,5	721.331		
64		34.771	227.875		
75		101.039,5	1.248.810	355.135,5	4.389.340
77		124.970,5	1.716.052		
100				202.835	7.923.242
Σ	$^*N_4 = (P_i/P_4)^4 \cdot N_i$	1.000.907	4.810.776	1.058.598	13.136.188
* Jf. AASHO, 1962 /11/.					

Neither of the two test beams were crushed at the end of the dynamic test, which means that the construction might have behaved well for a further 2-7 years of traffic. Under the circumstances for the test beam D - load from a truck on 500 mm bridge deck width - the bridge might have behaved well for 5-15 years of traffic; in this assumption climatic deterioration has not been taken into account.

DISCUSSION

The material tests covered cracks and crack development, development of strength and static/dynamic tests. Generally microcracks can be observed in concrete, and are caused by:

- Thermal stresses during curing.
- Frost action and use of deicer.
- Chemical reactions such as alkali-silica reactions
- Drying shrinkage connected with the hydration of cement.

Hence investigations of drilled cores have shown cracks parallel to the surface moving inwards concurrently with the intrusion of water and dissolved salt. This can result in differences between the compressive strength of a normally stored cylinder, a cylinder stored in a NaCl-solution and a drilled core depending on the direction of drilling. In this investigation most of the cracks were parallel to the surface and the bottom of the bridge deck, and a reduction in the compressive strength of 20-55% was therefore expected. Results from compression tests of the concrete 1977 - 1993 (table 2) do not show reductions, and therefore it cannot be concluded that ASR had any influence.

The bridge was built as a single-span two-hinged frame-tunnel and the bridge deck was fully fixed at the abutments. Loads on this system will give thrust-forces in the bridge deck and negative moment at the terminal points of the bridge deck. In the static and dynamic tests the static system was a simply supported beam; that a plate can take up a greater force than a beam was taken into account. The original construction was more capable of taking up the traffic load than the test beams indicate. Loads from heavy lorries have a greater influence on the fatigue strength than light traffic loads. In the tests it was attempted to force fatigue failure by loading the beams with a dislocated oscillation - 25-100kN and frequencies 0.971-

0.041Hz (period 1-25sec.). The max. load from a rear axle is 8 tons and the max. load from a front axle is 6 tons, which means that a 1 metre bridgedeck width can take up approx. 8 tons (two rear axles divided by two). From this point of view the chosen load levels are reasonable, but the chosen frequencies are not quite realistic. As long as the chosen frequency is not too far from the real frequency, it can be estimated that the frequency has only a slight influence on the fatigue failure, which is supported by the literature. The slower period may have caused the load to act quasi-dynamically, and the absence of shock excitation may have resulted in the test beams resisting a larger number of oscillations than they would have done in a purely dynamic test.

CONCLUDING REMARKS

The observations can be summarized as follows:

- No reinforcement corrosion was observed, the increase of ASR has been rather low since 1985 and the intrusion of chloride is less than 3 mm from the surface.
- Compression tests show that compressive strength has not been appreciably lower during the lifetime of the bridge. The deformation of the drilled cores was measured 2½-5 times larger than in non-damaged concrete.
- The observed damage is due to defects in the casting and the compacting of the concrete. These defects have resulted in damage from frost and ASR.
  In thin sections and impregnated polished sections there were observed:
  - A large amount of aggregate which was not surrounded with cement paste.
  - Inadequate air-entrainment and ASR-gel in cracks and pores.
  - Cracks indicating that the fresh concrete was inhomogeneous and that it must have been difficult to compact the concrete.
- In the static tests, failure occured in the test beams at the calculated level approx. 220kNm.
- In general the measured crack widths are smaller than the theoretical.
- The behaviour of the beams under static and dynamic loading was as expected.
- Neither of the two test beams were crushed at the end of the dynamic tests, which means that the construction might have behaved well for a further 2-7 years of traffic. The bridge as a whole might have behaved well for a further 5-15 years of traffic, if climatic deterioration is not taken into account.
- Static and dynamic load-carrying tests have shown that the observed damage has little influence on the load-carrying capacity of the structural members.

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MECHANICAL PERFORMANCE OF ASR AFFECTED NEARLY FULL-SCALE REINFORCED CONCRETE COLUMNS

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ABSTRACT

Mechanical characteristics of ASR affected reinforced concrete column were experimentally investigated in comparison with those of normal concrete column. Those test results indicate; (1) ASR improved the ductility of reinforced concrete column. (2) Although ASR reduced the compressive strength about 40 to 50 percent, ultimate load bearing capacity (N'ou) of the column was not so reduced than that we had expected. When columns have small transverse reinforcement, however, transverse reinforcement yielded before axial reinforcement yielded, so that N'ou was about 30 percent lower than that of normal concrete column. (3) Larger the size of test unit, effect of ASR on mechanical characteristics of reinforced concrete column became smaller.

Keywords: ASR, ductility, reinforced concrete columns, stress-pass, ultimate load

INTRODUCTION

During last some 15 years, many concrete structures deteriorated by alkali aggregate reaction (ASR) have been confirmed at various regions in Japan. As has been well known, compressive strength of ASR affected concrete decreases to about one half that of normal sound concrete. Reduction in modulus of elasticity is larger than that in compressive strength. Many experimental test results on reinforced concrete beam affected by ASR, subjected to bending moment, have been reported. There are, however, few researches on the reinforced concrete column subjected to axial force.

On the other hand, ultimate load bearing capacity of reinforced concrete column made with normal concrete is known almost proportional to compressive strength of concrete. Therefore, the authors fear that decrease in compressive strength of concrete due to ASR will cause a significant loss on bearing load capacity of reinforced concrete column.

Spirally reinforced and tied columns ($\phi 20\times 60$ cm and $20\times 20\times 60$ cm sized) were tested. Variables were axial reinforcement ratio, transverse reinforcement ratio, water-cement ratio and expansion of concrete due to ASR. Finally, spirally reinforced concrete columns $\phi 85\times 270$ cm and $75\times 75\times 240$ cm tied columns in size were tested.

TEST DETAILS

Materials and concrete

Ordinary portland cement having alkali content of 0.62% was used. Pit sand and river sand were used as fine aggregate. Reactive (determined by chemical method) crushed chert was used as coarse aggregate for ASR concrete. Non-reactive crushed stone for normal concrete. Sodium hydroxide was used as additional alkali. Water-cement ratio and alkali content (Na_2Oeq.) of concrete used were shown in Table 1.

Table 1 Water-cement ratio and alkali content of concrete

Mix No.	W/C (%)	Na_2Oeq.(kg/m^3)
N1	94.1	1.16*
N2	74.6	1.32*
N3	58.8	2.11*
N4	50.0	2.42*
A1	65.0	7.00
A2	58.8	7.00
A3	50.0	7.00
A4	40.0	7.00

* Alkalinity from cement
N; Normal concrete, A; ASR concrete

Test specimens

Compressive strength was determined by $\phi 10\times20$ cm cylindrical specimens. Free expansion of concrete was measured by $10\times10\times40$ cm prismatic specimens. Reinforced concrete columns were, for the most part, $\phi 20\times60$ cm sized spiral columns and $20\times20\times60$ cm sized tied columns, as shown in table 2 and 3. Variables were axial reinforcement ratio (P), transverse reinforcement ratio (Ps or Ph), water cement ratio of concrete (W/C) and free expansion of concrete (Ex.). Finally, four spirally reinforced concrete columns $\phi 85\times270$ cm and four $75\times75\times240$ cm tied columns were prepared to investigate the size effect.

All specimens and columns made with ASR concrete were cured under ambient condition of 38℃ and 100% RH. Normal concretes were cured in the 20 ℃ water tank.

ASR concrete columns were tested as of free expansion of concrete 1500μ, except unit No. S17, S22, H19, H21 and H27. Nearly full-size column specimens were tested

Table 2 Experimental program in spirally reinforced concrete column test and its results

Unit No.	P=Ast/Ae (%)	Ps=Aspe/Ae (%)	Concrete	f'c (MPa)	N'ou (KN)
S1	0.95	1.89	N3	37.8	1259
S2	1.44	1.89	N3	37.8	1309
S3	1.93	1.89	N3	37.8	1333
S4	3.03	1.89	N3	33.4	1255
S5	1.44	1.13	N3	33.0	1091
S6	1.44	1.89	N3	33.0	1177
S7	1.44	2.83	N3	33.0	1363
S8	1.44	1.89	N1	13.7	628
S9	1.44	1.89	N2	22.3	903
S10*	2.25	1.04	N3	32.9	19.3 MN
S11	1.14	1.89	A2	19.1	945
S12	1.71	1.89	A2	19.1	1028
S13	2.30	1.89	A2	19.1	1047
S14	1.71	0.93	A2	20.8	863
S15	1.71	1.13	A2	22.2	924
S16	1.71	1.89	A2	22.2	1074
S17	1.71	1.89	A2	21.2	1170**
S18	1.71	2.80	A2	20.8	1194
S19	1.71	1.89	A1	15.2	898
S20	1.71	1.89	A3	25.6	1192
S21	1.71	1.89	A4	33.2	1400
S22*	2.25	1.04	A2	23.1	18.4 MN

* $\phi 85\times270$cm sized
** free expansion of concrete: Ex.=2700x10^{-6}

Table 3 Experimental program in tied column test and its results

Unit No.	P=Ast/Ac (%)	Ph=Phoe/Ac (%)	Concrete	f'c (MPa)	N'ou (KN)
H1	0.68	0.41	N3	33.7	1373
H2	1.35	0.41	N3	33.7	1398
H3	2.03	0.41	N3	33.7	1516
H4	3.80	0.41	N3	33.7	1683
H5	1.35	0.27	N3	32.0	1348
H6	1.35	0.41	N3	32.0	1398
H7	1.35	0.82	N3	32.0	1441
H8	1.35	1.37	N3	32.0	1571
H9	1.35	0.41	N1	14.5	755
H10	1.35	0.41	N2	24.0	1130
H11	1.35	0.41	N4	43.4	1816
H12*	1.69	0.29	N3	32.9	19.4 MN
H13	0.68	0.41	A2	22.4	1128
H14	1.35	0.41	A2	22.4	1165
H15	2.03	0.41	A2	22.4	1187
H16	3.80	0.41	A2	22.4	1351
H17	1.35	0.27	A2	20.2	1081
H18	1.35	0.41	A2	20.2	1124
H19	1.35	0.41	A2	19.2	1120**
H20	1.35	0.82	A2	20.2	1185
H21	1.35	0.82	A2	19.2	1281**
H22	1.35	1.37	A2	20.2	1269
H23	1.35	0.41	A1	16.5	915
H24	1.35	0.41	A2	20.7	1149
H25	1.35	0.41	A3	28.8	1493
H26	1.35	0.41	A4	36.8	1818
H27*	1.69	0.29	A3	23.1	16.7 MN

* $75\times75\times240$cm sized
** free expansion of concrete: Ex.= 2500x10^{-6}

at free expansion came up to about 3000μ. Axial and transverse strain of concrete surface and of reinforcing bars and displacement of the columns at each applied load stage were measured. Initial strain of reinforcing bars of ASR concrete columns before loading were also measured.

RESULTS AND DISCUSSION

Ultimate load bearing capacity of column

Ultimate load bearing capacity (N'ou) of each column and compressive strength of concrete at the time of column tested are in table 2 and 3. Hoop reinforcement ratio (Ph) in the Table 3 is defined as follows:

$$Ph = Ahoe / Ac = As \cdot L / s / Ac \qquad (1)$$

where Ph = hoop reinforcement ratio (%)
Ahoe = effective area of hoop reinforcement (cm^2)
As = area of hoop bar (cm^2)
L = one hoop length (cm)
s = pitch of hoop reinforcement (cm)
Ac = total area of concrete (cm^2)

Compressive strengths of ASR concretes are 35 to 50% lower than those of normal concretes at the same water-cement ratio. Reduction in ultimate load of ASR affected columns, however, are 1 to 25%. Especially, when columns have adequate transverse reinforcement, the reduction ratio does not increase, even if the expansion of concrete is larger. The reduction ratio of nearly full-size spiral column is small as 5%.

The relationship between compressive strength and ultimate load of column is shown in Fig.1. Ultimate load is almost proportional to compressive strength. Ultimate loads of ASR affected concrete columns are 5 to 30% higher than those of normal concrete column at the same compressive strength.

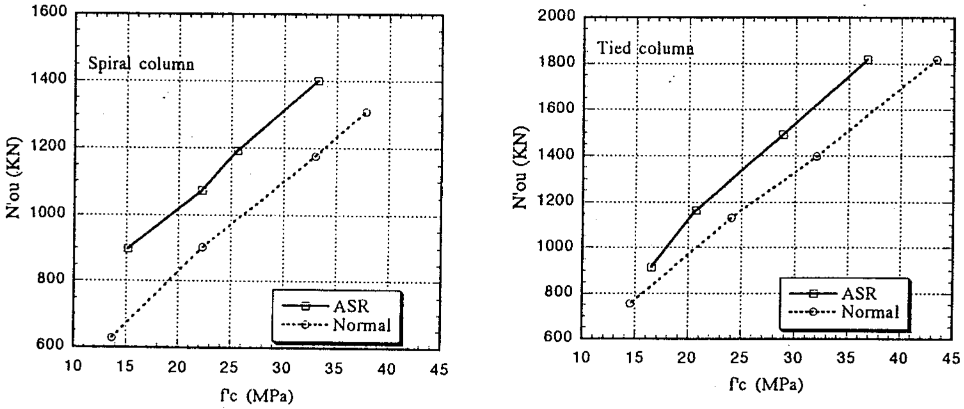

Fig.1 Effect of f'c on N'ou of column

Displacement of column

Typical examples of applied load versus strain curve of columns are shown in Fig.2. Strains of ASR concrete columns at a certain load are higher than those of normal concrete columns. As the Young's modulus of ASR concrete is remarkably decreased, strains become high. Consequently, displacement of the columns at a certain load are larger than those of normal concrete column. When columns have adequate transverse reinforcement, however, the ductility of ASR concrete columns is higher than that of normal concrete columns, as shown in Fig.3.

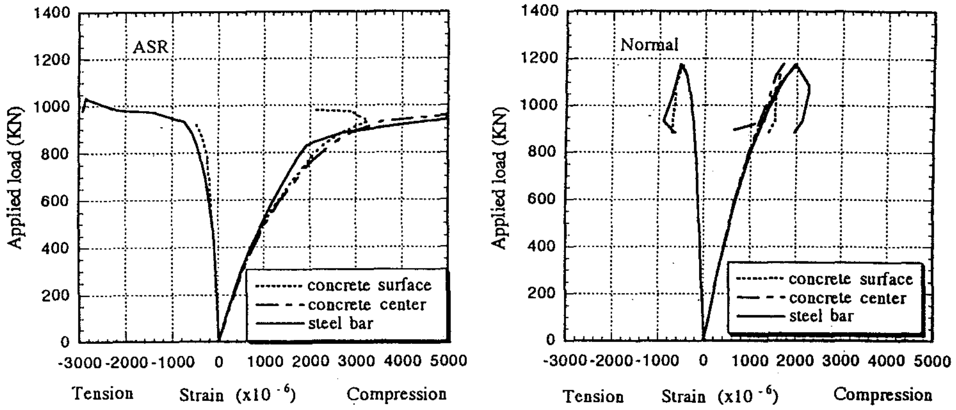

Fig.2 Typical examples of applied load versus strain curve

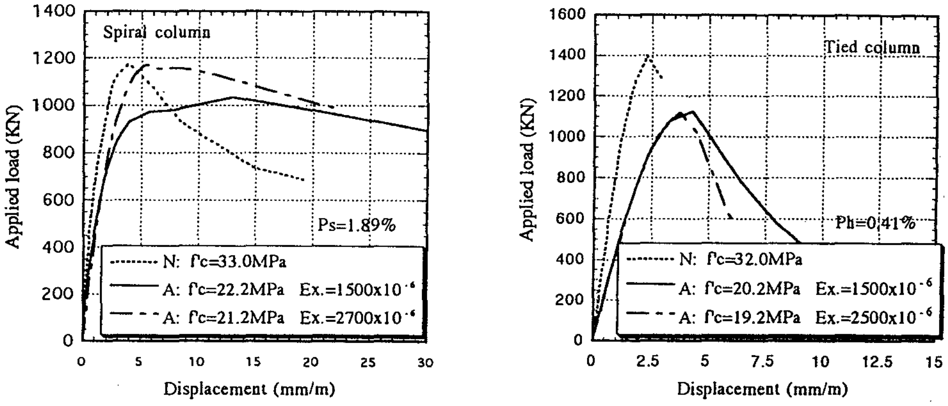

Fig.3 Effect of expansion of concrete on applied load versus displacement

Stress-pass of core concrete

Confining stress σ_2 ($=\sigma_3$) to be generated in circular core concrete of spirally reinforced columns, is given as follows:

$$\sigma_2 = (2\,A_s \cdot \sigma_{sp}) / (s \cdot D_e) \qquad (2)$$

where A_s = area of spiral reinforcement (cm^2)
σ_{sp}= stress of spiral reinforcement (MPa)
s = pitch of spiral reinforcement (cm)
De = effective diameter of spirally reinforced concrete column (cm)

Stress of spiral reinforcement (σ_{sp}) is determined by strains of spiral reinforcement as shown in Fig.2 and Young's modulus of steel. On the other hand, axial stress of core concrete σ_1 can be calculated as follows:

$$\sigma_1 = (N'_o - N'_s - N'_{cover}) / (\pi D_e^2 / 4) \qquad (3)$$

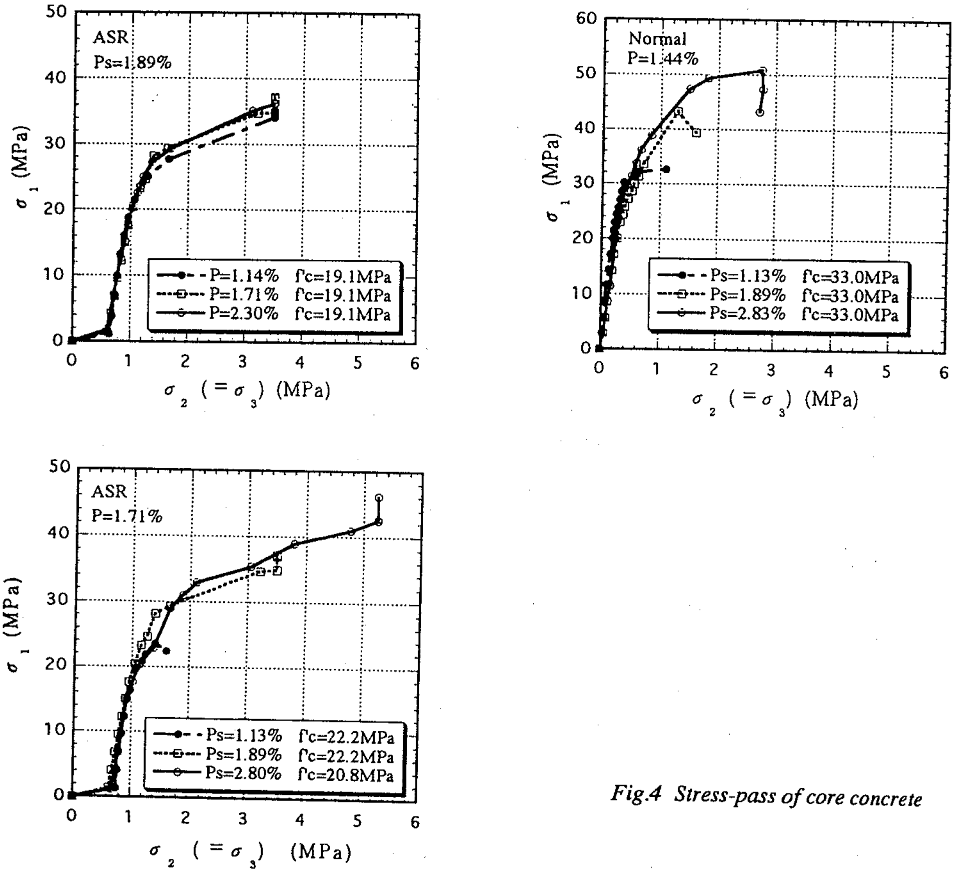

Fig.4 Stress-pass of core concrete

where N'_o = applied load (N)
N'_s = share of load by axial reinforcement (N)
N'_{cover} = share of load by cover concrete (N)

N'_s is obtained by strains of axial reinforcement, Young's modulus and total area. N'_{cover} is calculated by stress-strain curve which is determined with plain concrete column and area of cover concrete.

As shown in Fig.4, σ_1 increases with σ_2 ($=\sigma_3$) for ASR concrete column. Substituting yield strength of spiral reinforcement (f_{spy}) for σ_{sp} in equation (2), the maximum confining stress σ_{2max} can be obtained for this case. Stress-pass of normal concrete starts from origin, but for ASR concrete a sharp breakpoint is observed at about 2 MPa of axial stress and 0.6 to 0.8 MPa of confining stress. This may be caused by the initial stress in ASR concrete before loading. Stress-pass is hardly affected by axial reinforcement ratio (P) and σ_{2max} increases with spiral reinforcement (P_s), so that σ_{1max} increases with P_s. In normal concrete columns, however, increments of σ_1 are smaller than those of ASR concrete column, at the same P_s since σ_2 does not reach to σ_{2max}. Consequently, N'_{ou} of ASR concrete columns becomes higher than those of normal concrete columns at the same compressive strength of concrete (Fig.1).

Three dimensional stress of column

Confining stress of concrete in tied column cannot be obtained easily. Three dimensional stress diagram which express transverse reinforcement stress (σ_{sp} or σ_{sh}), axial reinforcement stress (σ_s) and axial concrete stress (σ_1) is devised (Fig.5). Axial stress of core concrete of tied column is calculated as : $\sigma_1 = (N'_o - N'_s) / A_c$.

In normal concrete columns, stress of axial reinforcement reaches to the yield value before that of transverse reinforcement does. Prior to loading, on the other hand, the initial tensile stresses are generated in reinforcements of ASR affected columns. At 1500μ of concrete expansion, axial reinforcement yields before transverse

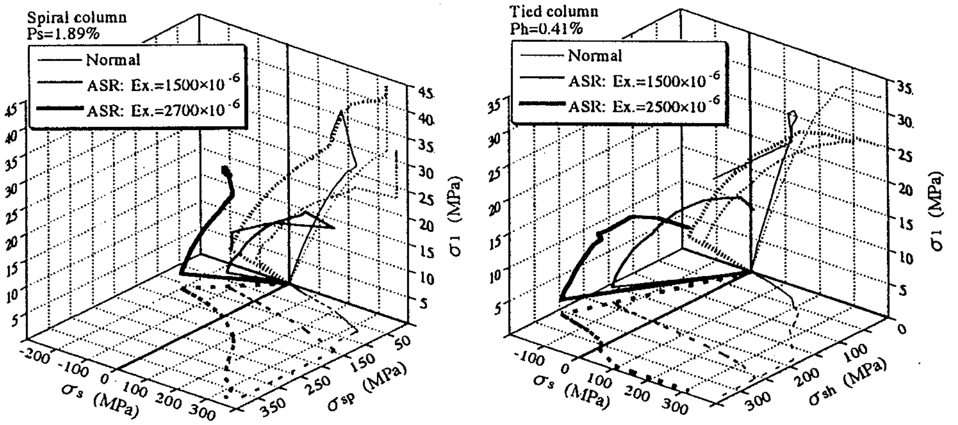

Fig.5 Three dimensional stresses of different expansion of concrete

reinforcement yields. Stress of transverse reinforcement σ_{sp} or σ_{sh} of ASR columns is larger than that of normal columns. Consequently, confining stress becomes higher at the time of yielding of axial reinforcement. At 2700μ of expansion, when columns have adequate transverse reinforcement, axial and transverse reinforcement yield almost at the same time. When columns have small transverse reinforcement, however, transverse reinforcement yields before axial reinforcement yields. In this case, confining stress does not increase more, so the ultimate load bearing capacity of the column does not increase thereafter.

Performance of nearly full-scale test specimen

Applied load versus displacement curve of nearly full-scale column is shown in Fig.6. Displacement of ASR affected columns after the ultimate load, is larger than that of normal concrete column. The ductility of the columns is improved by ASR. As for

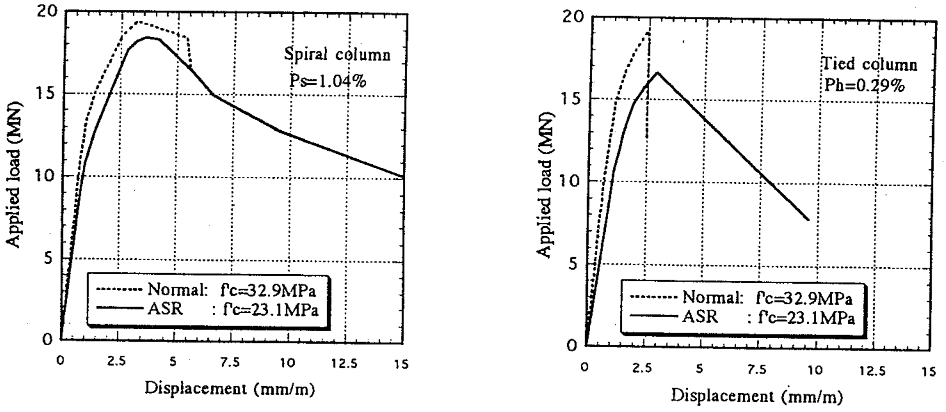

Fig.6 Applied load versus displacement of nearly full-scale test units

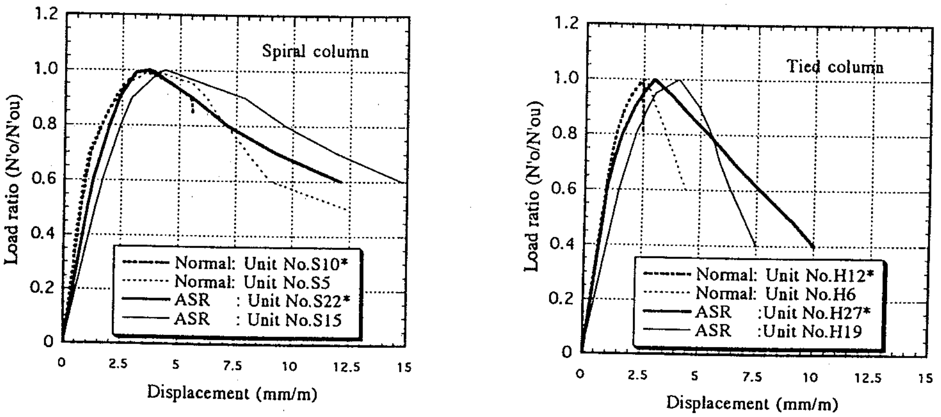

Fig.7 Effect of test unit size on load versus displacement

tied column of normal concrete, corner of hoop reinforcement is fractured in a brittle manner and load is suddenly decreased.

In normal concrete column, load ratio versus displacement curve is about the same regardless of test specimen size. Also it is clear that load ratio corresponding to the same displacement is higher for ASR affected column than that of normal concrete column. In ASR affected column, however, displacement of nearly full-sized column is smaller than that of small size test unit, as shown in Fig.7.

CONCLUSIONS

Conclusions are summarized as follows within the scope of the experimental works.

(1) The ductility of reinforced concrete column is improved by ASR.

(2) Initial tensile stress before loading in axial reinforcement due to expansion of concrete is profitable for load bearing capacity of columns, because a part of compressive stress caused by external force is off-set.

(3) Prior to loading, confining stress (σ_2) is generated in ASR affected core concrete. When columns have adequate transverse reinforcement, σ_2 is higher than that of normal concrete at the final loading stage, then the ratio of σ_1 to f'_c of ASR concrete is higher than that of normal concrete. Accordingly, reductions in ultimate load bearing capacity of ASR affected columns are 1 to 20%, though the compressive strength of concrete is reduced 35 to 50% by ASR at the same water-cement ratio. When columns have small transverse reinforcement, however, transverse reinforcement yields before axial reinforcement yields, so that σ_1 is not so increased. In those cases, ultimate load bearing capacity of columns is about 30% lower than that of normal concrete column.

(4) Ultimate load bearing capacity of ASR affected columns is 5 to 30% higher than that of normal concrete column at the same compressive strength.

(5) Larger the size of test specimun, effect of ASR on mechanical characteristics of reinforced concrete column becomes smaller.

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THE INFLUENCE OF STRESS INTENSITY AND TIME OF APPLICATION ON THE MECHANICAL PROPERTIES OF ASR AFFECTED CONCRETE

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ABSTRACT

The paper describes an experimental investigation to examine the effects of different stress levels on the mechanical properties of alkali silica reactive deteriorated concrete cores. Two concrete blocks made with ASR concrete were left for one year during the period of reaction. The blocks were then cored perpendicular to the direction of casting and these cores were tested to determine stiffness damage parameters, ultrasonic pulse velocity, crushing strength and weight gain. The cores were then subjected to different stress levels and left in this stressed state for another whole year and were then tested again. The results indicate that the application of permanent stress to ASR cores when the ASR has effectively ceased, leads to a decrease in the damaged state of the cores. Higher levels of stress result in greater reductions in the damaged state.

Keywords: alkali silica reaction; concrete; stiffness damage test; stressed/unstressed cores; elastic modulus; core compressive strength; ultrasonic pulse velocity; water absorption; expansion; stress; strain; strength.

INTRODUCTION

Because cores can expand when they are extracted from a large block of ASR concrete, the properties of the cores quoted should not be taken to be directly representative of the concrete in the members from which the cores were taken. McLeish(9) pointed out that in most cases part of the core will be from the cover, which may be more badly deteriorated than the region of the member restrained by the reinforcement cage. This point is also illustrated by Imai et al.(6) who carried out an investigation on concrete piers supporting the Hanshin Expressway in Japan. The compressive strength and the Young's Modulus of cores obtained from deteriorated piers were about 65% and 20%, respectively, of those taken from sound piers. Results from these were compared with field loading tests on the piers from which cores had been extracted showed that the tested piers had adequate load carrying capacity. This conclusion was also demonstrated by Fujii et al.(4). This would tend to support the view by Ono(10) that the difference between Young's Modulus of the concrete structure as a whole and that of cores drilled from the structure might be due to the different restraints that apply to each.

Hobbs(5) made the point that the cores may expand during and after coring and due to the possible damage arising from the coring process. He supported his viewpoint by an experimental test on two specimens which were restrained with an initial compressive stress of 2.5 N/mm^2 at 5 days after casting. He found that as the specimens expanded, the stresses increased to 3.5 N/mm^2 in one specimen and 4 N/mm^2 in the other. Once these stresses were attained, no further expansion occurred. On the removal of restraint the expansion continued.

However, Blight et al.(2) carried out a loading test and finite element analysis of a road bridge. They found that the modulus of elasticity measured in core tests, when used in the finite element analysis, gave deformational characteristics that correlated well with the behaviour of the bridge. An experimental study by Tomita et al.(13) on cores taken from restrained and unrestrained structural model specimens (300 x 300 x 500 mm) suggested that the future damage of existing concrete structures affected by ASR could be estimated from drilled cores. The recent study by Alexander et al.(1) indicate that the behaviour of ASR affected structures can be predicted on the basis of properties measured on sample cores.

The aim of the research reported here was to examine the effects of different stress levels on the mechanical behaviour of deteriorated concrete cores extracted from large blocks affected by ASR. The following parameters were measured in the cores over the two years of the research programme:

1. The stiffness damage properties of Elastic Modulus (E_c), Damage Index (DI) and Plastic Strain (PS).
2. The Water Absorption (WA).
3. The Ultrasonic Pulse Velocity (UPV).
4. The core compressive strength (f_c).

EXPERIMENTAL WORK

Concrete mix and materials

The basic concrete mix (ASR-I mix) was a 1: 1.57: 1.41 (cement: sand: coarse aggregate) by weight, with a cement content of 550 Kg/m^3 and a water/cement (W/C) ratio of 0.36. The cement was an ordinary Portland cement having an average alkali content of 0.60%. In order to accelerate the ASR reaction, the alkali content was raised to 12 Kg/m^3 by adding KOH to the mix. A second mix (ASR-II mix) was formed by replacing 11.9% by weight of the total aggregate in ASR-I mix, with fused silica. The detailed mix designs for the two mixes used are given in Table 1. Full details of the aggregate is given in Reference 11.

	ASR-I mix	ASR-II mix
Ordinary Portland Cement (OPC)	550 kg/m^3	550 kg/m^3
Coarse Aggregate (4.75 - 20 mm)	780 kg/m^3	780 kg/m^3
Fine Aggregate (< 5 mm)	865 kg/m^3	670 kg/m^3
Fused Silica (1.0 - 3.0 mm)	-	195 kg/m^3
Water/Cement Ratio	0.36	0.36
Alkali Equivalent	12 kg/m^3	12 kg/m^3

Table 1 - Concrete mix designs.

Sample size and curing conditions

The specimens used in this investigation were cored from concrete blocks that were 500 mm long by 500 mm high by 200 mm wide. Two concrete blocks were cast, one using the ASR-I mix and the other the ASR-II mix along with 100 mm cubes. After casting, the specimens were covered with hessian and polythene sheeting and left undisturbed in the casting room for 48 hours, after which the moulds were stripped. Subsequent to this the blocks were kept in water at 20 ± 1 °C for 2 days following which they were then removed to a water tank which was kept at a temperature of 38 ± 1 °C.

Direction of drilling

After one year, 16 cores of 74 mm diameter were taken from each block, the drilling was carried out perpendicular to the direction of casting. The trimmed cores were capped on both ends using the sulphur-carbon method as detailed in BS: 1881 part 120, 1983 section 5.3(3). All cores were produced with a L/D ratio of 2.5. Demec studs were then fixed along the length of core in three equally spaced rows at 120 degree intervals around the cores.

Application of preload

Four cores from each concrete mix were crushed to determine their strength. The remaining cores were then subjected to a variety of tests including the Stiffness Damage Tests (SDT), Ultrasonic Pulse Velocity (UPV) and weight determination (WA). Following this, the cores were subjected to a permanent compressive stress of either zero N/mm^2, 3.5 N/mm^2 or 7 N/mm^2. The cores were then placed in a curing tank at 38 ± 1 °C. The stress was applied through (170 x 170 x 8 mm) mild steel plates placed at both ends of the concrete core, as shown in Fig. 1. Each plate had one hole symmetrically drilled along each side to allow two 20 mm diameter mild steel rods, threaded at both ends, to be passed through and stressed against the plates. The rods were tensioned using a torque wrench and periodically retorque during the following 12 months to maintain the stress level. After one further year in the curing tank the cores were removed from the tank and the stress released. They were then tested again to determine the new stiffness damage properties, the Ultrasonic Pulse Velocity, weight change and core compressive strength.

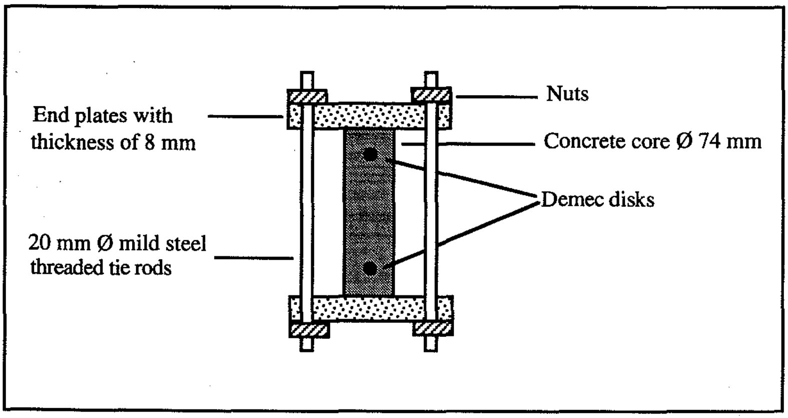

Fig. 1 - Stressed core.

TEST RESULTS AND ANALYSIS

The results presented were obtained in tests carried out on sixteen cores from each concrete mix. Four cores were subjected to the stress of 3.5 N/mm^2 and four to the stress of 7 N/mm^2 and another four cores were left unstressed as control specimens. The remaining four cores were crushed to determine the strength of the concrete at this time.

The results indicate the influence of permanent stress levels upon the mechanical properties of both types of reactive concrete used in the test programme and highlight the beneficial role that stresses due to imposed loads can play in ASR affected concrete structures.

The expansion data for the unstressed cores, shown in Table 5, indicates that the ASR had effectively ceased after the first year, so the changes in the mechanical properties resulting from the stressing are not significantly affected by continuing ASR within the concrete.

Elastic Modulus

The results given in Tables 2, 3 and 4 show that stresses applied after the ASR has virtually ceased cause significant changes to all the mechanical properties of ASR concrete. In the case of E_c, f_c and UPV these changes are very approximately linearly related to stress, regardless of the mix type. This could be explained by considering that the applied stress has closed up cracks that cross the direction of stress and that these cracks do not immediately re-open on removal of the stress.

In Table 4 the percentage increase in E_c in the two year old concrete cores that were stressed for twelve months is given relative to E_c for the two year old unstressed cores. It is seen that increasing stresses leads to increasing values of E_c. That is: $E_c\ (7\ \text{N/mm}^2) > E_c\ (3.5\ \text{N/mm}^2) > E_c\ (0\ \text{N/mm}^2)$

It is further seen from Tables 2 and 3 that the unstressed reactive specimens also show an increase in E_c for ASR-I and ASR-II concrete mixes during the second year of the test programme of 18% and 45% respectively. This increase in E_c in the unstressed cores is somewhat greater than would be expected for normal concrete and indicates that in certain circumstances ASR concrete exhibits appreciable autogeneous healing once the expansion is essentially complete. It is suggested that calcification of the gel results in a autogeneous healing process which causes E_c to increase(12). It is also seen that E_c for the stressed ASR-I mix is greater than for the stressed ASR-II mix.

	Age: One year	
	ASR-I mix Cores	ASR-II mix Cores
E_c (kN/mm^2)	25.50 ± 2.15	19.83 ± 1.68
DI (Dimensionless)	7.57 ± 2.73	11.56 ± 2.88
PS ($\times 10^{-6}$)	3.50 ± 1.30	4.73 ± 2.86
UPV (km/sec)	4.31 ± 0.03	4.03 ± 0.07
f_c (N/mm^2)	33.48 ± 3.04	26.27 ± 1.98

Table 2 - Mechanical properties of ASR concrete cores.

	Age: Two Years					
	ASR-I mix Cores			ASR-II mix Cores		
	Stressed to (7N/mm^2)	Stressed to (3.5 N/mm^2)	Unstressed	Stressed to (7N/mm^2)	Stressed to (3.5 N/mm^2)	Unstressed
E_c (kN/mm^2)	35.02 ± 0.85	32.42 ± 1.90	30.18 ± 2.64	32.67 ± 2.10	30.10 ± 0.81	28.80 ± 3.11
DI -	1.24 ± 0.58	1.48 ± 0.80	2.37 ± 1.64	2.03 ± 0.96	2.38 ± 1.62	3.51 ± 1.61
PS ($\times 10^{-6}$)	0.11 ± 0.90	0.56 ± 1.65	1.94 ± 1.56	1.02 ± 1.08	1.20 ± 1.52	4.50 ± 2.60
UPV (km/sec)	4.47 ± 0.03	4.50 ± 0.01	4.52 ± 0.03	4.31 ± 0.02	4.34 ± 0.03	4.38 ± 0.03
f_c (N/mm^2)	50.83 ± 2.18	48.84 ± 3.25	47.03 ± 2.00	44.40 ± 2.43	42.77 ± 2.02	41.28 ± 1.82

Table 3 - Effect of stress intensity on the mechanical properties of ASR concrete cores.

	ASR-I mix Cores		ASR-II mix Cores	
	Stressed to (7 N/mm^2)	Stressed to (3.5 N/mm^2)	Stressed to (7 N/mm^2)	Stressed to (3.5 N/mm^2)
% Ec	+ 16.03	+ 7.42	+ 13.43	+ 4.51
% DI	- 47.68	- 40.92	- 42.16	- 32.20
% PS	- 94.33	- 71.13	- 77.33	- 73.33
% UPV	- 1.10	- 0.44	- 1.60	- 0.91
% f_c	+ 8.08	+ 3.84	+ 7.55	+ 3.61

N.B. Minus (-) indicates % loss Plus (+) indicates % gain

Table 4 - Percentage difference in the mechanical properties after 2-years compared to their control cores.

Damage Index

It is known that the greater the DI the more damage present within the core(11, 12). It is apparent that damage is likely to be less for a concrete with a high modulus than for a concrete with a low modulus. This assumption is confirmed by the results given in Tables 2, 3 and 4 where increasing modulus is in every case accompanied by decreases in the DI.

Plastic Strain

It is seen that: PS (7 N/mm^2) < PS (3.5 N/mm^2) < PS (0 N/mm^2)

The maximum reduction in PS between the unstressed and the 7 N/mm^2 stressed case is 94% for ASR-I mix. This indicates that the stresses are causing a substantial reduction in the damage within the concrete. The PS is seen in Tables 2 and 3 to be greater with the ASR-II mix than with the ASR-I mix.

It is also noted that with both concrete mixes the unstressed cores also show a reduction in the PS during the second year of the programme; the maximum reduction being 44% in the case of ASR-I mix. This is again considered to be due to autogeneous healing of the microcracks(12).

Restrained Expansions

After one year, the measured expansions on blocks of ASR-I and ASR-II mixes were 1.32% and 2.25% respectively.

Table 5 shows that the immediate contraction for both ASR concrete mixes measured after applying the permanent stress, for the 7 N/mm^2 case was approximately twice that for the 3.5 N/mm^2 case indicating that the cores were behaving almost linearly elastic.

Subsequent further expansion was negligible or zero for the stressed cores during the second year. The unstressed cores continued expanding during this second year but only by 7% and 5% of the first year expansion for ASR-I and ASR-II respectively, indicating that the reaction was effectively exhausted.

Comparison of the immediate contraction figures with values calculated from the measured results for E_c given in Table 2 show that the former is approximately 2 to 3 times greater than the latter for both stress levels applied. This significant difference may be partly attributed to the limited accuracy of measurement of the former using a Demec Gauge. Another factor might be that a higher value of E_c is obtained from the Stiffness Damage Tests due to the results being obtained with a starting stress level of approximately 0.50 N/mm^2 which closes up some or all of the cracks and hence makes the core stiffer(12).

The results for immediate expansion on removal of the stress at the end of the second year are however in much better agreement with the values calculated from E_c obtained from the SDT carried out at that time. This tends to indicate that cracks did not substantially re-open upon removal of the stress.

		Immediately after preloading	After 12 months of preloading	Immediately after release of preload	Not preload
		ASR-I mix Cores			
Stressed to 7.0 N/mm²	M1	- 0.051	0	+ 0.017	-
	M2	- 0.064	0	+ 0.025	-
	M3	- 0.072	0	+ 0.020	-
	M4	- 0.058	0	+ 0.022	-
	Average	- 0.061	0	+ 0.021	-
Stressed to 3.5 N/mm²	M5	- 0.040	0	+ 0.013	-
	M6	- 0.032	+ 0.001	+ 0.017	-
	M7	- 0.038	+ 0.001	+ 0.018	-
	M8	- 0.030	0	+ 0.015	-
	Average	- 0.035	0	+ 0.016	-
Unstressed	M9	-	-	-	+ 0.110
	M10	-	-	-	+ 0.094
	M11	-	-	-	+ 0.082
	M12	-	-	-	+ 0.098
	Average	-	-	-	+ 0.096
		ASR-II mix Cores			
Stressed to 7.0 N/mm²	N1	- 0.122	0	+ 0.027	-
	N2	- 0.096	0	+ 0.035	-
	N3	- 0.116	0	+ 0.040	-
	N4	- 0.115	0	+ 0.030	-
	Average	- 0.112	0	+ 0.033	-
Stressed to 3.5 N/mm²	N5	- 0.051	+ 0.002	+ 0.028	-
	N6	- 0.080	+ 0.006	+ 0.022	-
	N7	- 0.056	+ 0.005	+ 0.020	-
	N8	- 0.070	+ 0.008	+ 0.024	-
	Average	- 0.064	+ 0.005	+ 0.023	-
Unstressed	N9	-	-	-	+ 0.110
	N10	-	-	-	+ 0.101
	N11	-	-	-	+ 0.143
	N12	-	-	-	+ 0.112
	Average	-	-	-	+ 0.116

N.B. Minus (-) indicates % contraction Plus (+) indicates % expansion

Table 5 - Expansions of stressed and unstressed cores.

Water Absorption

Table 6 shows the variation of Water Absorption (WA) with time for both concrete mixes. It is clear that the gel resulting from ASR-II mix absorbed twice the amount of water as the gel resulting from ASR-I mix. Though the variation in WA between the stressed and unstressed specimens is small, the results do suggest that increasing levels of stress might be associated with reduced water intake.

		ASR-I mix Cores			ASR-II mix Cores		
		Stressed to 7 N/mm²	Stressed to 3.5 N/mm²	Unstressed	Stressed to 7 N/mm²	Stressed to 3.5 N/mm²	Unstressed
WA (gram)	1st Year	1895.80	1884.80	1896.30	1795.10	1800.00	1796.10
	2nd year	1903.60	1893.82	1905.70	1810.44	1815.60	1813.30
Weight (%)		+ 0.41	+ 0.47	+ 0.50	+ 0.85	+ 0.87	+ 0.95

N.B. Plus (+) indicates % gain

Table 6 - Weight change determination.

This observation is in agreement with an experimental study carried out by McGowan and Vivian(8) on ASR mortar bars who found that increasing the magnitude of the applied force diminishes both the amount of water absorbed and the proportion of the volume change that causes widening of the crack plane which developed at right angles to the direction of the applied force.

Ultrasonic Pulse Velocity

In Tables 2 and 3, it is seen that the stressed specimens for both ASR-I and ASR-II mixes show an increase in UPV over the first year of approximately 5% and 8% respectively.

Comparing the stressed and unstressed specimens in Table 4, it is seen that the application of permanent stress during the second year results in only a small reduction in UPV.

Core Compressive Strength

One year after coring, four cores from each mix were crushed. The effect of permanent stress levels on the core compressive strength (f_c) of ASR concrete compared to that of unstressed cores is shown in Table 3. It is seen that:

$$f_c\,(7\ \mathrm{N/mm^2}) > f_c\,(3.5\ \mathrm{N/mm^2}) > f_c\,(0\ \mathrm{N/mm^2})$$

with f_c (7 N/mm^2) being nearly 8% greater than f_c (0 N/mm^2) for both ASR mixes. It is also seen that there are very significant increases in the values of f_c over the second year for the unstressed cores with increases in strength of 40% for ASR-I mix and 57% for ASR-II mix (see Tables 2 and 3). There are probably two main reasons for this gain in strength; calcification of the gel is considered to be the major factor with continuing hydration of the cement being expected to contribute an increase in strength of approximately 20% for both mixes.

An investigation by Koyanagi et al.(7) on ASR cylindrical specimens found that their crushing strength exhibited a significant increase from an age of 160 days. This increase was in their view due to ASR ceasing and further curing coming into effect.

Discussion and Conclusions

The results indicate that the application of permanent stress to ASR cores when the ASR has effectively ceased, leads to a decrease in the damaged state of the cores. Higher levels of stress result in greater reductions in the damage state. It is suggested that:

Damage (7 N/mm^2) < Damage (3.5 N/mm^2) < Damage (0 N/mm^2)

This can be explained as being due to the closure of the cracks within the core and autogeneous healing of the crack then taking place which has a beneficial effect in reducing the apparent damage within the core.

When a core is drilled from a structure, all the restraining stresses, due both to loading and the reinforcement, are released. This allows the core to expand and microcracks to develop during the period between drilling and testing the core. This research has demonstrated that applying a stress to the core for a period of time after coring but prior to testing has the effect of closing the microcracks that have developed both before and after drilling and hence result in less damage being identified by the Stiffness Damage Tests and other properties. This condition may more closely reflect the original damaged state of the concrete in the structure and so test results on such stressed cores may give a better indication of the true state of the concrete. A quantitative relationship between the damaged state of the concrete in-situ, prior to coring, and the condition of the stressed core at the time of testing, has obviously though not been established by this work.

While the test programme reported here was in progress McLeish(9) suggested that "if a core was subjected to a constant stress of 4 N/mm^2 for a period of time and was then increased to say l0 N/mm^2, then a higher modulus would be expected". He further stated that "this condition may be very similar to the stress state in a real structure".

The unstressed cores, exhibited the greatest damage but this damage decreased during the second year of the test programme but was still higher than was found in the stressed cores. The decrease in damage could be explained in part as being due to the calcification of the ASR gel.

Expansion along the vertical axes of the cores ceased for both concrete mixes on application of the stress. The unstressed cores did however continue to show a small expansion, but this was so slight that the reaction seemed to have been effectively exhausted.

Concrete cores from ASR-I mix exhibited less damage than those of ASR-II mix. Applying the stress tends to increase the stiffness of both mixes. However, the recovery of the stiffness for ASR-I mix was greater than that for ASR-II mix, bearing in mind that ASR-I mix was initially less damaged.

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FRACTAL ANALYSIS OF CRACKED SURFACE IN AAR CONCRETE

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ABSTRACT

The purpose of this study is to develop a method which can quantify aspect of damaged surface in concrete subjected to alkali aggregate reaction (AAR) in terms of fractal geometry. The paper presents a new evaluation method with fractal dimension analysis for AAR cracking. In the fractal analysis, cracking pattern of the damaged surface was digitized by using an image analysis which used box counting method. The results indicated the conditions of fractal analysis, the fractal character and the classification of AAR crack. Also, we investigated relationships between fractal dimension, AAR expansion and width of crack.

Keywords: Fractal Analysis, Box Counting Method, Fractal Dimension, Lower Limit of Division, Number of Division, Expansion, Width of Crack, AAR

INTRODUCTION

Since Mandelbrot introduced a new geometry concerning Fractal1), many disciplines have utilized the concepts of fractal. However, studies on the application of these concepts to cracked concrete have been scarce at best. A traverse method has been employed to evaluate cracked surface of AAR concrete for amount and width of the crack. So far, however, the concepts and method of quantifying the cracking aspect of the concrete have not been developed. This study is to introduce a new approach - fractal geometry concept - to the AAR crack. This concept provides useful tools for evaluating characterization of cracked surface in AAR concrete.

The concept of fractal has been successfully applied to a large number of phenomena, among them such as properties of mineral particles, crack of rock, cracking in damaged concrete, crack of pavement-concrete and landscape. Fractal is a shape made of parts similar to the whole in some way, and means irregular shaped figures characterized by a number D, called fractal dimension. D in Euclidean geometry is an integer and equals 1,2,3 for ordinary lines, surface and volumes, respectively. However, in the fractal situations, D can be a non-integer. The notion of a fractal between appropriate lower and upper dimensional bounds may be applied to comminution if the size distribution is fractal. In order to understand the relevance of fractal geometry for AAR crack it is necessary to discuss conditions of the fractal analysis, and investigate relationship between fractal dimension and AAR factors. In the study, we applied the concept of fractal to evaluating the cracks in damaged concrete due to AAR. Also, the simple traverse method was adopted to measure width of the cracks.

1. Fractal Dimension Analysis

The fractal dimension was analyzed to quantify the aspect of surface damaged concretes due to AAR. The objects of the analysis were cracking patterns of every description from the concretes, and box counting method was employed in the analysis. The fractal dimension depending on the box counting method is defined by cover theory as follows. The crack image is covered with square grids with size $r_0 \times r_0$, and changed the size of grids on given image. A simple paradigm for this box counting method can be constructed by referring to Fig.1. N indicates the number of square grids needed to cover the cracks (N changes with r_0). In the limit of r_0:

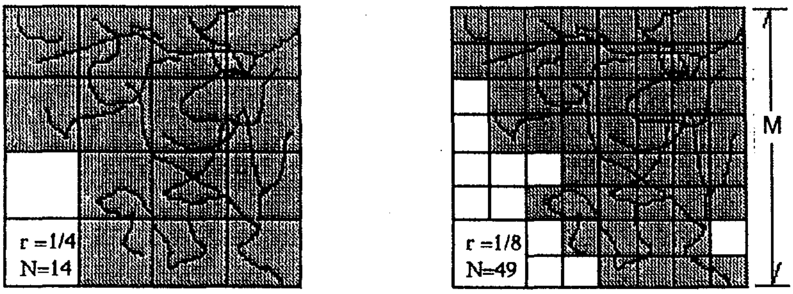


Fig. 1 Square cover method - grids of various size

$$N \propto r_0^{-D} \qquad (1)$$

where D is fractal dimension depending on the particular characteristics of the phenomena. N is plotted as a function of division size r_0, observation is fitted by linear regression. The fractal dimension D may be determined by finding the slope of Log N(r) as a function of Log r, and is better to use the following equation[2)].

$$D(r) = -\frac{d \operatorname{Log} N(r)}{d \operatorname{Log} r} \qquad (2)$$

where $r=r_0/M$, M is a maximum length of crack.

The flow chart of the fractal analysis is given in Fig.2. In the fractal analysis the image is divided with changing size of grid from upper limit to lower limit. The number of square grid covering cracks in the processed image is counted by thinning the original binary image of crack. The grid containing at least one thinned crack is one picture element - divided square grid. Crack width is not considered in the fractal analysis as the box counting method.

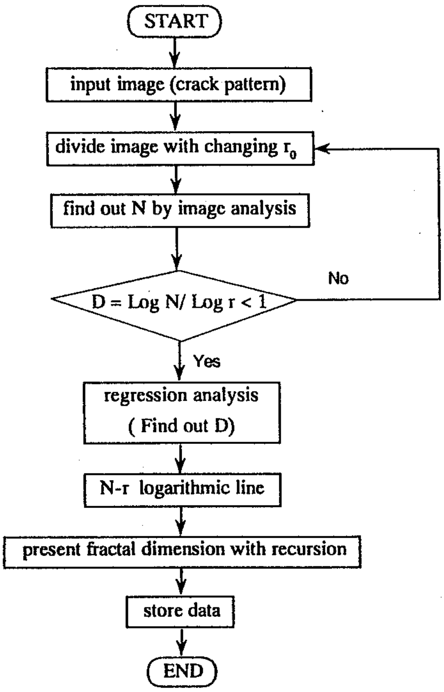


Fig. 2 System Flow of Fractal Dimension Analysis

In addition, ramiform aspect of cracking is quantified by shape factor (ϕ), and it is indicated as following expression:

$$M = \phi\sqrt{A} \quad (\phi \geqq 1) \quad (r_0 = M、 N=1) \qquad (3)$$

where A is area of square covered crack.

Total length of crack to unit area is quantified by crack density (C.D) , it will be given by

$$\begin{aligned} C.D = L / A_0 &= \phi^{D} A^{D/2} \varepsilon^{1-D} / A_0 \\ &= M^{D} \varepsilon^{1-D} / A_0 \qquad (4) \end{aligned}$$

where L is total extension of crack $(L=N\varepsilon)$, ε is lower limit of cracked length; A_0 is square area of fractal object.

Area of crack in term of crack ratio (C.R) is:

$$C.R = A/A_0 = (M/\phi)^2 / A_0 \qquad (5)$$

2. Result and discussion

Fig. 3 shows a plot of N(r) as a function of r ($r = r_0 / M$) depending on slope with logarithmic line of N and r_0 to determine the fractal dimension D. The values of fractal dimension range from 1 for lines to 2 for surface between appropriate lower and upper dimensional bounds. In the example the same division number was adopted.

2.1 Condition of fractal dimension analysis

In the fractal dimension analysis within box counting method, it was defined that the maximum division length (upper limit of fractal) corresponds with maximum extension of crack[3]. The lower limit of fractal is defined as follows. A simple example for this lower limit is constructed by referring to Fig.4. Because D in Euclidean geometry is an integer and equals 1 for ordinary lines, the size of division at which its fractal dimension is D=1 should be defined to lower limit of fractal. From this plot, that there is no longer fractal if its fractal dimension is D<1.

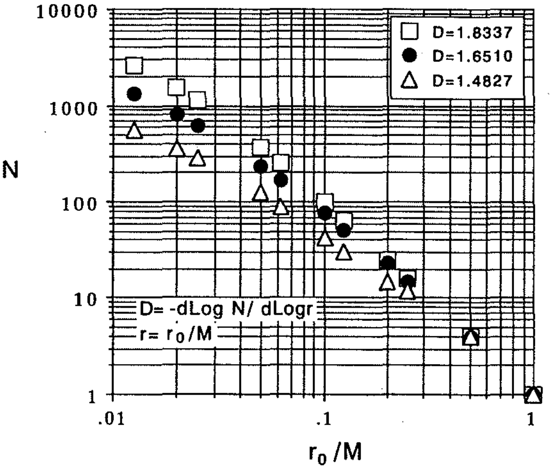


Fig. 3 An example analyzed fractal dimension (D)

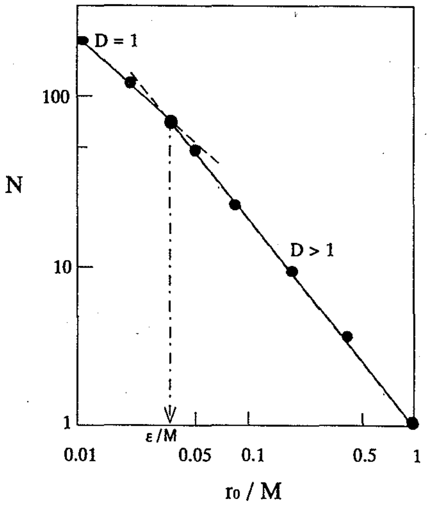


Fig. 4 Lower limit of fractal

The minimum division size (lower limit of fractal) for AAR crack and its effect on value of fractal dimension are investigate as follows.

The effects of lower limit of division (minimum division r_0) on result of analysis with slope for one upper limit are shown in Fig.5 and 6. This lower limit indicates a adopted minimum length to change division size by box counting method. The numeral in the explanatory notes indicated the division number. The fractal dimension increased with the increase in the minimum division size. These quantities of fractal dimensions were approximately equal when the same lower limit was employed. It is suggested that the division number did not influence the result of fractal dimension analysis in the same lower and upper bounds.

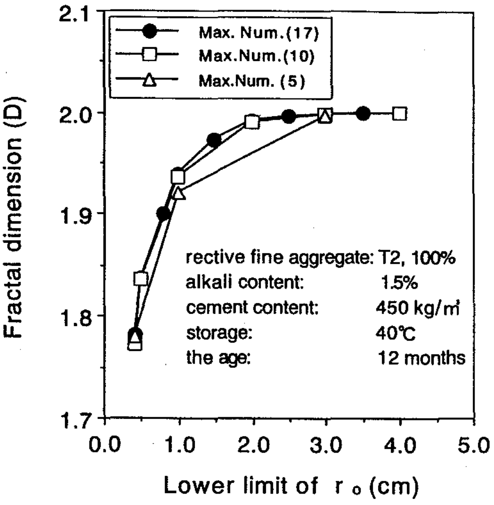


Fig. 5 Relationships between D and lower limit of r_o

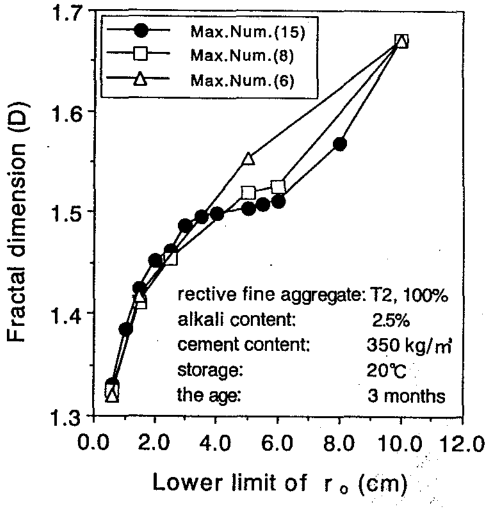


Fig. 6 Relationships between D and lower limit of r_o

The relationship between minimum division size and fractal character is shown in Fig.7. In this investigation the reactive fine aggregate with a maximum size of 5mm and the reactive coarse aggregate with a maximum size of 10mm were used in the AAR concretes, and the crack patterns of these concretes were analyzed with box counting method. From the results, there was a substantial division size range over which the fractal dimension is D>1, and when the division size (r_0) was smaller than maximum size of reactive aggregate, fractal dimension was D<1 in which there is no fractal character. Therefore, it can be confirmed that the lower limit of division having fractal character should correspond the maximum size used reactive aggregate at least.

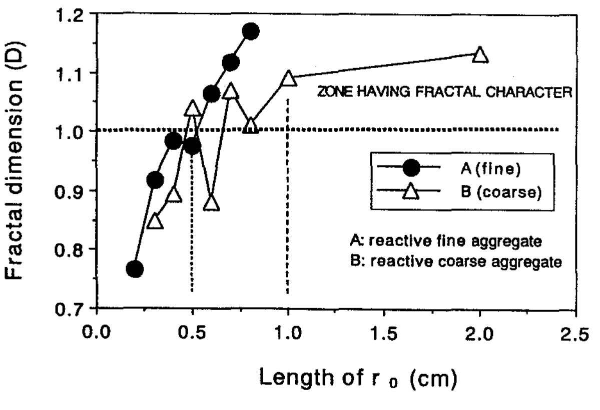


Fig. 7 Lower limit of r_0

2.2 Fractal dimension and expansion

The results of fractal dimension analysis for AAR crack are presented in Table 1. The results in the table showed relationship between fractal character values and expansion. From this table, it is appeared that aspect of cracked surface in AAR concrete was classified to the three categories; linear, planar and alligator and were expressed by fractal dimension ranging from 1.0 to 1.2, 1.2 to 1.5 and over 1.5 respectively. When the aspect was alligator, crack density was larger than 0.1, and the expansion of concrete reached to over 1.0%.

Table 1. Analyzed Fractal Characteristic Values and Expansion

Fractal dimension (D)	Shape factor (ϕ)	Crack density (C.D)	Crack ratio (C.R) (%)	Expansion (%)
1.80~1.89	1.50~1.35	0.74~0.87	44~56	0.40~0.80
1.70~1.79	1.81~1.49	0.35~0.79	31~49	0.20~0.56
1.60~1.69	2.13~1.67	0.27~0.55	22~37	0.17~0.36
1.50~1.59	2.40~1.86	0.17~0.37	12~29	0.10~0.23
1.40~1.49	3.26~2.76	0.15~0.23	9 ~18	0.05~0.14
1.30~1.39	3.80~2.45	0.11~0.16	6 ~16	0.05~0.12
1.20~1.29	4.62~3.30	0.05~0.11	5 ~11	0.05~0.07
1.10~1.19	4.91~3.74	0.04~0.07	4 ~7	0.05~0.06

Fig.8 is a plot of the relationship between fractal dimension and expansion. The fractal dimension increased with the increase in the expansion, and this increase rate was inclined to gradually less with the increase of expansion. The fractal dimension reached over 1.5 when the expansion was larger than 0.1% which is harmful for concrete structure[4]. The relationship between fractal dimension (D) and expansion (E) was revolved as following equation.

$$\mathrm{Log}\, E = 1.72\, D - 3.48 \qquad (6)$$

The propagation of AAR crack is essentially due to expansion. Fig. 9 shows the relationship between the expansion and the total width of cracks between the measuring plugs (mm). The plot compressed that the total width of cracks was smaller than the expansion, and not necessarily the expansion. This result agreed with that from S.Diamond[5].

Fig.10 shows the fractal dimensions versus time development when two kinds of alkali were used. The fractal dimension increased monotonically with the increase in the ages, and was different with kinds of alkali. That is to say, the aspect of cracked surface and the amount of crack were different with the kinds even though the same alkali content was used. The fractal dimension increased slightly with the age after 6 months, and it can be considered that the change of crack in image was mainly from the increase in the its width.

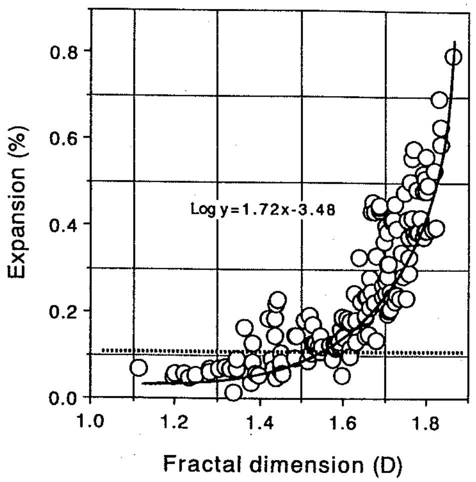


Fig. 8 Relationships between fractal dimension and expansion

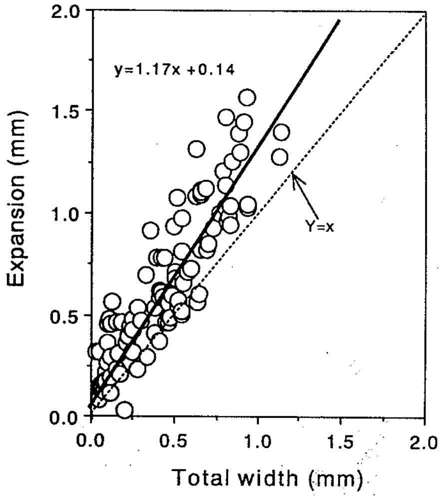


Fig. 9 Expansion and total width of cracks between the plugs

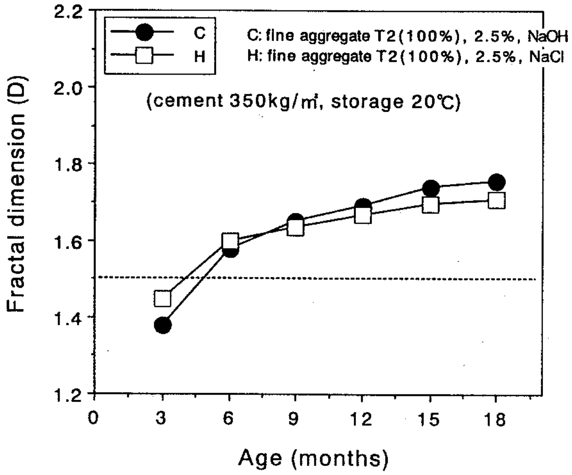
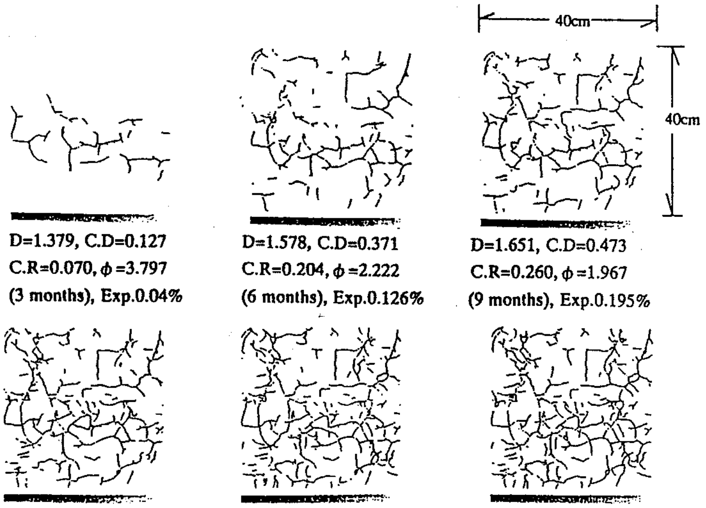


Fig. 10 Fractal dimension VS. time in 20℃, 100% RH. storage

Fig. 11 presents the crack patterns which were indicated by fractal dimension at the several ages. These crack patterns expressed a process in which the crack changed from linear to alligator, and were quantified by fractal dimension. It is observed that the effectiveness of this estimate with fractal dimension analysis was confirmed.

3. Conclusion

The box counting method with fractal dimension analysis for AAR cracking characterization was employed in this study. Through this method, the aspect of cracked surface of AAR concrete was analyzed and evaluated. The following was clarified by this study.



D=1.379, C.D=0.127
C.R=0.070, ϕ=3.797
(3 months), Exp.0.04%

D=1.578, C.D=0.371
C.R=0.204, ϕ=2.222
(6 months), Exp.0.126%

D=1.651, C.D=0.473
C.R=0.260, ϕ=1.967
(9 months), Exp.0.195%

D=1.692, C.D=0.545
C.R=0.300, ϕ=1.833
(12 months),Exp.0.262%

D=1.742, C.D=0.641
C.R=0.352, ϕ=1.689
(15 months),Exp.0.285%

D=1.756, C.D=0.674
C.R=0.371, ϕ=1.647
(18 months),Exp.0.290%

Fig. 11 Crack patterns at the several ages

(1) Cracking aspect of AAR concrete has fractal character and can be quantifically characterized by fractal values.

(2) The fractal dimension increased with the increase in the adopted minimum division size. The division number did not influence the results of fractal dimension analysis.

(3) The lower limit of division having fractal character should correspond with the maximum size of reactive aggregate used.

(4) AAR cracking aspect was classified to linear, planar and alligator by fractal dimension ranging from 1.0 to 1.2, 1.2 to 1.5 and over 1.5, respectively. It was observed that the expansion of concrete reached to over 1.0%, and the fractal dimension was D>1.5 at which the aspect of crack was alligator.

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MODELLING OF THE STRUCTURAL BEHAVIOUR OF AAR AFFECTED REINFORCED CONCRETE MEMBERS

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ABSTRACT

A non-linear two dimensional finite element analysis has been developed in order to assist in the understanding of the structural effects of alkali aggregate reaction (AAR) on concrete members. It has been observed that the application of relatively small compressive stresses to concrete during the development of the AAR can considerably inhibit expansion. This phenomenon has been shown to be important and has been included in the analysis as have the changes in material properties due to AAR. A brief description of the analytical model is given and experimental results from two tests are compared with the predictions of the analysis.

Key Words: Alkali aggregate reaction; finite element analysis.

INTRODUCTION

It has been observed that relatively small compressive stresses can have a considerable effect on the expansion of AAR specimens. Stresses of about 4 N/mm² can stop the expansion occurring almost completely. Figure 1 shows a typical relationship between expansion and stress (*Cope et al 1994).* To carry out any sensible analyses of AAR affected structures, this phenomenon needs to be included, in addition to the changes in material properties due to the development of AAR. A brief description of such an analysis follows, together with two examples of the use of the analysis.

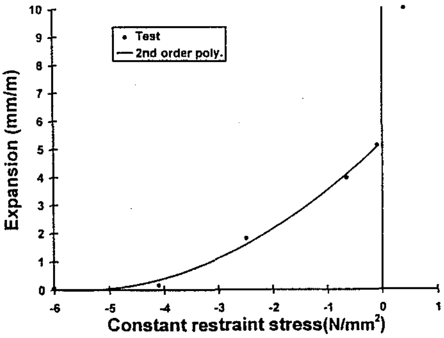

Fig. 1 Constant stress-expansion relationships

ANALYTICAL MODEL

A two dimensional, plane stress, finite element analysis, which includes material non-linearities, has been developed. This is capable of analysing members subjected to AAR and a set of external loadings, which can be varied or held constant throughout the development of the AAR. After the AAR is complete, the member can then be analysed up to failure, with a pattern of loading which can be different to that applied during the AAR.

The analysis can be considered to comprise three stages:

1. Analysis of the member under external loading.
2. AAR expansion analysis under the external loading.
3. Analysis of the affected member up to failure.

The first stage comprises a smeared crack non-linear finite element analysis for unaffected concrete. The third stage is similar but uses the deteriorating properties of the affected concrete as determined during stage 2. Stage 2 is described in more detail in the next Section.

In all the analyses, the constitutive model for the concrete in compression is based on plasticity theory. During the analysis of the effects of AAR on the concrete properties, for example, reductions in compressive strength and stiffness are included. A full description of the analysis is given elsewhere *(Cope et al 1994)*.

AAR EXPANSION ANALYSIS

This stage of the analysis simulates the period when AAR expansion is taking place. The total free expansion is applied in a number of increments. For each increment, the restrained expansions in the principal stress directions throughout the concrete, are determined at each sampling point from a family of curves of the same form as the stress-expansion relationship shown in Fig. 1. An initial strain analysis is then carried out using the restrained expansions as the initial strains. Generally, the member will not be in equilibrium. Therefore, the analysis is repeated using constitutive equations dependent on expansion level until convergence occurs. When convergence has occurred, the properties of concrete are modified, according to the values of expansion in principal strain directions throughout the concrete. The above procedure is then repeated until the total free expansion has been applied.

Since the material property changes are applied at the end of each incremental step, the incremental free expansion is chosen to be small, typically of the order of 0.02mm/m, in order to reduce the accumulated error and the chance of numerical disturbances caused by sudden large changes in material properties.

EXAMPLES

The finite element program has been used to analyse a large number of reinforced concrete members, full details of which are given elsewhere *(Cope et al 1994)*. Description of two of the examples analysed are given below.

Expansion analysis

In order to predict the strength and stiffness of a reinforced concrete member realistically, it is essential to initially predict the expansion and induced stresses within it to a reasonable degree of accuracy. It has been shown that the proposed expansion model is capable of predicting, relatively accurately, expansions of symmetrically reinforced specimens *(Cope et al 1994), (May et al 1992).* The inclusion of the expansion model into a finite element program, together with the models for cracking and change of material properties due to expansion, allows expansion analyses to be carried out, realistically, and on a wide range of reinforced concrete members. In this section, examples are given to verify the approach.

Example 1: Singly reinforced beam conditioned without load

Fig. 2 shows details of the smaller, singly reinforced beams tested by Cope *(Cope 1992).* These beams were prepared with a special mix designed by the Building Research Establishment and conditioned under water at 38°C to accelerate the AAR expansion. Expansions of the beams were monitored during the period of AAR at various positions and directions. The measurements taken included horizontal expansions along the top, horizontal expansions along the reinforcement and vertical expansions. The average values of the measured expansions were compared with the average free expansions of 100 x 200 mm cylinders, made of the same concrete and stored in the same conditions. These studies provided detailed information on the differential expansions on the surfaces of reinforced concrete members and the relationship between free and restrained expansions. The results of the tests have been used to verify the model for AAR expansion.

Fig. 3A shows the finite element representation of the beam. The data used for the analysis are given in Table 1. Concrete properties measured are without the effects of AAR. Figs. 3 B, C, and D show, respectively, the deformation of the beam, the cracks due to tensile stress in concrete caused by the deformation of the beam and the stresses induced in the reinforcement at a free expansion of 5 mm/m. The deformation shown in Fig. 3B has been amplified by a factor of 5.

Table 1 Parameters Used In Analyses

	Example 1	Example 2
Concrete Compressive strength (N/mm²)	50	45
Tensile strength (N/mm²)	2.5	2.5
Elastic modulus (kN/mm²)	40	40
Poisson's ratio	0.2	0.2
Ultimate crushing strain	0.0035	0.0035
Steel Yield stress (N/mm²)	400	560; stirrups 400
Elastic modulus (kN/mm²)	210	200

The average expansions measured on the beams are plotted against the average free expansions of the cylinders in Fig. 4. The predictions using the finite element analysis and those from elastic analysis assuming the expansions to be stress independent are also shown.

Fig. 5 shows the predicted stress distributions induced by AAR expansion through the mid-span section of the beam at different levels of free expansion ranging from 0.4 to 5 mm/m.

Fig.6 shows the predicted uniaxial compressive strength and initial Young's modulus

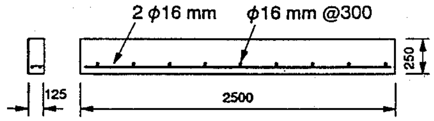

Fig. 2 Details of singly reinforced beam tested by Cope et al.

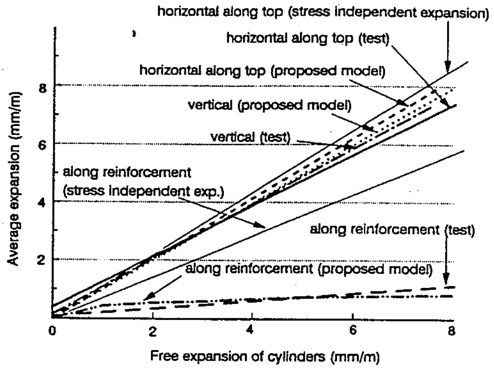

Fig.4 Comparison of expansion measurements with the predicted values using the proposed model and with those using elastic analysis assuming the expansion stress independent.

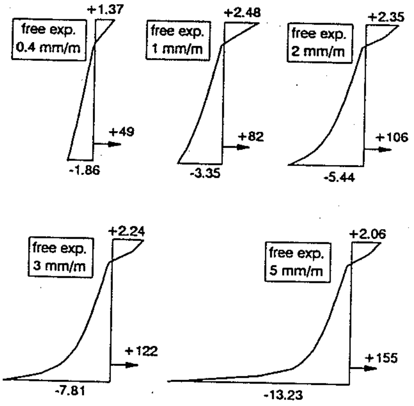

Fig 5. Predicted stress distributions across the sections of the singly reinforced beam tested by Cope et al, without loading during conditioning. Arrow represents reinforcement stress. Stress unit N/mm^{2}. Sign: + tension; - compression.

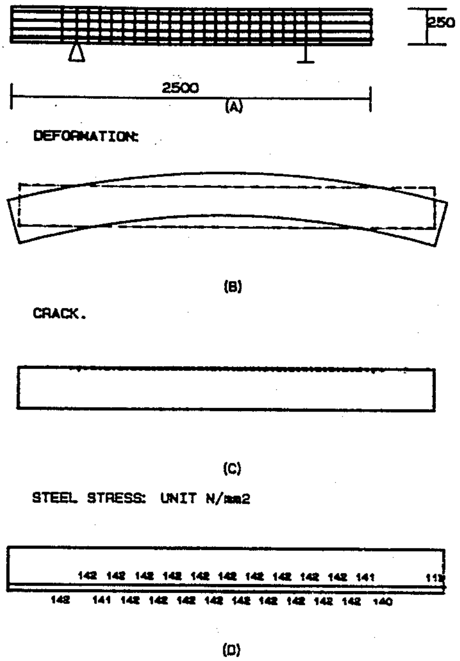

Fig. 3 A) Finite element model for the singly reinforced beam.

B) Deformation at free expansion of 5m/m (amplifying factor 5).

C) Cracking at top due to induced tensile stress in concrete.

D) Reinforcement stresses at free expansion of 5mm/m.

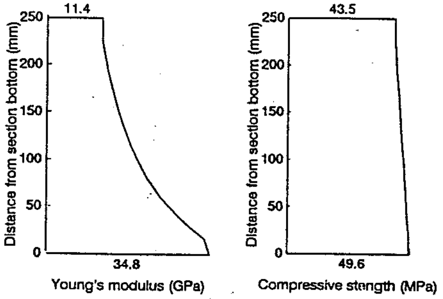

Fig. 6 Estimated Young's modulus and compressive strength through the section of singly reinforced beam conditioned unloaded.

across the mid span section when the free expansion is 5 mm/m using the material models described above.

From Figs 3 to 6 the following observations can be made:

a) The expansions due to AAR vary substantially throughout the depth of a beam. Fig.4 shows the difference between the expansion along the top and that along the reinforcement. The expansions predicted using elastic theory with stress-independent expansion vary less. The relatively lower expansion along the reinforcement demonstrates the sensitivity of AAR expansion to stress. The model proposed, taking into account stress history, was able to predict all the experimentally measured expansions closely. The differences between the predictions using the stress-independent expansion approach and the experimental results are significant. Strains predicted on the basis of stress-independent expansions are, therefore, not acceptable in the analysis of expansion effects due to AAR.

b) The distribution of stresses in a reinforced concrete member can be highly non-linear, Fig.5. This is a consequence of the shape of the stress-expansion curve. It can be seen that, in an un-symmetrically reinforced member, the induced compressive stress can be much higher than that which could be induced in uniformly restrained concrete. The maximum stress that could be induced in a uniformly restrained concrete is about 4 N/mm^2 (which would have prevented further expansion in the direction of the reinforcement) while the induced stress predicted at the soffit of the beam was about 13 N/mm^2.

c) The short vertical cracks along the top of the beam, shown in Fig.3C, are caused by the tensile stress generated by the differential expansion. AAR expansion can, therefore, induce tensile cracks in reinforced sections, in addition to those caused by the expansion of the gel within the aggregate and cement paste. In this example, tensile cracks along the top of the beam occurred at free expansions between 0.4 mm/m and 1 mm/m. After a free expansion of 2 mm/m, however, the cracks did not extend much further into the beam but widened slightly as the free expansion increased to 5 mm/m.

d) The predicted variations of the uniaxial compressive strength and the initial Young's modulus through the mid-span section for a free expansion of 5mm/m shown in Fig 6 are used in the analysis of the affected beam when it is subsequently subjected to external load, which is increased until member failure. It can be seen that the compressive strength is less sensitive to expansion than the Young's modulus: the maximum reduction in the strength, as determined from cube tests, was about 10% and the reduction in Young's modulus was up to 70%. This is in agreement with the trends in the material property-expansion relationship reported by other investigators and found from the authors' tests.

Example 2 : Danish beams

Bach et al *(Bach et al 1992)* have reported tests on beams without shear reinforcement (within the shear span) to investigate the effect of AAR on the shear capacity of members with very well anchored tensile reinforcement. These beams were also reinforced such that the shear failure occured at a load when the reinforcement was at only a low stress level. They found that AAR increased the shear capacity from 55 kN to 130 kN, despite the concrete strengths being reduced due to the AAR. Two of the beams in the tests have been analysed and the results are presented in this section. These are a control beam, A0, and a reactive beam, A12, with a free expansion of 5 mm/m.

Details of the beams are shown in Fig.7. The longitudinal reinforcement was cold-drawn steel with a nominal yield stress of 560 N/mm^2. The shear span to total depth ratio was 2.5. The beams were provided with long anchorage zones for the longitudinal reinforcement and also with stirrups outside the shear spans. The input data for the analyses are given in Table 1. The load deflection curves for the beams are given in Fig. 8.

Fig. 9A shows the finite element mesh used for the beams. Fig. 9B shows the predicted deformation of the beam, conditioned without load, subjected to AAR with a free expansion of 5 mm/m. The deformation has been amplified by a factor of 5. The predicted stresses induced in the concrete and reinforcement at a free expansion of 5 mm/m are shown in Figs.10 A to C. Comparison of the predicted crack pattern of the beams with and without AAR at a load near the failure load is shown in Fig.11.

a) The predicted failure mode of the control beam was a brittle shear mode with a major shear crack band in the shear span, Fig.11 B. This is in agreement with the mode of failure observed from the tests. The predicted failure load was, however, higher than the experimental value. The predicted failure load was 68.7 kN while the experimental value was 54 kN. There are several possible reasons for this. Some of the data required for the analysis were not available and were therefore estimated and there is usually considerable scatter of experimental results for the shear mode of failure.

b) In Fig.8, line OB represents the predicted hogging of the reactive beam, which was conditioned without loading. Line BC is the load-deflection curve from the analysis of load testing after the free expansion had reached to 5 mm/m. Curve OA is the predicted curve for the control beam. The corresponding curves from the tests are compared with the predicted curves in the figure. It can be seen that the predicted load-deflection curves are in general agreement with the test data.

c) The crack patterns observed in the tests for beam A5, which is similar to beam A12, suggested that the beam affected by AAR also failed in a shear mode but with the major diagonal crack forming further away from the support than that in the control beam A0. Between the lower end of the major diagonal crack and the support there are a series of short cracks which formed a short distance away from the soffit. The crack pattern seems to have been correctly reflected in the analysis, Fig. 11 A, although perfect bond was assumed.

d) The crack pattern described in paragraph c) above can be related to the stress distributions, shown in Fig.10, induced by AAR expansion. The compressive stresses induced in the concrete in the bottom of the member inhibit the tensile cracks occurring within a distance from the support, assuming that no bond slip occurs. Also there is vertical compression due to restraint of expansion due to the stirrups. Further away from the support, the tensile crack is delayed by the induced compressive stress. This delays the formation of the critical diagonal crack and forces the position of the crack away from the support. A series of short cracks in between the lower end of the critical diagonal crack and the supports observed in the tests for beam A5, were probably the results of the combination of the horizontal compressive stress and the

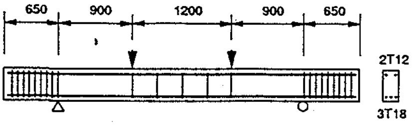

Fig. 7 Details of the beams tested by Bach et al.

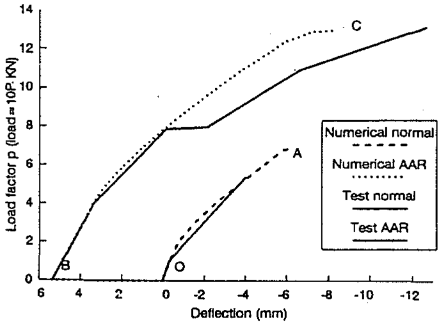

Fig. 8 Numerical and experimental load-deflection curves of the beams tested by Bach et al. The deflection measured on soffit under loading point.

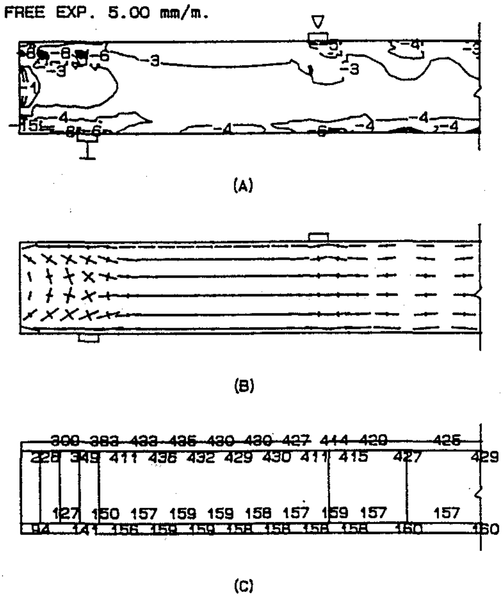

Fig. 10 (A) Concrete principal stress contours.
(B) concrete principal stress directions.
(C) reinforcement stresses in the reactive beam at free expansion of 5mm/m. Stirrups shown in (C) have yielded, stress - 400 N/mm².

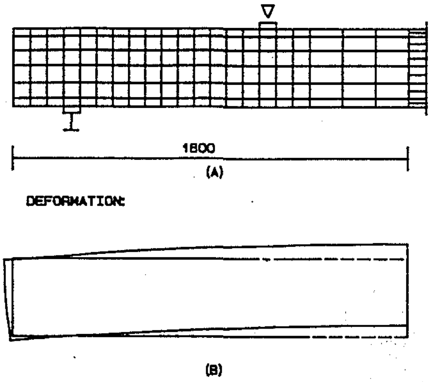

Fig. 9 (A) Finite element model for the beams tested by Bach et al.
(B) Deformation at free expansion 5mm/m

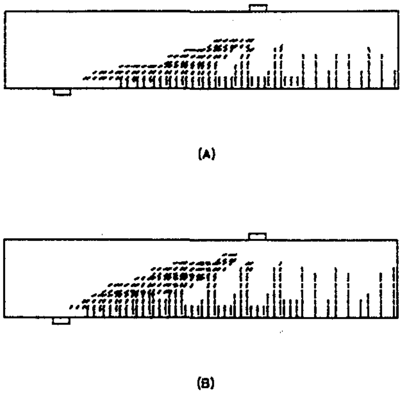

Fig. 11 Predicted crack patterns of the beams tested by Bach et al (A) with AAR and (B) without AAR.

shear stress. However, they may indicate local bond failure, which was not modelled in the analysis.

e) According to the analysis, when the control beam failed in shear, the maximum stress in the reinforcement was only 270 N/mm², which was less than half of the yield stress of the cold-drawn steel. For the reactive beam, the maximum predicted stress in the reinforcement was 540 N/mm², which was almost the yield stress, 560 N/mm², when the beam failed. The strength of the reinforcement was, therefore, fully exploited in the reactive beam. This was due to the prevention of the 'premature' shear failure, as in the unaffected beam, by the induced compressive stresses.

CONCLUSIONS

The agreement between the results from the analyses and the tests are generally good. The examples presented demonstrate and test the method. The technique provides useful insights into structural behaviour and explains test data. The tests on beams demonstrated that the effects of AAR on structural performance cannot be easily generalised. Its effects depend on the details of individual members and the loading regime during the expansion period. It has been demonstrated that the method described can provide a useful tool to investigate AAR effects in a range of structures.

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RESTRAINT EFFECTS ON THE PERFORMANCE OF VARIOUS ASR STRUCTURAL ELEMENTS

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ABSTRACT

Nearly all structural concretes that are affected by ASR were restrained during and subsequent to the ASR expansion. Therefore it is very important to consider the effects of external restraint on the behaviour of concrete elements affected by ASR. In this test programme, unreinforced cubes, cores and prisms were tested in this manner and various mechanical properties of the concrete determined both parallel and vertical to the direction of preloading. Test results from specimens with external restraint indicated a substantial reduction in damage compared to unrestrained specimens. The mechanical properties of the restrained samples were higher than those obtained from the unrestrained samples.

Keywords: axial load, crack modification, elastic modulus, external restraint, strength.

INTRODUCTION

It is well known that if ASR concrete is subjected to restraint from surrounding non-reactive concrete [1], reinforcement or applied stress then the dominant cracks form at right angles to the direction of the restraint as opposed to a random map cracking that occurs with unrestrained ASR expansion. It is therefore to be expected that the mechanical properties of ASR concrete will also be modified by restraint. Significant data has been published dealing with the effects of restraint on expansion [2-3] and expansion on compressive strength [4] but few authors have addressed the changes in other mechanical properties that are induced by restraint. Clark [5] who reported that restrained ASR specimens exhibited no reduction in their dynamic modulus of elasticity E_{cd} whereas similar unrestrained specimens exhibited 80% reduction in E_{cd} is one of the few authors to report this area. Chana [6] also reported that the loss of unrestrained elastic modulus due to ASR degradation was higher than the other mechanical properties.

TEST PROGRAMME

Size of the Specimens: The test programme was carried out using restrained cubes, prisms, cores, and reinforced concrete beams. The cubes were 100x100x100 mm, the prisms were 500x100x100 mm and the cores were 74 mm in diameter D and 200 mm in length L. In order to make the cores, blocks 250x250x200 mm were cast and four cores being taken from each block. The direction of drilling was also the direction of casting.

Mix Designs and Curing Conditions: The various mixes used were as detailed in Table 1. The cubes and prisms were made using mixes 1 to 6 of Table 1, whereas the blocks from which the cores were extracted used mixes 7 to 11. The fused silica was graded to pass through a 1 mm sieve, and the percentage of fine aggregate passing 0.6 mm was 55%. The particle sizes of the two aggregates were in the range of 4.8-20.0

mm and 4.8-10.0 mm. The ultimate expansion of the mixes at week 52 which were obtained from prisms are shown in Table 1 [7]. Ordinary Portland cement was used in this test programme and a sodium oxide equivalent level of 7.0 and 12.0 Kg/m^3 was achieved by adding KOH in the free water just prior to the mixing of the ingredients of the concrete. All the specimens were cured for four weeks in water at 20 °C from the day after casting and then transferred to the hot tank at 38 °C for another 48 weeks.

Table 1 - The proportion of various mixes used in the experimental work.

Mix No	20 mm Aggregate (Kg)	10 mm Aggregate (Kg)	Sand (Kg)	Fused silica (Kg)	Cement (Kg)	Water/ cement ratio	Na_2O equivalent (Kg/m^3)	Expan- sion (mm/m)
1	-	1060	300	240	550	0.41	12.0	15.54
2	-	1060	300	240	550	0.41	7.0	12.10
3	1060	-	300	240	550	0.37	7.0	12.82
4	-	1060	540	-	550	0.41	7.0	-
5	-	1060	540	-	550	0.41	-	-
6	-	1060	540	-	200	0.74	-	-
7	-	880	480	240	550	0.44	12.0	15.48
8	-	880	480	240	550	0.44	7.0	11.95
9	-	880	720	-	550	0.44	12.0	-
10	-	880	720	-	550	0.44	-	-
11	-	880	720	-	200	0.74	-	-

Preloading and Test Arrangements: The arrangements for preloading the samples is shown in Fig. 1. The intensity of preloading was 10 N/mm^2. The cores, were drilled from the blocks after they had been cured for 28 days at 20 °C. The cores were capped at both ends using the sulphur-carbon method. Half of the cores were restrained at 10 N/mm^2 and the rest unrestrained, after which they were all transferred into a hot tank at 38 °C for another 48 weeks. The cores were tested at week 4 and 52 to determine the modulus of elasticity and then compressive strength. The elastic modulus was found using the Stiffness Damage Test Procedure. In this method, incremental loads are applied up to 20 KN in 5 loops (cycles). In the calculation of the modulus of elasticity, only the average of the last 4 loops is taken into account.

Surface Characteristics of Restrained Cubes: The crack pattern in the restrained ASR concrete was significantly different from that of the unrestrained specimens. The restrained cubes exhibit a longitudinal macro crack which is continuous from one cube to the rest. This would suggest that ASR cracks will easily propagate along a single path from one part of an ASR damaged structure to another. The prisms and cores expanded at right angles to the direction of restraint and exhibited no signs of map cracking.

Performance of Cubes and Prisms under Stress: The cube is undoubtedly the most convenient specimen to use when large number of crushing test are required for concrete control purposes. Prisms on the other hand can give a much better estimate [8] of the uniaxial compressive strength of concrete. The performance of restrained and unrestrained cubes and prisms with various ASR and control mixes are presented in the Tables 3 to 4. The abbreviation used in this section are shown in Table 2.

TEST RESULTS

Cubes Test Results: Reference to the ASR mixes (1, 2 & 3) in Table 3, indicate that the compressive strength values of the restrained cubes of M1 and M2 are higher than that of identical unrestrained cubes (M). Results indicate that those tested parallel to the direction of restraint (M2) gain higher compressive strength compared to those tested perpendicular to the direction of restraint. The percentage increase of M1 and M2 ASR cubes for mix 1 (12 Kg alkali) were 8.3% and 25.7%, for mix 2 (7 Kg alkali) were 14.3% and 23.1% and for mix 3 (7 Kg alkali, gap graded mix) were 11.6% and 30.4% respectively. The ratio of the compressive strength of the restrained and unrestrained cubes of the ASR mixes in Table 4, columns 2 and 3 suggest that ASR mixes with lower levels of alkali gain more compressive strength due to restraint than the mixes with 12 Kg alkali. The compressive strength values of the control mixes 5 and 6 are shown in Table 3. It is seen that the cubes show no significant change in compressive strength due to restraint, regardless of the direction of restraint. However in the case of the mix 4 (control mix with excess alkali, Table 1), cubes tested both parallel (M2) and perpendicular (M1) to the direction of restraining indicate 21% increase in compressive strength with respect to the unrestrained cubes (M). The reason for the increase in compressive strength in this case is not understood particularly as it is noted both M1 and M2 cubes show similar increases in strength. It can be concluded therefore that ASR mixes exhibited higher compressive strength due to restraint and the increases differ depending on the direction of testing relative to the direction of the restraint.

Table 2 - Description of the symbols that have been used.

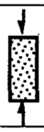
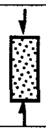
Letters	Description of the symbols	Letters	Description of the symbols
f_{cu}	Cube crushing strength.	M	Unrestraint cubes tested in a random direction.
P	Unrestraint prisms tested axially.	M1	Restrained cubes tested perpendicular to the direction of restraint.
P1	Restrained prisms tested axially		
E_c	Modulus of elasticity.	M2	Restrained cubes tested parallel to the direction of restraint.

Prisms Test Results: The values of the uniaxial compressive strength of the prisms are presented in Table 3. From the test results it is clear that the compressive strength values of the ASR prisms are lower than those of the controls. The mode of failure of the ASR prisms was progressive whereas in the case of the control prisms it was a sudden failure. Fig. 2 shows the types of the failure associated to the ASR and control prisms. It can be seen in Table 4 (column 1) that the uniaxial compressive strength of the restrained ASR prisms is greater than that of the unrestrained prisms. The level of alkali has an influential effect upon the increase in compressive strength as a result of restraint. When comparing the restrained and unrestrained prisms in Table 4 (column 1), it can be seen that with mixes 2 and 3 having 7 Kg alkali, the difference is 112.8% and 120.7% and in the case of prisms with 12 Kg alkali (mix 1) the difference is 49.1% only. Therefore the increase in uniaxial compressive strength in the restrained prisms with 7 Kg alkali is greater than the increase with 12 Kg alkali. It is seen in Table 4 that with control mixes 5 and 6 (column 1), the test values of the restrained and unrestrained uniaxial compressive strengths of the controls are similar to each other. Control mix 4 (with excess alkali) exhibited higher uniaxial compressive strength due to restraint but this increase is less significant than the increase associated with the ASR mixes.

Variation of Cubes and Prisms with respect to each other: The compressive strength of the cubes and prisms were compared with each other and the ratios are shown in the Table 4 columns 4 and 5. Regarding the unrestrained cubes and prisms in column 4, it can be seen that the ratios of the ASR mixes 1, 2 and 3 are higher than those of the control mixes 4, 5 and 6. The highest ratio of 181.6% belongs to mix 3 and the lowest ratio of 50.1% belongs to mix 4. The ratio of the 12 Kg alkali mix 1 is similar to that of the control mixes 5, and 6 yet the 7 Kg alkali mixes 2 and 3 do not show a similar ratio to the controls. Therefore the variation in ratio between the ASR and control mixes suggest that this ratio is higher for low level of alkali mixes. In columns 5 of

Table 4, the ratios of the restrained cubes and prisms are presented. These ratios unlike the unrestrained case in column 4, exhibit closer values between the ASR and control mixes. Therefore restraining did not show any adverse effect as far as these ratios are concerned. In column 1 of Table 4 the strengths of restrained prisms are compared with unrestrained prisms . In columns 2 and 3 the strengths of restrained cubes are compared with unrestrained cubes. It is seen that the gain in compressive strength due to restraint in the prisms is greater than in the cubes. The ratios in column 2 are lower than column 3 indicating that cubes tested parallel to the direction of restraint exhibit a higher strength than those tested perpendicular to the direction of restraint.

Table 3 - Typical test results of cubes and prisms of the alkali and normal mixes at week 52.

Names	1: Direction of restraining 2: Direction of testing		Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6
	1	2	f_{cu} (N/mm^2)					
P Prisms	Unrestraint		15.1 17.0 18.7	14.9 15.0 15.1	11.6 12.5 13.3	34.7 39.3 40.2	35.4 41.3 43.6	19.1 20.7 21.7
P1 Prisms			24.6 24.9 26.0	29.8 32.0 37.4	24.9 25.8 29.1	44.8 47.2 51.0	39.8 42.9 46.2	19.4 20.8 22.6
M Cubes	Unrestraint		31.5 32.0 33.3 34.0	31.0 37.5 40.5 41.8	31.5 33.2 37.2 38.7	53.5 55.5 59.0 61.0	63.0 68.5 72.0 74.0	35.0 36.0 36.0 36.5
M1 Cubes			33.0 35.2 36.0 37.5	41.0 41.5 43.2 46.8	36.1 39.0 40.0 42.0	68.0 69.5 70.0 70.6	61.5 67.5 71.5 74.0	35.5 36.0 37.1 37.2
M2 Cubes			39.5 41.0 41.5 42.5	44.5 45.5 47.5 48.2	41.0 44.5 46.1 52.0	66.0 66.7 70.0 74.0	68.0 69.5 72.0 75.0	33.8 35.5 37.0 37.5

Stiffness of Cores under Stress: In the short-term and under rather low stress, concrete behaves nearly elastically. In the long-term it is more complicated as "creep" plays an influential role in the magnitude of strain under constant stress. According to BS 8110 [9] the elastic stiffness of structural concrete can be determined from an empirical expression related to the cube crushing strength. Such values do not take into account the individual constituents in the concrete, such as cement paste or aggregate. The residual stiffness of ASR concrete is usually assessed from cores taken from the structure concerned. Tables 5 and 6 present the experimental and theoretical values of the elastic modulus E_c of the test specimens. In Table 5 values of E_c at week 4 when the expansion was very small have been presented. At this stage all the samples were unrestrained. The values in Table 6 show the results at week 52 with some of the specimens being restrained from week 4. The test results at week 4 and 52 are compared with their theoretical values which have been obtained from the BS [9]. The British Standard recommends that, for concrete with an average, high quality dense aggregate, the modulus of elasticity at 28 days, $E_{c,28}$, is obtained from the cube strength at 28 days $f_{cu,28}$ by the following expression:

$$E_{c,28} = 20 + 0.2 f_{cu,28} \qquad \text{(see Table 5)}$$

and the modulus of elasticity of concrete at an age (t) by

$$E_{c,t} = E_{c,28}\ (0.4 + 0.6f_{cu,t}/f_{cu,28}) \qquad \text{(see Table 6)}$$

Table 4 - Percentile ratios of the cubes and prisms.

Mix No	Percentile ratio %				
	1 $\frac{P1-P}{P}$	2 $\frac{M1-M}{M}$	3 $\frac{M2-M}{M}$	4 $\frac{M-P}{P}$	5 $\frac{M2-P1}{P1}$
1 (ASR), 12 Kg	49.1	8.3	25.7	93.4	63.1
2 (ASR), 7 Kg	120.7	14.3	23.1	151.3	40.2
3 (ASR), 7 Kg	112.8	11.6	30.4	181.6	72.6
4 (control)	25.1	21.5	21.0	50.1	45.1
5 (control)	7.2	-1.2	2.4	73.1	65.3
6 (control)	2.0	1.4	0.3	75.1	72.2
Column 1	Restrained prisms compared with unrestrained prisms.				
Column 2	Restrained cubes tested perpendicular to the direction of loading compared with unrestrained cubes.				
Column 3	Restrained cubes tested parallel to the direction of loading compared with unrestrained cubes.				
Column 4	Unrestrained cubes compared with unrestrained prisms.				
Column 5	Unrestrained cubes tested parallel to the direction of loading compared with restrained prisms.				

Effect of ASR upon Elastic Modulus: In general the rate of expansion and the total expansion depend very much on the reactive aggregate, cement type, cement content and the environment [10]. Therefore the effect of ASR on the engineering properties of concrete and in particular, upon the elastic modulus cannot be generalised. In order to evaluate the elastic modulus of the ASR specimens, 2 ASR concretes were used and compared with 3 control mixes. The variation of the compressive strength of the mixes are also shown in the Tables 5 and 6 and used to find the theoretical elastic modulus of the mixes at week 4 and 52. The results clearly indicate a significant reduction in the elastic modulus compared to that of the controls. Factors that effect the elastic modulus of ASR concrete are discussed below.

Table 5 - Elastic modulus and compressive strength of cores at week 4.

Block No	Core No	Cube f_{cu} N/mm^2	Core E_c N/mm^2	Mean E_c N/mm^2	$E_{c,28} = 20+0.2f_{cu,28}$ KN/mm^2	Mix details
A	A1, A2 A3, A4	34.5	18.6, 24.7 20.2, 21.4	21.6	26.90	Mix 7, ASR mix, 12 Kg alkali.
B	B1, B2 B3, B4		22.7, 20.7 22.2, 22.8			
C	C1, C2 C3, C4 C5, C6 C7, C8	38.2	22.6, 21.9 23.7, 25.1 24.0, 19.4 21.1, 22.4	22.5	27.64	Mix 8, ASR mix, similar to 3 but 7 Kg alkali.
D	D1, D2 D3, D4	36.5	36.3, 37.3 37.7, 39.6	37.8	27.30	Mix 9, control mix, 12 Kg alkali, no reactive aggregate.
E	E1, E2 E3, E4	57.0	43.4, 47.6 44.1, 44.5	44.9	31.40	Mix 10, control mix, no alkali, control mix.
F	F1, F2 F3, F4	30.6	36.6, 38.6 32.7, 36.9	36.2	26.12	Mix 11, control mix, no alkali, 200 Kg cement/m^3.

Effect of curing upon elastic modulus: The data in Table 5 indicates that at week 4 the values of compressive strength and elastic modulus of the ASR concrete are lower than that of the control mixes. ASR cores series A and B (mix 7 with 12 Kg alkali) have 39% and 52% lower compressive strength and elastic modulus than the cores series E (control mix 10). In the case of mix 8 (7 Kg alkali) these values reach 33% and 50% compared to the control cores series E. This indicates that the losses in compressive strength and elastic modulus at week 4 are similar, regardless of the alkali level. In Table 6 comparing the unrestrained ASR cores (i.e. A4, B1, B2 and C5 to C8) with the control cores (i.e. series D, E and F) at week 52, it can be seen that with 12 Kg alkali (i.e. A4, B1 and B2) the values of compressive strength and elastic modulus have dropped by 57% and 83% whereas with 7 Kg alkali (C5 to C8) the compressive strength and elastic modulus have reduced by 55% and 82% compared to the control cores (i.e. E3 and E4, mix 10). Therefore with both 7 and 12 Kg alkali the elastic modulus exhibited greater degradation than the compressive strength. Cores B3 and B4 in Table 5, have been kept out of water in the laboratory after week 4. It is seen that the modulus of elasticity of cores B3 and B4 are slightly higher than that of cores A4, B1 and B2 which have been cured at 38 °C.

Table 6 - Elastic modulus and compressive strength at week 52.

Core No	f_{cu} Cubes N/mm^2	f_{cu} Cores N/mm^2	f_{cu} Cores mean	E_c KN/mm^2	E_c mean KN/mm^2	$E_{c,t}$ KN/mm^2	Loading case
A1, A2 A3	44.4	45.4, 44.4 47.5	45.8	18.1, 14.4 20.4	17.6	25.3	Restrained
A4, B1 B2	32.5	24.8, 19.5 20.0	21.4	8.3, 9.5 7.0	8.3	20.8	Unrestrained
B3*, B4*	36.3	28.0, 28.7	28.4	8.3, 10.7	9.5	22.3	Unrestrained
C1, C2 C3, C4	45.2	47.3, 46.7 45.1, 48.6	46.9	17.3, 18.6 14.0, 15.2	16.3	25.0	Restrained
C5, C6 C7, C8	33.8	23.9, 25.4 24.3, 21.2	23.7	8.8, 9.9 9.0, 7.1	8.7	20.9	Unrestrained
D1, D2	66.6	59.0, 61.7	60.4	33.2, 33.5	33.4	56.5	Restrained
D3, D4	55.1	48.9, 50.1	49.5	27.9, 35.8	31.8	49.4	Unrestrained
E1, E2	75.2	64.8, 62.8	63.8	49.8, 49.2	49.5	53.5	Restrained
E3, E4	74.9	71.6, 65.4	68.5	49.2, 49.7	49.4	53.4	Unrestrained
F1, F2	44.5	38.9, 39.4	39.2	39.8, 47.1	43.4	52.6	Restrained
F3, F4	42.7	42.3, 36.4	39.4	44.0, 42.1	44.6	44.8	Unrestrained
*B3 and B4 cured in the laboratory condition							

Effect of alkali upon elastic modulus: Series D cores are made with mix 11 (control mix with extra alkali only). Comparing cores D and E (normal control mix), indicates that the enhancement of alkali has lowered the compressive strength and elastic modulus of the cores. The decrease in compressive strength and elastic modulus at week 4 are 36% & 16% and at week 52 are 26% & 36% respectively. This indicates that curing specimens for 52 weeks at 38 °C improves the compressive strength, compared to week 4 and reduces the elastic modulus. Cores series D and F (control mix, low compressive strength) at weeks 4 and 52 exhibits a similar effect.

Effect of restraint upon elastic modulus: All the specimens in Table 6 have been cured at 38 °C from week 4 to week 52. Half of the cores from each mix have been restrained from week 4 to week 52. It is seen that the elastic modulus of the restrained ASR cores are almost double than that of the unrestrained cores, in both 7 and 12 Kg alkali cores. In the case of the control mixes, the effect of the restraint is not significant, save in the case of cores series D, mix 9 (control mix with extra alkali only), where the elastic modulus of the restrained specimens is slightly higher than that of the unrestrained.

Comparison of the Theoretical and Experimental Values: Reference to the Tables 5, indicates that at week 4 the theoretical values of E_c in the ASR cores are higher than the experimental values, whereas in the case of the control mixes, experimental values are higher than the theoretical values. It is seen, in Table 6 that at week 52 the control mixes have lower experimental values of E_c than the theoretical values. In the case of the unrestrained ASR cores, with 7 and 12 Kg alkali they show 58% and 60% losses of elastic modulus compared to the theoretical value. However when the cores are restrained these losses reached 35% and 30% respectively. Therefore unrestrained cores exhibit greater loss of elastic modulus compared to the theoretical values, and when cores are restrained the loss of elastic modulus is lower.

CONCLUSION

[1] The expansion in the direction of stress is considerably reduced but increased expansion is exhibited in the direction at right angle to this. The cracking is also effected with cracks forming along the stress direction rather than the map cracking which occurs in unstressed concrete.

[2] The results of restrained cubes indicated that, preload decreased the amount of damage in both the parallel and perpendicular directions, however, the damage in the direction of preloading was less than the perpendicular direction.

[3] The loss of strength due to ASR is very much reduced in the direction of long-term stresses compared with the cross direction where the loss in strength is usually greater. The gain in compressive strength and elastic modulus in the uniaxial prisms due to restraint was higher than the gain in compressive strength from cubes.

[4] Application of external restraint modified the crack patterns on the surface of the ASR concrete. This indicates that a constant applied compressive stress can eliminate cracking in the direction perpendicular to the compression. A stress of 10 N/mm^2 was sufficient to stop the cracking in the lateral direction. Although lower stresses were not tried, lower stresses would be expected to give lower increases in the mechanical properties. Increase in the uniaxial compressive strength of the prisms was 120%, the increase in the elastic modulus of cores was 112% and the increase in the compressive strength of cubes was 30%.

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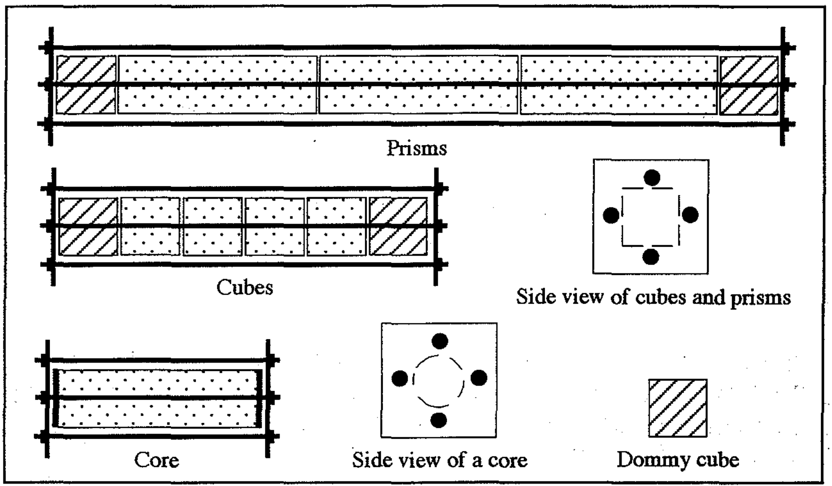

Fig. 1 - Method of applying external restraint on various specimens.

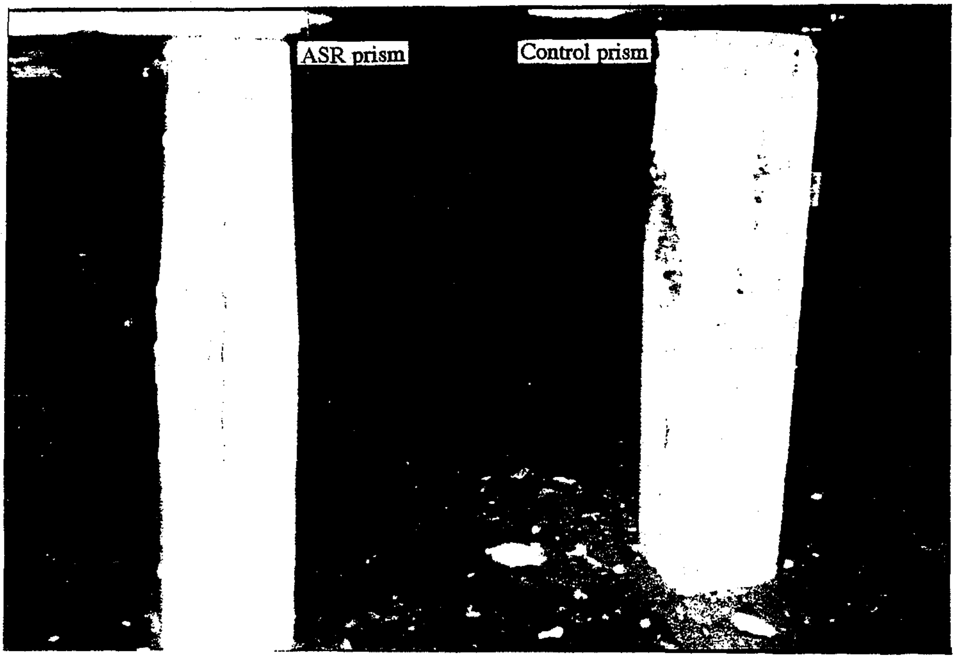

Fig. 2 - Failure of ASR and control concrete prisms.

RELATING ASR STRUCTURAL DAMAGE TO CONCRETE COMPOSITION AND ENVIRONMENT

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ABSTRACT

Over 100 core samples have been taken for analysis from nine bridges and a river wall in South West England. The structures all showed some ASR type cracking and most had already been subject to detailed diagnosis, monitoring and test programmes for 10 years. On each structure samples were taken in matched sets of nominally identical mix composition, from identical structural members in similar environments. These sets cover concrete pours ranging from uncracked through to severely cracked. The emphasis was on pours with cracking from slight to moderate expansion and the most extreme cracking was deliberately avoided. The degree of cracking and ASR damage has been assessed using procedures and tests from the IStructE 1992 Report 'Structural Effects of ASR'. For some structures insitu RH has been measured and expansion tests have been undertaken to determine potential for further damage. The cores are being analysed for alkali contents and petrographically so that the relationship of mix characteristics to field damage can be evaluated.

Keywords: Structural ASR Damage, Petrography, Diagnosis, Alkali Analysis.

INTRODUCTION

This paper sets out the general approach and methodology being developed in a study by SS&D and BRE to relate the severity of structural effects in a range of structures with ASR to the nature and variability of the composition of samples of the concrete. Some initial trends are indicated here and the further results will be published in due course. It is hoped that this study will help bridge the gap between the procedures and data used in structural analysis and those used in diagnosis, petrography and laboratory studies on ASR.

The knowledge of the composition of concrete in structures with ASR damage in the field potentially provides the best basis for developing and calibrating specifications for minimising the risk of damage from ASR (Hawkins 1995). However the difficulties of reliably relating the analysis of core samples to the original mix composition and in quantifying the current and future ASR damage have deterred studies in this field. Compared to the massive literature on accelerated short term tests on mortar bars and concrete prism testing, there is little data on the field composition related to damage and most of that is based on relatively few samples.

Nixon et al. 1987 carried out a detailed study of alkali variation on ASR cracked foundations. Since then a number of limited studies of alkali contents and variability in structures have been carried out by the authors and others (eg Livesey et al. 1996), which have further developed the procedures. It was considered that the methods of assessing damage in structures set out in the "Structural Effects of ASR" (IStructE 1992) and the developments in detailed petrographic diagnosis and analysis of concrete, including those in "Diagnosis of ASR" (Palmer 1992), provided the basis for a more comprehensive programme to evaluate the composition of concretes which are suffering ASR damage in the UK. However it was appreciated that it would be necessary to develop and refine procedures as data came available.

The South West of England has many cases of ASR damage (Wood 1992). In a few structural elements this has caused serious structural damage, but in most it has just produced some unsightly cracking without structurally detrimental damage. Since the early 1980s Mott MacDonald, and more recently Structural Studies & Design, have carried out detailed structural assessments (Wood & Johnson 1993) on about 50 structures with ASR in the South West of England. Agreement was reached with the owners of 10 of these structures for the coring for samples for detailed analysis by BRE. Relevant data from the owners testing and structural assessment programmes is being made available to BRE to complement their data.

SELECTION OF STRUCTURES AND CORE SETS

The primary objective of the sampling programme was to obtain matched sets of 6 to 10 cores from very similar structural elements, which showed a range of ASR cracking damage, ie from no apparent cracking up to moderate damage (1mm/m). With a few exceptions, pours of concrete showing severe cracking (> 2mm/m) were avoided, so that the compositional data would be concentrated on samples at the interface between 'ASR with no significant damage' and 'ASR with just significant damage', which is of most interest for calibrating specification requirements.

Table 1 Summary of coring for all structures

Ref	Structure	Town	Owner	Built	Cores Taken	IStructE AAR Class	Aggregate Type
1	Exe Bridge, Upstream.	Exeter	Devon CC	1968 - 70	14	B - D	R
2	Exe Bridge, Downstream.	Exeter	Devon CC	1970 - 72	5	C - D	R
3	Marsh Mills Viaduct	Plymouth	DTp	c1970	32	A - n	S(G) & S(L)
4	Plympton Hill	Plymouth	DTp	c1970	12	C - D	S(G) & S(L)
5	Voss Farm Column	Plymouth	DTp	c1970	3	A	S(L)
7	Burbarrow Bridge	Nr Bristol	Avon/ DTp	1966 - 67	8	A - D	T
8	Tiverton River Walls	Tiverton	NRA	c 1970	12	A - D	U
10	Countess Wear Flood	Exeter	Devon CC	1966	11	D	R
13	New Grindle Brook	Clyst St Mary	Devon CC	1967	15	C - D	R
14	Kempstown	South Molton	Devon CC	1976	6	B	U

About 30 structures were considered, before selecting 10 where there was easy access to suitable matched sets. Table 1 shows the main details of the selected structures including the range of 'Structural Element Severity Ratings' for the cored elements determined from the IStructE 1992 procedures. In each structure, sets of elements were selected which had been cast in successive pours of concrete and which showed a range of severities of damage. For example, Plympton Hill Bridge has 2 rows of 6 columns with cracking damage apparent in the outermost columns which are slightly more exposed to weather. Six of these columns were cored for analysis. A similar set was taken on the columns of Marsh Mills Viaduct Fig. 1 and from the severely damaged column from the adjacent Voss Farm Bridge, which was retained after demolition. On other structures the sets of cores have been taken into retaining walls and bridge abutments. As far as possible coring locations were selected so that the risk of alkali migration leading to concentration and/or leaching was minimised.

The 10 structures cover four distinct aggregate types, as shown in Table 2 below. Most of them had cement from Plymstock cement works, which is known to have had high alkali, up to 1.2% Na_2O equiv. in the early 1970s, but this has since been reduced. All the structures had been petrographically diagnosed as having ASR as a major factor in the development of cracking, or were of a similar mix to cases which had been diagnosed. Confirmatory diagnosis is included in the test programme.

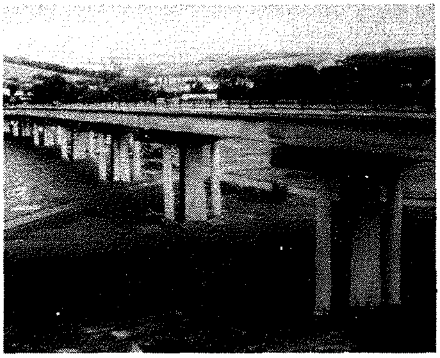


Figure 1. Marsh Mills Viaduct

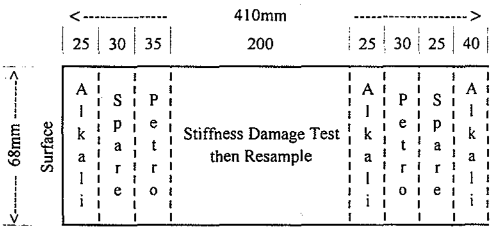


Figure 2. Typical Initial Core Subdivision

CORING AND CORE SUBDIVISION

The coring was carried out to a standard procedure using wet diamond coring with a 75mm barrel to give a 68mm diameter core. The use of larger cores increases the risk of cutting reinforcement, which can both damage the structure and make the cores less suitable for structural testing. Core locations were selected off the line of reinforcement and at least 100mm from a surface crack to ensure, as far as possible, that intact cores were obtained. In most cases cores of 400mm to 600mm length were obtained to provide sufficient material so that sets of tests could be carried out on one core to facilitate direct comparisons between the results. The coring was carefully supervised and logged with cores cleaned, dried, cling film wrapped and then packed for delivery to BRE. Checks have confirmed that wet coring does not lead to significant alkali loss.

After initial inspection and recording at BRE, the cores were dry cut into ~25mm slices, or 200mm lengths for Stiffness Damage Testing, prior to further tests. A typical initial subdivision is shown in Fig. 2. As the data from the initial testing becomes available further selections for sample subdivision and testing are made.

SITE MEASUREMENTS OF DAMAGE AND HUMIDITY

To quantify the degree of damage from ASR, crack summation has been used to estimate 'expansion to date' following the IStructE procedures. At each coring location the width and spacing of cracks were recorded. However these cracks result from the combined effects of ASR expansion, thermal, shrinkage and structural strains, in typically 1m^3 of concrete, in the vicinity of the core. These estimates of 'expansion to date' were made on a simple basis ignoring the effects of stress and restraint. For more detailed analysis, where stress or restraint levels are significant, their influence in reducing or increasing the crack intensity is being considered. In most cases, sampling was carried out in areas of low stress and low restraint so that the simple estimate of expansion would be valid.

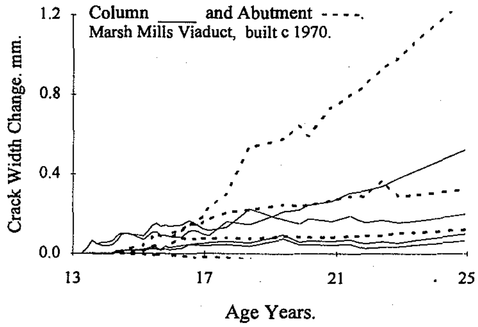
There are many causes of differential strains which develop in concrete as it is cast, matures and dries. Differential thermal strains and shrinkage strains between the surface and core of the material are the 2 main phenomena. These typically induce strains of 0.2 - 0.4mm/m, which is close to the tensile strain capacity of concrete (0.3 -0.4mm/m) at which cracking initiates. This is shown on IStructE 1992 Fig. 13. As ASR effects are superimposed on these normal residual strains, the initial cracking, with ASR expansions of up to 0.6mm/m, arises mainly from the interaction of ASR with thermal and shrinkage effects. In setting limits to ASR expansion in specifications it will be important to distinguish between ASR expansions which, by themselves, will induce cracking and the lower level of ASR expansion which will unacceptably accentuate the cracking from early thermal and shrinkage effects.

The timescale of crack development with UK aggregates is still uncertain. From records of monitoring crack widths on many structures since the mid 1980s there is clear evidence of a progressive increase in crack widths with time, as shown for structural members cored for BRE on Marsh Mills Viaduct in Fig. 3.

The long term equilibrium relative humidity in the concrete at a depth of 75 - 100mm has been recorded adjacent to the core location on many of the structures. A wooden dowel is sealed into a 16mm diameter hole and allowed to equilibrate over a period of months. The moisture content of the dowel is used to determine the equilibrium relative humidity RH. The results show significant variations in RH typically between a 23% dowel mc (ie 94%RH) and 50% dowel mc (ie 100% RH) within a bridge abutment. This moisture variability is clearly a factor in the rate at which the potential expansions and cracking develop in different parts of the structure.

STRUCTURAL TESTS

As most of the 10 structures have already been subject to detailed structural testing programmes the additional testing for this study has been concentrated on the expansion testing of some cores and the Stiffness Damage Testing of most cores.



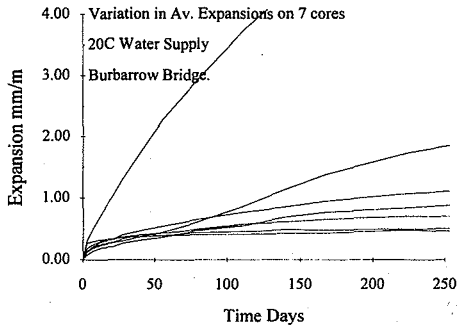
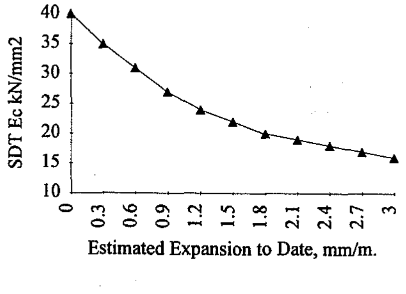


Figure 3. Long term crack movements.

Figure 4. Expansion test results.

Expansion testing, at 20°C in water supply conditions, has been carried out on some of the 200mm core lengths after Stiffness Damage Testing. This test shows if the ASR has stabilised or if it may in time, and with sufficient moisture availability, develop more severe cracking. Fig. 4 shows a typical example of the high variability of average expansions for seven cores from Burbarrow Bridge. There is a similar variability in expansion along the length of each core. Expansion tests over 5 or more years are needed to indicate the long term potential for further expansion. Data on these and other structures shows a good relationship between expansion tests and site monitoring of movements and crack growth. In comparisons between mix composition and ASR damage to structures, the potential for further expansion and cracking must be considered.

The development of ASR microcracking produces a marked reduction in the stiffness of concrete and the development of hysteresis (Wood et al. 1989). This provides a sensitive indication of 'expansion to date' for the material being analysed. 200mm lengths were tested at Sheffield University using the Stiffness Damage Test (SDT) to measure the Young's Modulus (Ec) and the hysteresis (DI). Fig. 5 shows the estimated relationship between Ec and expansion to date which is being refined and re-calibrated using data from this study and laboratory studies (Jones 1994). Fig. 6 shows the range of expansions to date estimated from the SDT Ec for the full set of samples and shows how the material covers the range from normal concrete to 'expansions to date' of over 2mm/m.



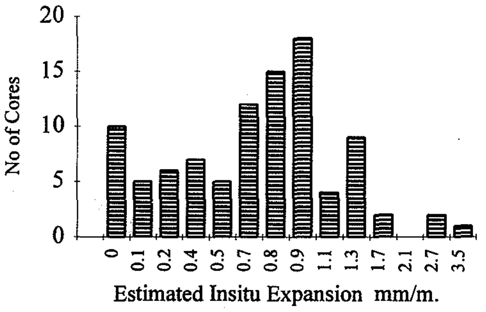


Figure 5. Estimated Expansion from SDT.

Figure 6. Range of Estimated Expansions.

ALKALI ANALYSIS

The 25mm concrete slices were crushed, dried and ground according to the British Standard method (BS1881 Pt. 124 1988). Acid soluble and water soluble extracts were analysed for sodium and potassium oxides. The extraction procedure can extract alkalis from aggregates which would not be 'effective' in ASR. Blank determinations are necessary to separate these from the 'effective' alkalis, predominantly from cement, considered in specification (Hawkins 1995). In the case of aggregate types R and S (Table 1) aggregate alkalis have been shown to be insignificant. In the case of other aggregate types further analyses are required.

An example of the analytical data obtained is shown in Fig 7 for samples from Kempstown bridge abutments. This shows considerable scatter in the alkali contents of individual 25mm slices and there are differences between the surface and heart concrete alkalis. The depth of surface effects due to weathering and contamination must be determined. These will depend on the quality and integrity of the concrete and the exposure conditions. In estimating the original mix alkali content only the analyses from the interior should be taken into account.

With a sufficient number and distribution of slices the original mean alkali content can be determined together with variations between cores, between pours and between different elements in the structure. The mean alkali content of cores from different locations can indicate whether there are significant differences in the concrete composition due to cement content or alkali variation. This can be a valuable guide for the interpretation of different degrees of deterioration within the same structure.

The size of the 25mm x 68mm diameter slices was chosen to provide a measure of the local concrete composition. It was recognised that surface effects and the heterogeneity of concrete would need to be considered before relating local composition to that in the structure. In comparing alkali data with tests the scale of sample needs to be considered. Slice alkali data may relate to petrographic measures of local damage. Average expansion and stiffness data on cores may relate to the average core alkali. Alkali values averaged over larger volumes need to be considered for assessing overall structural damage and defining alkali for specification. Further analyses of structures will help to refine the statistics and their significance.

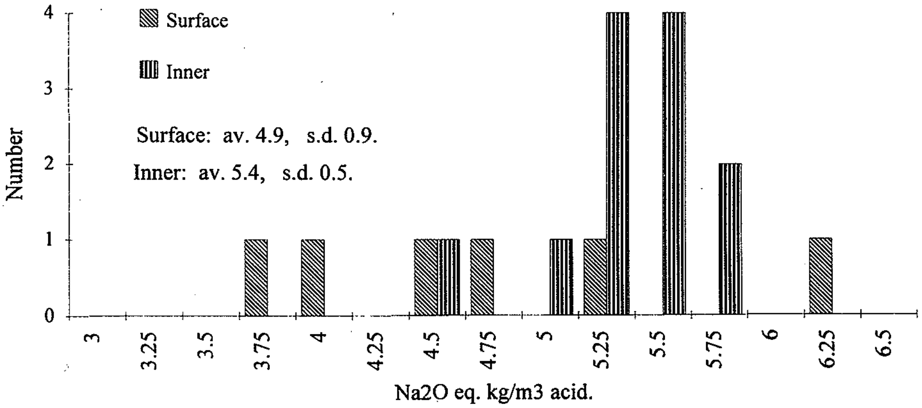


Figure 7. Kempstown. Na_2O Equiv. (Acid). Variation at surface and at depth.

PETROGRAPHIC ANALYSIS

The initial petrographic analyses was carried out to:

- categorise the mix types of various cores, see Table 2.
- identify the potentially reactive rock types and ASR features for diagnosis.
- assess the severity of microcracking in the samples.

The refinement of petrographic examination has enabled ASR damage to be identified in samples, even when it does not have sufficient effect to produce identifiable cracking on the structure. Further data to relate the level of ASR microcracking damage recorded petrographically to the site severity of structural cracking and the Stiffness Damage Test data, is being collated. The initial petrographic analyses are identifying features which merit more detailed study and this is in progress. This includes evaluation of other mix characteristics and deterioration processes which may have interacted with ASR to develop the cracking.

Table 2. Mineralogy of aggregates

Concrete Type R Structures 1, 2, 10, 13.
Aggregate: Part crushed natural gravel, predominately sandstones with metaquartzite, vein quartz and a little chert.
Reactive Minerals: Mainly Sandstones and some Chert

Concrete Type S Structures 3, 5, 6,
Coarse Aggregate: Moorcroft limestone in S(L). Hinkston Down granite in S(G).
Fine Aggregate : Cherts with finer quartz grains and some shell.
Reactive Mineral: Chert 3 to 8mm in fine aggregate.

Concrete Type T Structure 7.
Coarse Aggregate: Carboniferous limestone.
Fine Aggregate : Severn sea dredged with a little chert and shell.
Reactive Minerals: Chert

Concrete Type G Structures 8, 14,
Coarse Aggregate: Crushed rock mainly greywacke.
Fine Aggregate: Quartz rich with sandstones etc
Reactive Minerals: Greywacke

CONCLUSIONS

The results provide a quantitative record of mix composition relative to the local severity of cracking and structural effects from Alkali Silica Reaction which can help relate laboratory studies to field performance.

This methodology enables surface effects to be differentiated from the inner concrete characteristics to determine to the original concrete composition. It is possible to identify significant differences between pours or between concrete elements. When mix analysis is used in conjunction with measures of expansion, cracking and microcracking severity, insitu RH values and core tests for residual expansion and stiffness, it provides a basis for the assessment of a structure.

The variability found in the analysis of composition, test data on cores and in crack damage to structures highlights the need for comprehensive sampling, so that values of the means and standard deviations can be determined for all tests. There is a significant risk that tests and analyses based on isolated spot samples may give misleading information.

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DEFORMATION BEHAVIOR OF REINFORCED CONCRETE BEAMS DETERIORATED BY ASR

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ABSTRACT

Experimental studies were made on the behavior of singly reinforced concrete beams without shear reinforcement which were severely deteriorated with wide cracks caused by alkali-silica reaction (ASR). Analytical study was also made on the mechanical property of the beams, especially on the effect of pre-stress caused by the restriction of ASR expansion by reinforcement. Compared with beams without deterioration, yield starting bending moment of the severely deteriorated beams decreased. There are two possible reasons. One is because neutral axis depth increased and the length of moment arm decreased due to the presence of prestress, and the other is low Young's modulus. In case of the deteriorated beams, deflections at the yield starting point decreased because flexural rigidity of the deteriorated beams increased. The ultimate bending moment did not change, whether deterioration due to ASR was heavy or nil, because after the start of yielding, flexural cracks extended and moment arm length increased rapidly even in case of the deteriorated beams and the length, consequently the ultimate moment, came to be the same.

Keywords: ASR, beam, deformation, yield starting moment, ultimate bending moment

INTRODUCTION

Various approaches have been made to investigate the mechanical properties of reinforced concrete members damaged by alkali-silica reaction (ASR) for the safety assessment of damaged structures.

It has been reported, as a result of loading tests on beam members, that the stress due to ASR-induced expansion restrained by reinforcement increases the rigidity of the member, in some cases shifting the failure mode from shear to flexure(Hobbs 1988) and enhances shear capacity(Cope & Slade 1992). ASR damage therefore does not necessarily reduce the ultimate load-bearing capacity, but rather can increase it (Koyanagi *et al.* 1987). While, it has also been reported that such damage reduces the yield strength of beams (Inoue *et al.* 1989), and this has been ascribed to the effects of the existing stress of reinforcement.

The authors discuss here the mechanical properties of reinforced concrete beams with different steel ratio, which were deteriorated severely by ASR with crack widths of 0.6 to 0.7 mm. Static loading tests were made on the reinforced concrete beam specimens with and without deterioration, and strength properties such as cracking, yield and ultimate moments as well as deformation properties were obtained. Analytical study was also made especially on deformation properties.

EXPERIMENTAL PROCEDURE

Scope of the experiments

The beam specimens investigated were rectangular beams 10x18x170 cm in size with singly reinforcement and without shear reinforcement. Two levels of reinforcement ratios, "S" and "L", were employed as variables in combination with Series A with ASR damage due to alkali addition and Series N with no ASR damage. These made four types of beam specimens: AS, NS, AL, and NL beams.

Materials and mix proportions

The cement used was ordinary portland cement (Equivalent Na_2O = 0.61%). As for the fine aggregate, non-reactive sand from the Nagara River was used. As for coarse aggregates, gravel from the Nagara River and crushed bronzite andesite (Dissolved silica:Sc = 180, Reduction in alkality:Rc = 124 mmol/lit) were used as inert and reactive coarse aggregates, respectively. As reinforcement, deformed bars of 10mm (D10: fy = 380 MPa, fu = 535 MPa) and of 13 mm (D13: fy = 371 MPa, fu = 556 MPa) were used.

The unit content of water and cement was 176 and 352 kg/m^3, respectively. The sand-aggregate ratio was 0.45. When using the reactive aggregate, its portion was adjusted to 60% of the total coarse aggregate. The alkali to be added is Na_2SO_3, which has a high reactivity while scarcely affecting the concrete properties. Total equivalent Na_2O content of 3% of cement weight was adopted. The slump and air content of freshly mixed concrete were approximately 10 cm and 4%, respectively, for both N and A specimens.

Specimen, loading and measurement

The beam specimens were fabricated using two D10 or D13 bars as the main reinforcement with a cover depth of 20 mm. The steel ratios of S and L specimens were 0.92% and 1.66%, respectively. Shear reinforcement was not provided.

The beam specimens were wet-cloth-cured in a thermostatic chamber of 20°C for 4 weeks after demolding. "A" beams were then spray-cured with the room temperature raised to 40°C for approximately 4 months to accelerate ASR, and then left in the thermostatic chamber of 20°C. "N" beams were left in a laboratory atmosphere for 4 weeks after being wet-cloth-cured.

Loading tests were conducted on the beam specimens at an age of 2 years. The loading was applied on two symmetrical points with a span of 125 cm. Two A beams and one N beam of Series S and Series L were tested. The following measurements were made: the loads, the displacements at the loading points and supporting points and the longitudinal deformations on one side of the beam within the moment span by high-sensitivity displacement gages fixed at 3 cm from the top and bottom, with the gage length of 20 cm. The state of the loading and measuring manner and the cross section of the specimen are shown in *Fig. 1.*

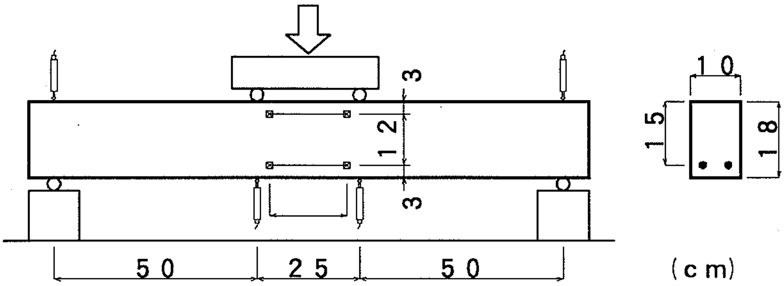


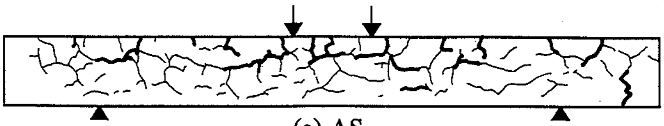
Fig. 1. Loading and measuring manner and cross section of beam.

Along with the beam tests, various mechanical properties of concrete were obtained; the static Young's modulus and compressive strength using specimens 10 cm in diameter and 20 cm in length, the dynamic Young's modulus and flexural strength using specimens 10 x 10 x 40 cm in size, and the splitting tensile strength using specimens 15 cm in diameter and 15 cm in length. These specimens were cured and stored as same manner as beam specimens.

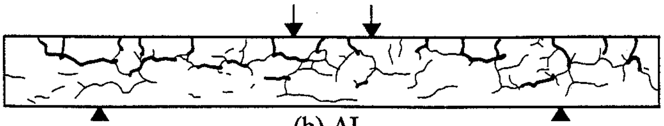
RESULTS OF EXPERIMENT

Cracking due to ASR

Numerous cracks developed in A beams during the spray curing at 40°C. The cracks concentrated mostly in the upper part of the beams where no reinforcement was provided. The cracks were mostly longitudinal near the vertical center and transverse on the top edge, and the width of some of them reached 0.6 to 0.7 mm. Typical crack patterns are shown in Fig. 2. Both S and L beams showed a warp (upward deflection) of 12 to 14 mm for total length of 140 cm. The differences in the reinforcement ratio led to no significant differences in the cracking pattern and the amount of warp.



(a) AS



(b) AL

Fig. 2. Typical crack pattern of deteriorated beams

Results of beam test

All the S series beams failed in flexure. On the other hand, in the L series with a large reinforcement ratio, the N beams without ASR showed a shear failure, while the A beams with ASR showed a flexural failure.

The initial portion and the whole measured load–displacement curves of the beams are shown in *Figs. 3a and 3b*, respectively. The curves of the A beam couples agree well for both S and L. From *Fig. 3a*, both S and L beams of Series A have higher initial rigidity than those of Series N. The load at which the slope of the curves abruptly changes, which corresponds to the first crack, is 60% or more higher for Series A than for Series N. *Fig. 3b* reveals that, regarding the S beams, which failed in flexure, the yield starting point of Series A is approximately 10% lower than that of Series N, but the load–carrying capacity of Series A increases as the deformation increases after yielding, resulting in almost the same or slightly larger ultimate load–bearing capacity than that of Series N. The maximum loading point can be called as the strength failure point. The deformation at the strength failure point of series A is larger than that of series N. As for the L beams, a direct comparison is difficult, because the Series N beam failed in shear. However, a marked increase is observed in the load–bearing capacity of

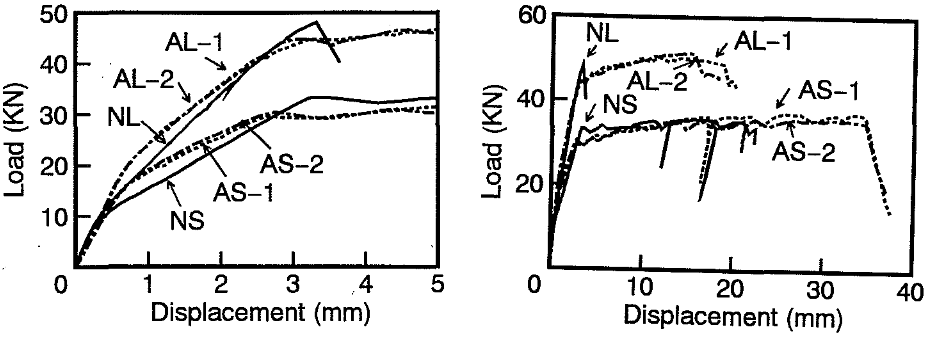


Fig. 3. Load–displacement curve (left:initial part, right:entire part)

Series A associated with the deformation after yield starting, similarly to the case of the S beams. Series A had a 6% higher ultimate load–bearing capacity and sufficiently larger deformation at the strength failure point than that of Series N, because shear failure occurred in Series N. Measured cracking moment(M_c), yield starting moment(M_y), and the maximum moment(M_u) of the beams are tabulated in *Table 1.* The depth of the neutral axis was calculated from the measured strains. The changes in the neutral axis depth are shown in *Fig. 4* with the neutral axis ratio η which is defined as the ratio of the depth of neutral axis to its effective depth. The neutral axis ratios decreased gradually when applied load increased. The ratio decreased rapidly after yield starting the bars. The ratio of Series A at the time of the yield starting were significantly higher than those of Series N. The neutral axis ratios are also given in *Table 1. Table 2* gives the various strengths and moduli of elasticity of Series A and Series N, measured at the time of the beam tests. The A–to–N ratios are also included in the table.

Table 1. Test results of beam specimens.

Specimen	Mc	Mct	Mc/Mct	My	Myt	My/Myt	Mu	Mut	Mu/Mut	η	F.P.
AS	3.61	1.72	2.10	7.55	7.38	1.02	9.14	7.98	1.15	0.524	flex.
NS	2.14	3.40	0.63	8.33	7.51	1.11	8.95	7.98	1.12	0.294	flex.
AS/NS	(1.69)			(0.91)			(1.02)			(1.78)	
AL	4.78	1.72	2.79	11.27	12.15	0.93	12.74	13.82	0.92	0.566	flex.
NL	2.38	3.40	0.70	12.05*	12.45	-	12.05*	13.82	-	0.350*	shear
AL/NL	(1.62)			(-)			(1.06)			(-)	

N.B.: moment in kN–m, η=neutral axis ratio at My, F.P.=failure pattern

Table 2. Mechanical properties of concrete.

Series	Strength (MPa)			Young's modulus (GPa)	
	Compression	Flexure	Tension	Static	Dynamic
A	40.3	3.18	2.69	17.8	20.9
N	40.6	6.29	3.01	24.9	28.0
A/N	0.99	0.50	0.90	0.72	0.74

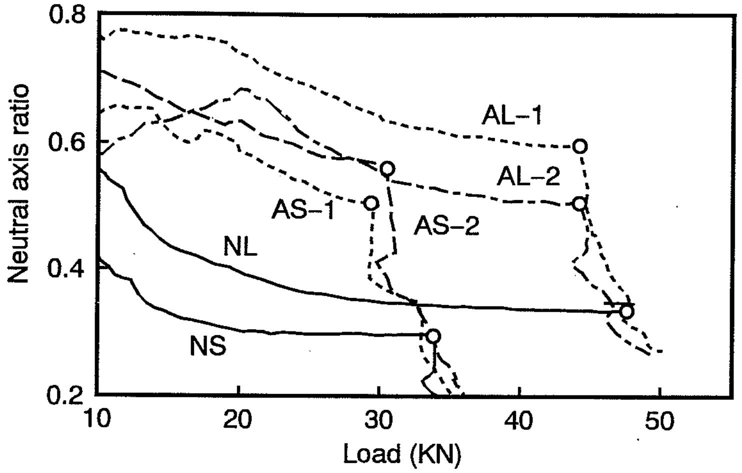


Fig. 4. Change of neutral axis ratio.

Discussions on test results

The calculated cracking bending moment (M_{ct}), yield starting bending moment (M_{yt}), and ultimate bending moment (M_{ut}) of the beams are given in *Table 1* in relation to the measured values. The values for Series A are averages of the two beams. The cracking moment, M_{ct}, was calculated by the elastic analysis of beams in which the extreme fiber stress was assumed to be the flexural strength of concrete, f_b. As for the yield starting moment, M_{yt} was calculated by assuming the strain distribution of concrete at the time of the yielding of the steel bars to be linear elastic. Here the ratio of measured Young's modulus of concrete to that of the bars (206 GPa) was adopted as the Young's modulus ratio. M_{ut} was calculated using a simple equation, which assumes the stress block of concrete to be rectangular, from the yield strength of the bars and compressive strength of concrete.

The measured M_c of the Series A beams is 2 to 3 times the calculated value in both cases. Such large differences are explained as follows: the cracking due to ASR reduced the strength of the standard test specimens, thus reducing the calculated M_{ct} values for Series A, while it increased the cracking load on the reinforced beam specimens in which the reinforcing bars restrain the ASR–induced expansion of concrete, causing chemical prestress. For this reason, the cracking bending moment cannot be related to the test results of the specimens for the strength test. As for Series N, the measured values are lower than the values calculated by assuming the extreme fiber stress to be f_b. This may be attributed to the effect of drying shrinkage as well as of the scale effect (Uchida *et al.* 1992).

Regarding the S beams, the measured M_y in Series A is close to the values calculated in consideration of E_c. For N beam, the measured one is approximately 10% higher than the calculation. Regarding the L beams, the measured values are slightly lower than the calculated value. Taking into account the fact that the apparent yield moment increases as the beam specimen size decreases (Koyanagi *et al. 1989)*, M_y decreased in case of heavily deteriorated beams. In some instances of the past loading tests, no remarkable differences were found between the yield load of ASR–damaged beams and corresponding undamaged one. The reason may be that the cracking was not so severe and they did not cause such large reductions in the neutral axis ratio as in the present testing.

In case of deteriorated beams, expansion due to ASR was arrested by reinforcing steel and the prestress was generated. During the loading test, the changing process of a neutral axis depth was measured. In case of ordinary reinforced concrete beams, the depth of neutral axis changed suddenly after cracking. In case of deteriorated beams, on the other hand, the change was small even after cracking and the presence of prestress was recognized.

After yield initiation of reinforcing steel for ASR-affected beams, the depth of neutral axis decreased rapidly because flexural cracks developed rapidly in cracking parts and the moment arm length increased. This leads to an increase in the resisting bending moment. The ultimate strengths of both series were therefore the same as in case of flexural failure.

ANALYSIS OF DEFORMATION

Method of analysis

Effects of the induced stresses caused by the restriction of ASR expansion on the bending moment at the yield starting point M_y and of load-deflection curve of the beam were examined analytically. The change of Young's modulus due to ASR was also examined. The object of the analysis was limited to S beam specimens which failed in flexure both in Series A and N.

The analysis was made in such a way that a certain amount of initial tensile pre-strain was given in reinforcing bars and corresponding prestress was in concrete. Moment-curvature relationships in a cross section were calculated with the "plane strain remaining" assumption and the stress equilibrium law. The stress-strain relationship of steel reinforcement was assumed to be elasto-plastic and that of concrete both in compression and in tension was to be linear elastic. Two kinds of Young's modulus were adopted; E_c = 25 and 18 GPa (see *Table 2*). Four levels of initial pre-strain were selected; i.e. 0, 500, 1200 and 1500 x 10^{-6}. The strain at yield point was 1850 x 10^{-6} as an average.

Result of analysis

The relationship between the yield starting moment to the initial strain in reinforcement and Young's modulus of concrete is illustrated in *Fig. 5*. M_y decreased when E_c was small, but the effect was not so remarkable. M_y decreased when the prestrain increased. But the effect of the pre-strain was so small as less than 10% when the initial strain was smaller than 60% of the yield point strain. The effect of the initial strain increased remarkably when it exceeded 80% of the yield strain, especially under smaller E_c.

Analytical load-deflection diagrams of the beam specimens under various initial strain are shown in *Fig. 6*. When the initial strain increased, the slope of the initial part of the diagram, i.e. the flexural rigidity, increased and the initial cracking load, which corresponded to the point where slope of the curve changed suddenly, also increased. This is because of the effect of the chemical prestress induced by the restriction of ASR expansion.

The change of the neutral axis ratio during loading is illustrated in *Fig. 7*. In Series N the neutral axis ratio dropped suddenly from ca. 0.5 to 0.3 at cracking. When the initial strain increased, the dropping point, i.e. the cracking load, increased, whereas the degree of dropping decreased. At the yield starting point, the neutral axis ratio increased as the initial pre-strain increased, and the yield load decreased slightly, as mentioned before. After yield started in Series A, the neutral axis ratio decreased rapidly and load increased slightly because the moment arm length increased. Finally, the load in Series A coincided with that in Series N.

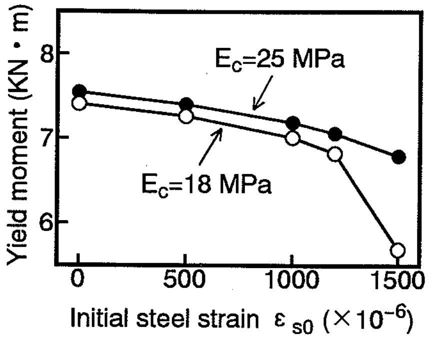


Fig. 5. Initial pre-strain vs. yield starting moment.

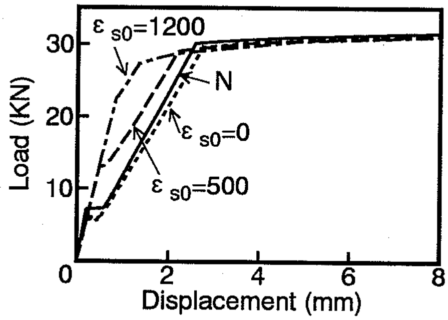


Fig. 6. Load-displacement diagram (analysis).

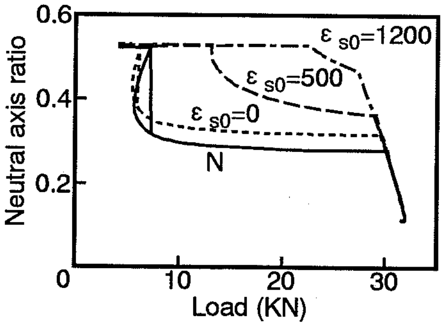


Fig. 7. Neutral axis ratio vs. load (analysis).

The reason why ASR increases the neutral axis ratio is discussed below. The property of a beam with severe cracks by ASR is considered to be shifted from that of reinforced concrete to prestressed concrete by the presence of induced stresses. In other words, tensile force on the bars causes an axial compressive force in concrete, which increases the effective cross-section, lowering the neutral axis at the time of yield starting, and M_y corresponding to the yield starting point is reduced when compared with normal N beams. This is also affected by the reduction of Young's modulus. Since the reduction in E_c of concrete on the compression side may depend on the beam size, it is not clear if the E_c values agree with those determined from the standard test specimens.

CONCLUSIONS

Loading tests were conducted on singly reinforced concrete beams without shear reinforcement which were severely deteriorated with many wide cracks by ASR. Analytical study was also made. The following conclusions were obtained:
(1) Severe ASR damage increases the neutral axis ratio at the time of yield starting when compared with undamaged beams. This reduces the arm length of the moment of the internal force, and consequently reduces the yield starting bending moment.
(2) The increase in the neutral axis ratio is not only due to the apparent reduction of Young's modulus of concrete, but also due to the presence of prestress (prestrain) caused by the ASR-induced expansion restrained by the bars.
(3) The reduction of yield starting moment is remarkable when the prestrain is quite large and the Young's modulus is low enough.
(4) After the yield starting of bars, the deformation of the bars concentrates on the cracked portions. This reduces the neutral axis ratio rapidly and increases the arm length, and thus the ultimate load-bearing capacities in the case of flexural failure are independent of the damage of ASR.
(5) Load-deflection relationships and change of neutral axis are well simulated by analysis, where the effect of the stress caused by the restriction of ASR expansion is taken into considerations.

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SOME CONSIDERATIONS ON THE EVALUATION OF POTENTIAL RESIDUAL EXPANSION OF AAR AFFECTED HYDROELECTRIC DAMS

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ABSTRACT

This paper examines recent theoretical models of the development of concrete expansion due to the alkali-aggregate reaction and some of the parameters influencing this reaction in an attempt to determine the possibility of using laboratory tests on cores drilled from hydraulic structures to estimate future expansion with a certain degree of confidence. The tests recommended are expansion tests on concrete prisms at 38°C and 100% R.H., immersion in a NaOH solution at 38°C and measurements of the concrete's pore solution alkali content. These tests and theoretical mathematical models combined with field measurements of deformation and stresses, may provide a means of predicting the development of concrete expansion. Three Hydro-Québec dams, Beauharnois, La Tuque and Témiscouata, are analyzed in this light from the point of view of the monitoring instrumentation, the concrete expansion observed there and the various tests conducted.

Keywords: Alkali-silica reaction, finite-element modeling, mathematical model

INTRODUCTION

Assessing the future expansion of concrete at a dam or other hydraulic structures affected by the alkali-aggregate reaction (AAR) calls for a theoretical as well as an experimental investigation if the following two-part questions are to be answered: 1) at any point on a dam, what will the annual expansion rate be for the next few years? and will this rate remain constant? 2) how long will this expansion last? and what will the final expansion be?

The AAR is a complex physicochemical reaction and neither the silica gel formation nor the concrete expansion mechanisms can be fully explained yet. The determining parameters are also not understood in their entirety. In order to develop an efficient mathematical model of concrete expansion, this model will have to be validated with expansion tests in the laboratory as well as field measurements.

The first questions that come to mind concern the modeling of the expansion mechanism. Which model to choose? Can a correlation be established between this model applied to samples of laboratory tested concrete made with reactive aggregate extracted from cores and the model to be developed for an expanding structure for which expansion measurements are available?

AAR EXPANSION MODELS

The AAR reaction model consists basically of three periods: initiation, development, and a rest period. During the initiation period, silica gel is produced by the reactive mineral phases. In the case of the alkali-silica reaction, the quantity of reactive material (opal, chert, flint) can be relatively small whereas in the case of the slow/late alkali-silica reaction (ASSR) the amount of reactive phase (microcrystalline and deformed quartz) is quite high.

In a dam, the onset of cracking, which indicates the end of the initiation period for the alkali-silica reaction (ASR), is generally less than 10 years compared to less than 20 years for the ASSR. The ASR can last 30 ±10 years and the ASSR, over 50 years (Charlwood et al., 1992). On the other hand, the expansion rate and silica production are lower with the ASSR. Note that the values quoted are mean values.

The concrete expansion rate in dams affected by AAR ranges from 20 to 200 microdeformations/year (με/yr) (0.002 to 0.02%/yr). The ASR yearly expansion rate is often between 100 and 200 and the ASSR, between 20 and 100 με. Based on the preceding values, the total expansion of a dam could reach 2,000 to 10,000 με (0.2 to 1%) over its lifetime.

ASSUMPTIONS UNDERLYING AAR DEVELOPMENT

Hobbs (1993) proposes a reaction model in which the volume of gel produced before the development period is controlled by the chemical reaction between the hydroxide ions and the reactive silica (assuming that there is sufficient water in the concrete to maintain the reaction). His mathematical model based on tests on concrete at 100% R.H. at 38°C, 20°C and under outdoor exposure conditions produced two important hypotheses: 1) long-term expansion decreases with longer initiation times; 2) the duration of the initiation time is proportional to the temperature and the exposure condition in a ratio of 1, 4 and 7 for the samples tested at the above exposure conditions respectively.

This study suggests that it should be possible to correlate the expansion of samples tested in laboratory at 38°C with the expansion of in-service structures.

Furusawa et al. (1994) also tried to model the expansion of mortar samples mathematically, based on the ASTM C277 mortar prism test. Their model assumes that the main reaction mechanism depends on the hydroxide and alkaline ion diffusion rate in the reactive aggregate. The reaction development period does not begin until a porous zone surrounding the aggregate, a zone which depends on the specific surface area of the aggregate, is filled with the reaction product (gel). Interestingly, their model suggests that the reaction initiation period depends on the diffusion coefficient of the aggregate and that the final expansion of the mortar mixes is not proportional to the duration of the initiation period but depends, among other things, on the values of these coefficients. This final expansion seems to reach the same limit value for a certain type of aggregate and for the same initial quantity of alkalis. If the diffusion coefficient of the aggregate is reduced, the same final expansion is attained but it takes longer.

The reaction rest period depends on the depletion of one of the three basic reactive components, namely the reactive silica, the alkalis or the water.

On the basis of the models described above, it appears to the authors of this paper that, from a practical point of view, it is important to determine whether in fact the concrete expansion rate of a dam can be related to the final expansion. If so, it should be possible to estimate the final expansion from the mean expansion rate.

It seems idealistic, a priori, to hope that such a simple relation between the expansion rate and the final expansion could be developed, considering all the many factors influencing the reaction. But, if all these factors together with their effect on the reaction are known, there should be a means of incorporating them into a mathematical model relating the expansion of structures to that of laboratory-tested samples.

FACTORS INFLUENCING THE AAR

The major factors influencing the AAR are listed below together with the way each is handled in order to integrate it into a mathematical model:

- Quantity of reactive silica: this value can be evaluated by expansion tests or other suitable methods (petrography, SEM, etc.)
- Quantity of alkalis in the concrete that come from the cement and, possibly, from the aggregate and the environment. In the case of dams, it is sometimes possible to assess the quantity of original alkalis if the dam construction date is known and the amount of cement can be determined by microscope analysis. The amount of alkalis contributed by the aggregate could be assessed by alkali extraction methods. For instance, Grattan-Bellew (1994) estimated that at the Sanders Generating Station (Ontario, Canada), the limestone aggregate contributed to the alkali content of the concrete pore solution by 1.2 kg/m^3.
- Confinement pressure due to the weight of the dam and to the reinforcing effect and pressure generated by the AAR. Generally, the confinement pressure in a gravity dam is such that the longitudinal expansion is less than the lateral expansion, which in turn is less than the vertical expansion. The latter is in fact similar to free expansion because it is due only to the weight of the concrete in the dam. Rogers et al. (1994) observed that the confinement pressure needed to prevent concrete expansion is around 4.1 MPa. Stark et al. (1993) performed tests on triaxially confined concrete cylinders and, on the basis of their findings, suggested that 2.07 MPa suffice to stop concrete expansion. Durand et al. (1992) performed laboratory measurements of the pressures generated by the AAR and found values of up to 7 MPa. The internal stresses caused by concrete expansion will increase over time. Charlwood et al. (1992) have modeled the expansion of concrete as a function of the variations in stress with time and found that the variation of the concrete expansion with the log of the stress offers a strong correlation with field observations.
- Adequate amount of water to sustain the reaction. In a dam, it is assumed that, for all practical purposes, there is sufficient water in the concrete for the AAR to take place. The water can come from that excess of the cement hydration water (in a mass concrete the W/C ratio is high) and from infiltrations.
- The geometric configuration of the structure influences the state of the internal stresses and expansion. Geometric discontinuities will increase expansion at the junctions. Finite-element modeling in these specific cases is an essential tool for monitoring developments (Charlwood et al., 1992, and Léger et al., 1995). For example, in the event of a saw cut in a dam, it is becoming standard practice to use finite-element analysis.
- Concrete quality and the physical/mechanical properties of the structure. Microcracks associated with the thermal shrinkage of concrete, the construction joints, and construction defects contribute to the nonhomogeneity of the structure, which makes it more difficult to predict how expansion will develop at any specific location on the dam.
- Thermal shrinkage of the concrete will conteract the expansion due to the AAR in dams.
- Temperature. The temperature distribution inside a dam can be estimated by analyzing the heat transfer. Léger et al. (1995) propose the use of a monthly temperature distribution during the year and a weighted average of these 12 values to obtain an average for the entire year.

It is obvious from the foregoing considerations that finite-element analysis using the factors given above represents the best method that has been developed so far for modeling the distribution of stresses and deformations in a dam. Léger et al. (1995), for example, have already developed a finite-element analysis model of the behavior of a dam which takes account of confinement, temperature, humidity and aggregate reactivity.

The expansions and deformations estimated by the models currently in use will be compared to those measured on dams in service. In this way, an expansion monitoring program in the field combined with a suitable model will allow real-time follow up of the dam. However, it will not offer a short-term estimate of future variations in the concrete expansion of the dam, for which we suggest that different tests be performed on cores. The most suitable tests seem to be expansion tests on cores and on a concrete made with aggregates extracted from drilling cores and, also, measurements of the alkali concentration in the concrete's pore solution.

LABORATORY TESTS ON CORES FROM A DAM

Effects of drilling on the core samples

The first effect to be considered is that of the drilling operation on the concrete of the samples. In addition to the fact that the sample will be broken into several pieces, the action of the drill introduces cracks in the pieces of the cores to be used for the tests. As soon as the samples are removed, the stresses will be relieved and the sample will expand which may further crack the concrete in the sample. Another important point to consider is the handling of the cores. If they are not sealed immediately to prevent humidity loss, the drying action can fix part of the hydroxide alkalis in the pore solution. According to Stark et al., 1993, those alkalis will no longer be available when rehydration occurs later. Their laboratory tests suggested that carbonation was somehow involved in the effect.

The most appropriate tests to be conducted on the concrete core samples seem to be the expansion tests at 38°C and 100% R.H., those at 38°C in a 1M NaOH solution, and measurements of the concrete's pore solution alkali content.

Test at 38°C and 100% R.H. (CSA A23.2-14A, concrete prism expansion) (CSA, 1994) on cores

It is suggested that the test be performed at a temperature of 38°C so that expansion results will be obtained within two years. Although, expansion tests may be performed at different temperatures within the range that the structure may experience during its lifetime, the present authors consider that 38°C is reasonable. The only information missing is the factor linking the expansion rate at this temperature to the in situ dam concrete expansion.

The mass and length of each sample are measured periodically until each reaches equilibrium, which may take a few days if the samples are kept damp and anywhere between 2 and 20 weeks if kept in the laboratory in a dry environment. Equilibrium is considered reached when the expansion rate starts to change and, also, when the mass of the sample becomes more stable (Bérubé et al., 1993). Once equilibrium is attained, the time counter is reset to zero and the expansion is measured at regular intervals.

On the basis of a long series of tests, Bérubé et al. (1994) established limit values for the potential expansion. At one year, an expansion of 0.015% is considered being on the average while a value of 0.02% and over indicates some high expansion potential.

Expansion test on concretes immersed in a 1M NaOH solution at 38°C

The aim of this test is to determine whether the concrete of an in-service structure is showing any signs of expansion in an environment where there is a suitable quantity of available alkalis. In a way, it represents a measurement of the amount of reactive silica still present in the concrete.

The method is identical to that of accelerated test CSA A23.2-25A (Canadian Standards Association, 1994) which consists of immersing concrete samples in an alkaline solution of 1M NaOH. However, the temperature of this solution is set at 38°C instead of the 80°C specified for the CSA test because the authors, like Bérubé et al. (1993) consider that it is more realistic to test at a lower temperature for a longer period of time. Shayan et al. (1988) have shown that testing concrete in 1M NaOH at 80°C produces erratic results too.

Bérubé et al. (1994) suggest some expansion limits at one year on samples of in-service concrete to evaluate their reactivity. They offer four possible explanations in the case of low expansion : 1) the aggregate is not reactive and the concrete expansion is due to other mechanisms 2) the aggregate was, but no longer is, reactive because the reactive phases have been consumed 3) the concrete was so cracked that the reaction product fills the cracks 4) the concrete is so impervious that the alkaline solution can barely penetrate it. The present authors also believe that, if the reaction products are sufficiently fluid — or conversely, the concrete is permeable enough — they may pass into the immersion solution.

It is clear that interpretation of the results on the basis of just one test may be complex and should really be based on judgment and confirmed by other tests. For example, if aggregate is extracted from drilled cores and the two tests just described are repeated (100% R.H. and immersion), we would have confidence in determining whether or not it contained reactive phases. These two tests and the previous one with concrete cores in an immersed solution of 1M NaOH at 38°C should show about the same expansions because concrete made with extracted aggregates will have a high alkali content of 5.3 kg/m^3 (CSA A232-14A-1994).

Alkali concentration measurement of the pore solution of concrete cores

Grattan-Bellew (1994) and other researchers have observed that the extent of concrete degradation (cracking and expansion) due to the AAR depends, among other things, on the amount of alkalis in the concrete's pore solution. Below a certain minimum quantity of alkalis, the reaction cannot continue. Since the AAR consumes alkalis, measurement of the alkali concentration in the pore solution at different times could reveal whether any variation occurs in the reaction rate with time and, ultimately, if there is any deceleration in the reaction (if the alkali concentration decreases).

The most promising method developed so far is to extract the pore solution from a concrete sample using a high pressure steel die apparatus. Barneyback and Diamond (1981) further developed the Longuet method (Longuet et al., 1973) and have used it on various samples of cement and mortar pastes. If the high-pressure method is not readily available, a useful alternative is the hot-water alkali extraction method developed by the Ontario Ministry of Transport (Rogers and Hooton, 1989). Bérubé et al. (1994) suggest a number of criteria that could be applied to assess the concrete's expansion potential according to its alkali content by the hot-water leaching method. According to those criteria, an amount of over 3.0 kg/m^3 Na_2O eq. shows a high expansion potential.

It should be remembered, as pointed out by Bérubé et al. (1994), that the alkali content measured by the hot-water leaching method is liable to be an overestimate if earlier formed silica gels are also dissolved.

The foregoing remarks indicate that measurements of concrete expansion and alkali content performed on core samples provide a useful means of obtaining an impressive volume of data which, as far as we know, remains the only short-term way of predicting variations in the behavior of a dam.

FIELD MONITORING OF CONCRETE DEFORMATION IN A DAM

In any program of field monitoring of concrete deformation in a hydraulic structure, the choice of measuring instruments and their location are two major factors to be considered if the dam expansion due to the AAR is to be modeled correctly. Each instrument will be selected on the basis of its cost, usefulness, accuracy and long-term performance. The location of the measuring instruments is dictated by two different needs, namely, to establish a definition of the dam behavior as a whole and, also, to asses the behavior of the dam at some precise location (e.g. geometric discontinuity) where the accumulation of deformations and stresses may disturb the operation of mechanical parts or even jeopardize the structural integrity of the dam itself. Furthermore, new components can be added to an existing measuring system should major repair works such as dam cuts be undertaken.

Three case studies of Hydro-Québec dams will now be used to illustrate the approach the utility has adopted for assessing the residual expansion of concrete based on laboratory tests and field measurements of dam deformation.

HYDRO-QUÉBEC CASE STUDIES

Beauharnois dam

The Beauharnois dam is located at the end of the Beauharnois canal 40 km southwest of Montreal, Québec. It is 865 m long and comprises three power plants with a total of 37 generating units with one concrete gravity dam to the right and one to the left. The structures were built in three stages over the years from 1928 to 1961. The aggregate is reactive Potsdam sandstone excavated from the canal.

Measurements of the vertical displacements showed 60, 27 and 53 με/yr for the gravity dams, power plants and water intakes respectively (Gocevski and Rivest, 1993). In addition, all three plants are shifting slowly upstream, plants #1 and #2 at a rate of 1 mm/year, and plant #3 at a rate of 0.5 mm/year. Tests at 38°C and 100% R.H. on cores revealed an expansion rate of 77 με/yr.

The instrumentation at this dam comprises 16 pendulums, two in the gravity dam on the right-hand bank, four in the intakes and ten in the power plants, together with various monitoring benchmarks. Also ten instrumented cylinders from Université de Sherbrooke were installed, three two-dimensional units in 1987 and seven three-dimensional units in 1992. The former were interpreted by Bois (1994) who reported that the instruments at plant #1 indicated a compression of 2 MPa in the longitudinal axis and 4 MPa in the vertical direction since they were installed at the dam. The two cylinders at plant #3 show an isotropic compression of 3 MPa since their installation.

La Tuque dam

Built in 1938-40, La Tuque dam is located on the Saint-Maurice River, 405 km northeast of Montréal. It comprises two gravity dams, one on each bank, a spillway and an intake abutting the plant. The total length is 424.5 m and the maximum height, 53.3 m.

The concrete is affected by the ASSR. The coarse aggregate is composed of granite and granitic gneiss which is slightly reactive to the alkalis in the cement.

The dam is fitted with various instruments, including a topographic monitoring system with 19 measuring points and three inverted pendulums, jointmeters and boreholes extensometers (Buffex type) near the saw cut made in the dam in 1993.

The monitoring data show that over the last ten years the structures have been slowly moving upstream, except at the junction between the water intake and the gravity dam on the left-hand bank, which are at an angle of 141° to each other, where the displacement is downstream. In both cases, the maximum displacements are around 1.3 mm/year. Uplift is approximately 1 mm/yr for the dams and the water intake, and about 2 mm/yr at the joint, which represents 33 and 66 με/yr respectively (Veilleux, 1992).

Three instrumented cylinders were installed in 1995-96, namely two near the cut, at the junction between the intake and the gravity dam on the left bank, and one in the gravity dam on the right bank. The core samples collected are currently used for potential-expansion tests, as discussed earlier.

Témiscouata dam

Témiscouata dam is a retaining structure built on the Madawaska River in 1930 about 100 km south of Rivière-Du-Loup, Québec. It was in 1952 that workers first observed that the apron slab had expanded. In 1993 and 1994, the deterioration of the AAR affected dam was such that it was demolished and rebuilt. The coarse aggregate used in the new concrete mix was considered to be slightly reactive but it was employed nevertheless because it was the only one available. To improve the concrete quality and prevent the risk of future expansion, 7.5% silica fume was added to the mix.

Samples of the concrete were taken from the new dam at the end of 1993 and expansion tests at 38°C and 100% R.H. are now under way to determine whether expansion will occur even with the use of silica fume. Also, upstream of the dam, on the right-hand bank, two test columns have been poured using different types of concrete. The first mixture is identical to the one used for the original dam, the second is that used for the new dam but without any silica fume. An instrumented cylinder has been installed in each column. The columns are exposed to the same conditions as the dam. Samples of each concrete have been taken and tests at 38°C and 100% R.H. are currently being performed so that expansions measured in the laboratory can be compared to those recorded in the field.

CONCLUSION

Theoretical mathematical models for the development of the alkali-aggregate reaction, such as those of Hobbs and Furusawa, will become more and more essential if we are ever to correlate the expansion of samples, including cores subjected to laboratory tests, with real expansion in a structure affected by AAR.

In the last few years, several dams have been the subject of a drilling program to determine the physical state of the concrete together with microscope studies to identify the extent or presence of the AAR. This work was completely separate from other drilling operations done to install instruments, drains or other items. In the authors' opinion, it would be wise to use cores from all drilling operations for the expansion tests and measurements of the alkali content. The expansion tests most strongly recommended are the 38°C and 100% R.H., the immersion test in alkaline solution at 38°C, and measurement of the alkali concentration of the concrete's pore solution.

The last point concerns the field measurements of concrete expansion in dams. Knowing these, it is then possible to correlate between free expansion determined in the laboratory tests and real expansion in the field. If eventually, a finite-element-based numerical simulation of the dam behavior is required, the information gathered— laboratory and field expansions, alkali content in the pore solution— could be used to complete the model.

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INFLUENCE OF ALKALI-AGGREGATE REACTION ON FLEXURAL PROPERTIES OF STEEL FIBER REINFORCED CONCRETE

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ABSTRACT

In accelerated tests of 3 days at 100°C, with silica glass aggregate, alkali-aggregate reaction (AAR) causes a reduction of both flexural strength and compressive strength, and an increase in the flexural toughness in both plain concrete (PC) and steel fibre reinforced concrete (SFRC). The steel fibre in SFRC can reduce AAR expansion to a certain extent and thus its effect on the flexural strength of the concrete, when the AAR expansion is in the low range. In SFRC the thresholds of alkali content and the amount of reactive aggregate can be slightly increased with respect to that of PC.

Key words: alkali-aggregate reaction, steel fibre reinforced concrete, strength

INTRODUCTION

Alkali aggregate reaction results in an expansion of the concrete. Steel fibres in concrete may suppress this kind of expansion to a certain extent. This paper discusses whether the thresholds of alkali content and the amount of reactive aggregate can be increased and whether the attack of AAR on the properties of reinforced concrete can be reduced by the use of steel fibre.

MATERIALS

The cement used in this experiment is manufactured by the Jiang Nan Cement Group, Nanjing, China. Its mineral composition, chemical composition and physical properties are listed in Tables 1, 2 and 3 respectively.

Table 1 Mineral Composition of Cement

Mineral Composition	C_3S	C_2S	C_3A	C_4AF	Free-CaO
%,by weight	55.21	21.59	4.54	16.29	0.29

Table 2 Chemical Composition of Cement

Composition	SiO_2	Al_2O_3	CaO	MgO	SO_3	Fe_2O_3	Na_2Oequiv.
%,by weight	22.06	5.13	65.37	1.06	2.03	5.36	0.40

Table 3 Physical Properties of Cement

Fineness(%,by weight)		Flexural Strength(MPa)			Compressive Strength(MPa)			Specific Area
80µm Sieve Residue	20µm Sieve Residue	1d	3d	28d	1d	3d	28d	
0.06	2.64	4	6.7	9.5	18	40.5	68.9	3560cm^2 /g

The steel fibre being used was a straight type of fibre with a square cross section and was 35 mm in length. Its length to equivalent diameter ratio was 44. The coarse aggregate was crushed stone with a range in size of 5mm -- 20mm and was inert. The fineness module of the sand was 2.7. Artificial silica glass with a particle size range of 0.16mm--5mm was used as the reactive aggregate. Potassium hydroxide (KOH) was used to increase the alkali content of the concrete, which was 1.94 kg/m^3 .

EXPERIMENTAL METHODS

Cross Tests

Firstly, cross tests were designed to determine the proper amount of steel fibre to be used from the point of economics and its effectiveness in suppressing AAR expansion. Three factors were evaluated in the cross tests:

(a) volume percentage of steel fibre - V_f : 0, 0.5, 1.0, 1.5%
(b) the additional alkali content - AL: 0, 4, 6, 8 kg/m^3
(c) the amount of reactive silica glass- RA: 0, 15, 25, 35 kg/m^3 .

It was found that steel fibres with a volume percentage of 1.0% will be sufficient.

Mix Proportions

The amount of the main materials in the concrete specimens being used for flexural tests are shown in Table 4. The water cement ratio was 0.42. The specimens are divided into two groups. The first group did not contain steel fibre (Specimens P1 to P5). The second group (Specimens S1 to S5) contained 1.0% steel fibre. The range of additional alkali (Na_2O equiv.) was 0 to 8 kg/m^3 of concrete. The reactive silica glass ranged from 0 to 32 kg/m^3 (Table 5).

Table 4 Amount of Main Materials Used in Concrete Specimens

Cement(kg/m^3)	Coarse Aggregate(kg/m^3)	Sand(kg/m^3)	Water(kg/m^3)
486	1004	669	204

Table 5 Volume Percentage of Steel Fibre, Alkali Content and Amount of Silica Glass in Concrete

Specimens	*P1*	*P2*	*P3*	*P4*	*P5*	*S1*	*S2*	*S3*	*S4*	*S5*
Volume of Fibre (Vf,%)	0	0	0	0	0	1.0	1.0	1.0	1.0	1.0
Alkali Content (kg/m^3)	0	2	4	6	8	0	2	4	6	8
Silica Glass (kg/m^3)	0	8	16	24	32	0	8	16	24	32

Expansion Test and Flexural Test

The size of the specimens used for the expansion and flexural tests were 100×100×500mm. After being cast and demoulded, the specimens were cured at 20±3°C, 90% R.H. for 7 days, and then the initial length (Lo) was measured. They were then put in a curing box and cured at 100°C water vapour for 72 hours. After the box was turned off and the specimens were cooled to room temperature in the box, they were withdrawn from the box at a day before their final length(Lt) was measured. The expansion ratio of the specimens was calculated as follows:

$$\varepsilon = \frac{L_t - L_o}{L_b} \times 100\%$$

in which, ε: expansion ratio(%); L_t: the length of concrete specimens after being cured; L_o: the initial length of specimens; L_b: the gauging distance.

After the concrete specimens were measured, flexural tests were carried out. The flexural strength of the concrete was calculated using the following formula:

$$\sigma = \frac{PL}{bh^2}$$

in which, σ: flexural strength; P: ultimate flexural load; L: span distance, 300 mm; b: width of specimens, 100mm; h: height of specimens, 100mm.

The flexural toughness of concrete is expressed by the flexural toughness index, σ_b as stated by the Japan Concrete Institute.

$$\sigma_b = \frac{TL}{\delta_b h^2}$$

in which, δ, the deflection, equals L/150, in this case it is equal to 2mm; T equals the area under the load-deflection curve at the point of δ.

Ultrasonic Test

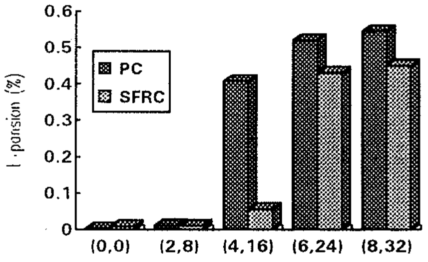
In order to detect the microfracture of the concrete which was due to the alkali-aggreagte reation, an ultrasonic test was carried out on the specimens prior to the flexural tests being carried out. Emission and receiving detectors were fixed on the two ends of the specimens. The transmission distance of the ultrasonic wave was 500mm, which was equal to the length of the specimens.

RESULTS AND DISCUSSION

The expansion ratio(ε), the velocity of ultrasonic wave transmission through the concrete specimen(v), the ultimate flexural strength and the compressive strength of the various specimens are shown in Table 6 and Figs.1--4.

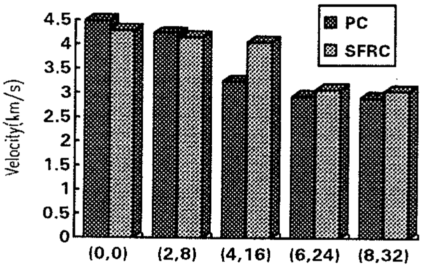
Table 6 Main Test Results

Specimen	*P1*	*P2*	*P3*	*P4*	*P5*	*S1*	*S2*	*S3*	*S4*	*S5*
Expansion(%)	0.003	0.012	0.407	0.520	0.545	0.009	0.011	0.056	0.430	0.450
Velocity(km/s)	4.49	4.24	3.25	2.94	2.90	4.29	4.16	4.06	3.08	3.04
Flexural Strength(MPa)	8.82	7.38	3.06	2.88	2.79	11.62	9.42	8.57	5.38	4.99
Compressive Strength(MPa)	57.0	51.8	44.0	31.5	24.8	68.3	58.5	51.8	34.5	27.8



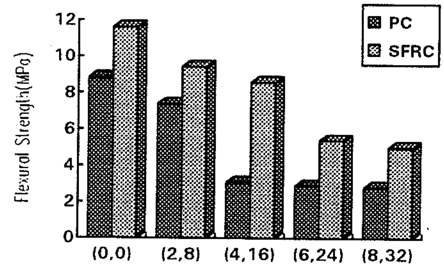
(Alkali Content, Amount of Silica Glass, kg/m^3)

***Fig.1** Expansion Ratio vs Alkali Content and the Amount of Silica Glass in Plain Concrete(PC) and Steel Fibre Reinforced Concrete(SFRC)*



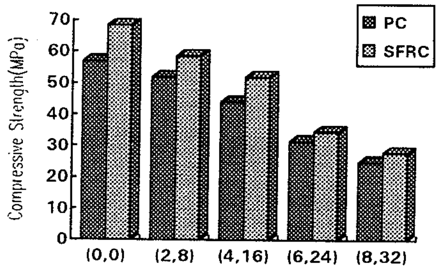
(Alkali Content, Amount of Silica Glass, kg/m^3)

Fig.2 Velocity of Ultrasonic Wave Transmitting Through Concrete vs Alkali Content and Amount of Reactive Silica Glass in PC and SFRC



(Alkali Content, Amount of Silica Glass, kg/m^3)

Fig.3 Influence of AAR on Flexural Strength of PC and SFRC



(Alkali Content, Amount of Silica Glass, kg/m^3)

Fig.4 Influence of AAR on Compressive Strength of PC and SFRC

From Fig.1, it can be seen that the expansion ratio of the SFRC is lower than that of the PC at the same alkali content and at the same amount of reactive silica glass. In other words, the thresholds of the alkali content and the amount of reactive aggregate in SFRC is able to be increased compared to that of PC . This is due to the restriction on AAR expansion as a result of the steel fibre. For example, if the expansion ratio of the SFRC is specified as 0.05%, the alkali content can be increased by about 1.8 kg/m^3 and the amount of silica glass can be increased by 8 kg/m^3 when compared to that of PC.

These results can also be confirmed by the ultrasonic wave test. The velocity of ultrasonic wave transmission through plain concrete specimens is lower than that through steel fibre reinforced concrete while the alkali content and the amount of silica glass in SFRC is the same as that in PC (Refer to Fig.2). This shows that the microfracture caused by AAR in the SFRC is less than that in the PC and it is the the steel fibre which reduces the AAR attack.

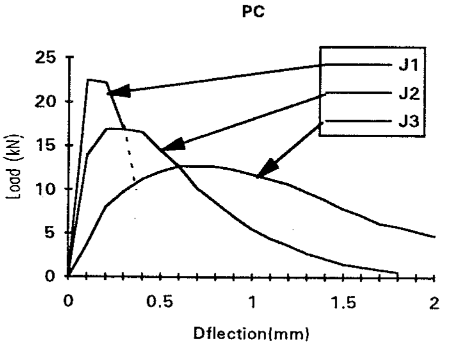
Alkali silica reaction causes a dramatic loss of flexural strength and a reduction of compressive strength both in SFRC and PC (Fig.3 and Fig.4). When alkali content and silica glass are at 4 kg/m^3 and 16 kg/m^3 respectively , the expansion of the plain concrete reaches 0.407% and its flexural strength is reduced from 8.82 MPa to 3.06 MPa, while its compressive strength is decreased from 57.0 MPa to 44.0 MPa. Under the same conditions, the expansion of steel fibre reinforced concrete was only 0.056%. Its flexural strength and compressive strength are decreased from 11.62 MPa to 8.57 MPa and from 68.3 MPa to 51.8 MPa respectively, this loss of flextural strength in SFRC is far less than that in the PC.

Alkali-silica reaction also influences the shape of the load-deflection curve of both plain concrete and steel fibre reinforced concrete. The curve of the concrete without AAR is steep and narrow, this is changed to flat and broad when AAR exists (Refer to Fig.5 and Fig.6). Alkali silica reaction results in an increase of the flexural toughness index. For example, the flexural toughness index of plain concrete increases from 8.93 to 16.25 when AAR expansion reaches 0.426% (Fig.5 Curve J3), and the flexural toughness index of the SFRC with steel fibres 0.5% improves from 9.25 to 21.26, while expansion ratio reaches 0.50% (Fig.6 curve J6). These changes can be attributed to more original microcracks caused by the AAR. When the concrete is loaded, the cracking pattern of concrete changes from a few points of propagation to many points of propagation and as a result the concrete is able to absorb more energy before it is damaged.

The principle of steel fiber on AAR expansion is similar to its effect on creep, drying shrinkage and deformation of concrete under loading. Some researches[1-3] have shown that steel fibers restrained the creep of cement matrixes and reduced the free shrinkage of concrete. The deflection of reinforced concrete beams was reduced and the stiffness after cracking was enhanced because of the addition of steel fibers[4-5] .

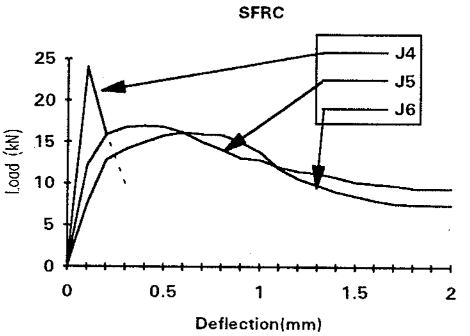
Koyanagi etc[6] also indicated that addition of steel fiber made the AAR crack length of reinforced concrete beams shorter with respect to plain concrete beams.

Steel fibers restrain the physical expansive process of AAR other than the chemical reaction of AAR.



	AL, kg/m³	RA, kg/m³	ε, %	V, km/s	δb
J1	0	0	0.010	4.40	8.93
J2	6	25	0.311	3.22	8.86
J3	8	35	0.426	3.26	16.25

Fig. 5 Influence of AAR on Load-Deflection Curve of Plain Concrete



	AL, kg/m³	RA, kg/m³	ε, %	V, km/s	δb
J4	0	15	0.021	4.07	9.26
J5	6	25	0.325	3.04	23.74
J6	8	35	0.467	2.97	21.26

Fig.6 Influence of AAR on Load-Deflection Curve of SFRC (Vf=0.5%)

CONCLUSIONS

Steel fibre in concrete can restrict AAR expansion at low expansions. The thresholds of alkali content and the amount of reactive aggregate can also be increased slightly when compared to those of plain concrete.

Alkali silica reaction causes a reduction of flexural strength and compressive strength both in plain concrete and steel fibre reinforced concrete. It also substantially changes the pattern of the load-deflection curve. The flexural toughness index of concrete is improved when the alkali silica reaction exists.

Future research could determine if deformed, hooked, crimped, paddled or enlarged ends steel fibres may be better than straight fibres in restricting the expansion of the alkali-silica reaction. Steel fiber may be useful in concrete structural practice in reducing sensitivity to small expansions.

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Preventive Effects

of

Mineral Admixtures

on

Alkali-Aggregate Reaction

EVOLUTION OF CHEMICAL AND PHYSICAL PARAMETERS OF BLENDED CEMENT MORTARS SUBJECTED TO THE NaOH BATH TEST

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ABSTRACT

Mortar bars incorporating two different alkali-reactive aggregates and containing various amounts of two different supplementary cementing materials (SCMs) were subjected to the NaOH bath test. Measurements of expansion, water permeability, pore liquid composition, and $Ca(OH)_2$ content were performed periodically on the specimens. Expansion data showed the ability of SCMs (condensed silica fume and pulverised fuel ash) to reduce mortar bar expansion. Furthermore, the SCMs were able to hydrate in a short time so that they were capable of reducing the OH^- ion concentration in the pore solution within the mortar specimens as a result of a reduced permeability of mortar and an incorporation of alkalies by pozzolanic reaction products. These results prove the suitability of the NaOH bath test method as a means for evaluating the effectiveness of SCMs against alkali-silica reaction.
Keywords: NaOH bath test, pozzolanic reaction, supplementary cementing materials

INTRODUCTION

Since 1986, the accelerated mortar-bar expansion test in a 1M NaOH solution at 80°C, also formerly known as South African NBRI test (Oberholster & Davies, 1986; Davies & Oberholster, 1987a), has largely been used for predicting the potential alkali-reactivity of concrete aggregates. Published work has also shown the good potential of this test for evaluating the effectiveness of supplementary cementing materials (SCMs) against alkali-silica reaction (ASR) (Davies & Oberholster, 1987b; Bérubé & Duchesne, 1992a; Shayan, 1992).

However, in the ASTM version of the NaOH bath test (Test Method C1260-94), no mention is made about the use of this test for assessing the performance of SCMs against ASR, probably because it was thought unlikely that, after the short (two days) curing period prior to immersion in the hot sodium hydroxide solution, the SCMs have sufficiently hydrated to become so impermeable as to prevent NaOH from penetrating the small cross section of the mortar bars (Hooton & Rogers, 1992).

This hypothesis did not appear to be supported by the results of more recent studies dealing with the effect of SCMs on the pore liquid composition within alkali-reactive mortar specimens immersed in NaOH solution (Berra *et al.*, 1994a; Bérubé *et al.*, 1995).

The aim of this paper is to search throughly into this subject. The behaviour of two types of SCMs, one condensed silica fume (CSF) and one pulverised fuel ash (PFA), is discussed with respect to the time evolution of the expansion, water permeability, portlandite content, and pore liquid composition in mortar bars made with siliceous aggregates of high or moderate alkali-reactivity when subjected to the NaOH bath test.

MATERIALS AND METHODS

Table 1 gives the chemical and physical characteristics of the Portland cement (PC) and SCMs used in this study.

In addition to the control (0% SCM), various SCM contents were used as replacement by mass of cement: 5%, 10%, 15% and 20% for CSF, 5%, 15%, 25% and 35% for PFA.

The effectiveness of the two SCMs were tested in the presence of a natural italian siliceous aggregate (designated NSA) of known moderate reactivity in service or an artificial very reactive siliceous aggregate such as fused quartz (FQ) (density = 2.20 g/cm^3; SiO_2 content = 99.2%; alkali content = 0.01% Na_2O equivalent). The natural aggregate consisted of rhyolite rocks with large amounts of strained quartz as the main reactive component and was used in the standard gradation (0.15-4.75 mm) as specified in ASTM C1260-94. The fused quartz was used in the 150-300 μm size fraction and was combined with a natural non-reactive quartzitic sand (density = 2.64 g/cm^3; water absorption = 1.0%) having the ASTM gradation. Fused quartz replaced the corresponding fraction of natural sand by an amount equal to 4% by mass of the total aggregate (Berra *et al.*, 1994b).

Table 1 Chemical and physical characteristics of Portland cement and supplementary cementing materials used.

Oxide (%)	Cement	PFA	CSF	Oxide (%)	Cement	PFA	CSF
SiO_2	22.11	56.02	96.20	Na_2O equivalent			
Fe_2O_3	4.76	7.45	0.40	Total	0.43	1.96	0.41
Al_2O_3	3.62	29.14	0.35	Available	0.29	0.43	0.13
CaO	65.93	0.04	0.43	Water-Soluble	0.13	0.03	0.13
Free CaO	0.14			L.O.I. (%)	0.72	3.03	0.98
MgO	0.57	0.40	0.20	Density (kg/m^3)	3160	2330	2200
SO_3	1.51	0.40		Surface Area, Blaine (m^2/kg)	306	446	25,000**
Mn_2O_3	0.04	0.07		Autoclave Expansion (%)	-0.01		
TiO_2	0.08	0.30		Bogue Compounds (%)			
P_2O_5	0.11	0.15		C_3S	64.67		
SrO	0.04			C_2S	14.67		
Na_2O				C_3A	1.56		
Total	0.11	0.73	0.18	C_4AF	14.47		
Available	0.08	0.14	0.04				
Water-Soluble	0.02	0.03	0.04				
K_2O							
Total	0.49	1.87	0.35				
Available	0.32	0.44	0.14				
Water-Soluble	0.16	0.003	0.14				

**BET method

The mortar bars (water/cement weight ratio = 0.47; aggregate/cement weight ratio = 2.25; bar dimensions = 25x25x285 mm) were cast and stored in accordance with the ASTM C1260-94 test procedure.

The length of the bars was periodically measured using a comparator with a sensitivity of 0.001 mm.

The OH^- ion concentration in the pore solution and the portlandite content within the cementitious matrix were determined after the 1-day storage of the bars in the water bath at 80°C and during their period of immersion in a 1M NaOH solution at 80°C.

The OH^- ion concentration was measured by a pH-meter with a Radiometer type G202 A electrode and a calomel reference electrode (Dent Glasser & Kataoka, 1981) on the pore solution expressed from each bar through the use of a high pressure apparatus (550 MPa).

The portlandite content was determined by thermogravimetric analysis (30-950°C at 10°C/min under N_2 atmosphere) on powdered, dried (through acetone and ethyl ether treatment) mortar samples obtained from a portion of each bar.

For the permeability measurements, cylindrical specimens (38 mm in diameter and 76 mm in height) were used instead of prismatic ones. However, the storage conditions were the same.

Water permeability was measured at room temperature (about 20°C) using de-aired water and a tri-axial permeability cell in which driving and confining pressures were 0.67 and 1.34 MPa, respectively. Before testing, each cylindrical specimen was vacuum saturated using a standard test procedure (AASHTO T227-83, 1988). The permeability coefficient, K, was calculated according to Darcy's law.

TEST RESULTS AND DISCUSSION

Expansion

Figures 1a and 1b show the effect of replacing various amounts of Portland cement by PFA or CSF on the expansion of mortar bars incorporating FQ or NSA aggregate at varying times of immersion in the NaOH bath.

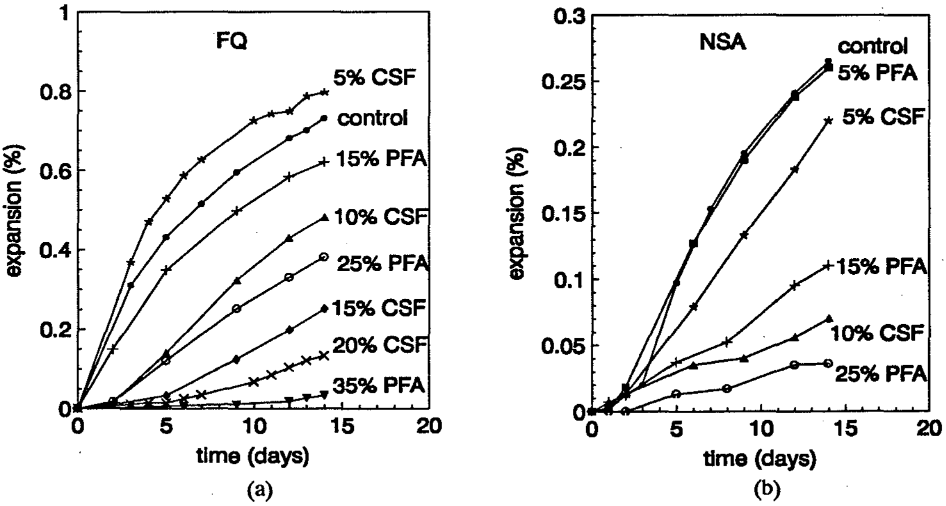


Fig. 1 Effect of replacing various amounts of Portland cement by PFA or CSF on the expansion of mortar bars incorporating FQ or NSA aggregate.

As expected, in the absence of SCMs (control specimens), mortar bars incorporating either FQ or NSA showed 14-day expansions above 0.10%, that is the value reported in the Appendix of ASTM C1260 as indicative of innocuous behaviour of aggregates. SCM addition always reduced the expansion of mortar bars except for the specimens containing aggregate FQ and 5% CSF, which showed larger expansion than the control (pessimum effect). However, in most cases 14-day expansion was still above 0.10%.

The effect of SCM content on the 14-day expansion of mortar bars is shown in Figure 2.

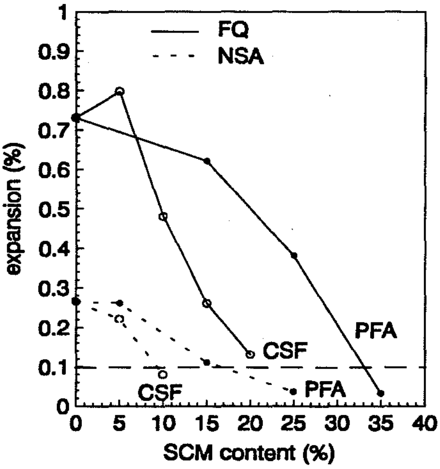


Fig. 2 Effect of SCM content on the 14-day expansion of mortar bars.

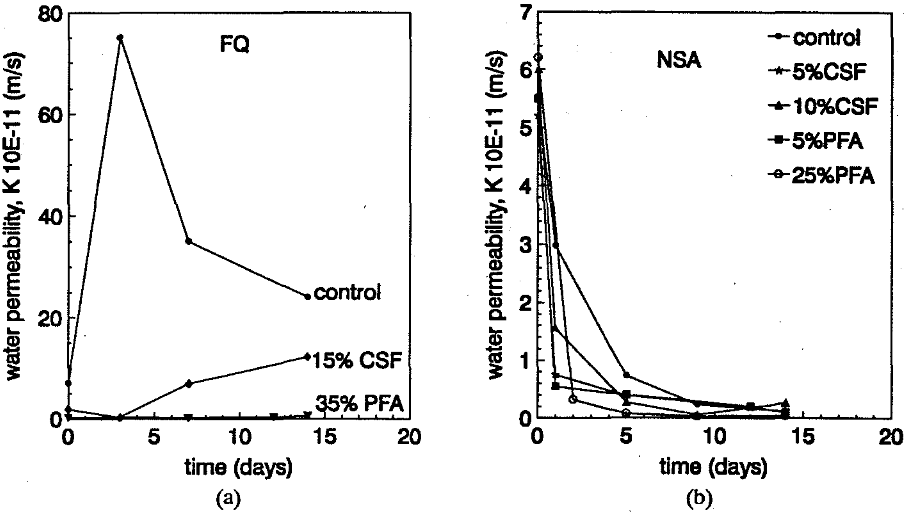
With aggregate NSA, dosages of about 10% for CSF and 17% for PFA are sufficient to reduce the expansion level below 0.10%. With aggregate FQ, due to its higher alkali-reactivity, dosages of about 33% for PFA and higher than 20% for CSF are needed.

Water permeability

Figures 3a and 3b show the values of the coefficient of water permeability, K, for some representative mortar specimens at different times of immersion in the NaOH bath at 80°C. The K value at the time zero (initial permeability) is that measured after the 1-day storage of the specimens in the water bath at 80°C.

With aggregate FQ (Fig. 3a), at a given time, the permeability of blended cement mortars was always lower than that of the control specimen. Furthermore, the change in the permeability with increasing immersion time in NaOH solution was very different for the two types of mortars. The permeability curve exhibited a maximum for control specimen and a minimum for mortar containing 15% CSF. No appreciable variation of permeability with time was observed for mortar containing 35% PFA.

With aggregate NSA (Fig. 3b), the initial permeability of blended cement mortars was comparable to that of the control specimen (about $6 \cdot 10^{-11}$ m/s). During the period of immersion in the NaOH solution, the permeability coefficient decreased for all the mortars down to about $0.1 \cdot 10^{-11}$ m/s. However, this permeability decrease was slower for the control specimen.



(a)

(b)

Fig. 3 Effect of replacing various amounts of Portland cement by PFA or CSF on the water permeability of mortars incorporating FQ or NSA aggregate.

The results in Figures 3a and 3b may be explained taking in mind that alkali-silica reaction may lead to an increase of the mortar permeability, while a reduction in the permeability may occur as a result of a progressive reduction of the porosity of the cementitious matrix due to cement hydration, pozzolanic reaction, and ASR gel formation.

However, blended cement mortars characterized by similar values of permeability (Fig. 3b) showed very different expansion levels (Fig. 1b). This suggests that permeability alone cannot explain the effectiveness of SCMs against ASR.

Pore liquid composition

Figures 4a and 4b show the OH^- ion concentration in the pore liquid within the same mortars as those of Figs. 3a and 3b at different times of immersion in NaOH solution.

After the 1-day storage of mortars in the water bath at 80°C, the OH^- ion concentration in the pore solution was always very low because of a significant leaching of alkali ions out of the specimens (Berra *et al.*, 1994b).

During the period of immersion in the NaOH bath, the OH^- ion concentration within control specimens increased rapidly, because the relatively high permeability of these specimens promoted the penetration of the NaOH from the surrounding solution.

Conversely, for blended cement mortars a slower increase in the OH^- ion concentration or a plateau was observed during a certain period of immersion, depending on the reactivity of the aggregate and the type and amount of SCM used.

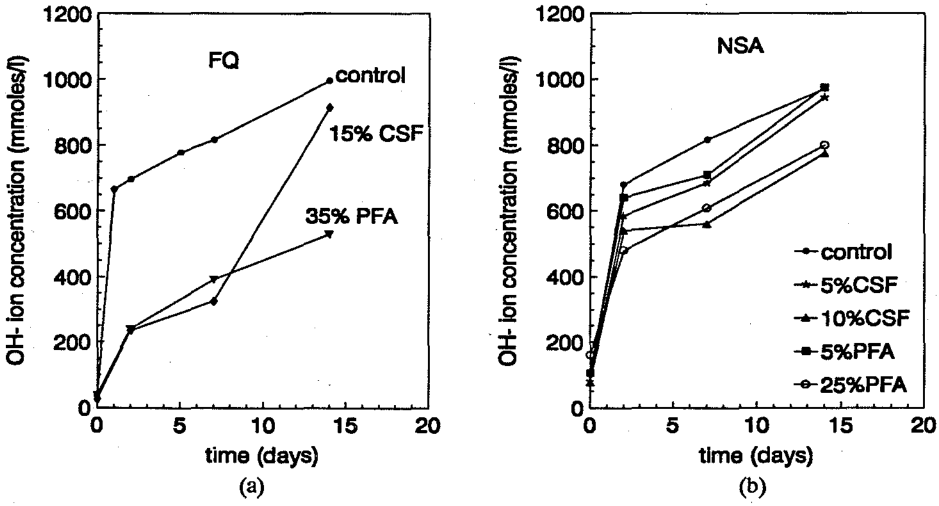
During this immersion period, all the mortars, except 15% CSF, exhibited no appreciable change in the permeability (Figs. 3a and 3b), thus suggesting that the reduced concentration of the OH^- ions within blended cement mortars is largely related to the incorporation of alkali hydroxides into the supplementary, low CaO/SiO_2 CSH gel produced by pozzolanic reaction.

The larger the amount of a given SCM, the lower will be the OH^- ion concentration in the pore liquid and the longer the time period for which a relatively low OH^- ion concentration will be maintained.

For CSF and PFA tested, a diluent effect cannot be invoked to explain the reduced concentration of the OH^- ions in the pore solution, because the total alkali content of these SCMs was comparable (CSF) or above (PFA) to that of the Portland cement used (Table 1).

After 14 days of immersion in a 1M NaOH solution, the OH^- ion concentrations within mortars made with aggregate FQ and 35% PFA (Fig. 4a) or with aggregate NSA and 10% CSF or 25% PFA (Fig. 4b) were still far from having reached the equilibrium concentration. Accordingly, the 14-day expansions of these mortars were very low (Figs. 1a and 1b).

In the other cases examined, the amount of SCM used and, consequently, the OH^- ion consumption by pozzolanic reaction was insufficient to counteract the alkali-reactivity of the aggregate tested, so relatively high expansion levels were recorded.



(a) (b)

Fig. 4 Effect of replacing various amounts of Portland cement by PFA or CSF on the OH^- ion concentration in the pore liquid within mortar bars incorporating FQ or NSA aggregate.

These results are in very close agreement with those of Bérubé *et al.* (1995), who observed that, similarly to more realistic expansion tests performed in air at 100% R.H., the most critical mechanisms involved when testing SCMs by the NaOH bath test are still alkali reduction (by dilution and/or incorporation) and consequent pH decrease in the mortar pore solution.

Portlandite content

Figure 5 shows the portlandite content of some representative mortar specimens as a function of their immersion time in the NaOH bath.

In the absence of SCMs, the portlandite content of control specimens was virtually independent of the aggregate type and immersion time up to about 4 days of immersion in the NaOH solution. Thereafter, a reduction of portlandite content was observed, particularly in the mortars incorporating FQ aggregate (more reactive mortars), because of the formation of a diphasic gel system consisting of CSH and siliceous gel, the latter being responsible for expansion (Wei & Glasser, 1989).

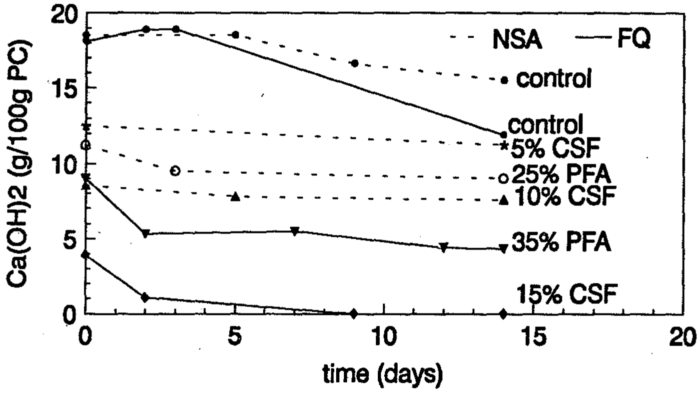


Fig. 5 Time evolution of portlandite content of mortars.

Blended cement mortars generally contained less portlandite than control specimens because of the diluent effect and pozzolanic activity of SCMs. However, at a given immersion time, the differences between the $Ca(OH)_2$ contents of the control and blended cement specimens shown in Figure 5 are indicative of the pozzolanic activity of SCMs, the portlandite content being expressed as g $Ca(OH)_2$/100 g Portland cement.

In most cases, these differences were very high just after the 1-day storage of the specimens in the water bath at 80°C, thus indicating that the storage conditions were suitable to accelerate the pozzolanic reaction of the SCMs tested. This was particularly evident for CSF which proves to be a more pozzolanic material compared to PFA.

As can be calculated from the data in Figure 5, the portlandite consumption varied from 1.14 to 0.80 g $Ca(OH)_2$/g CSF and from 0.22 to 0.18 g $Ca(OH)_2$/g PFA when the content of CSF and PFA in the mortars was varied from 5% to 15% and from 25% to 35%, respectively.

Based on the data in Table 1 and Figure 5, and assuming a CaO/SiO_2 mole ratio of 1:1 for pozzolanic CSH (Bérubé & Duchesne, 1992b), it can be calculated that, after the 1-day water storage, about 96%, 76%, and 68% of silica contained in the CSF would be transformed to pozzolanic CSH when the CSF content is 5%, 10%, and 15%, respectively. With 25% and 35% PFA, the percentage of silica transformed was about 32% and 24%, respectively.

During the period of immersion in the NaOH solution, further $Ca(OH)_2$ consumption was mostly related to the amount of SCM unreacted after water storage. Thus, further portlandite consumption was observed only for mortars containing PFA or high CSF content (Fig. 5).

These considerations coupled with the results of the pore liquid composition suggest that pozzolanic reaction products formed during water storage are able to incorporate alkali hydroxides during the subsequent immersion in the NaOH bath.

CONCLUSIONS

The storage conditions of the accelerated mortar-bar expansion test in NaOH solution (24 hours in a moist room at 20°C, 24 hours in a water bath at 80°C and then 14 days in a 1M NaOH solution at 80°C) are suitable to accelerate the pozzolanic reactions of SCMs such as condensed silica fume and pulverised fuel ash.

Pozzolanic reaction development produces a reduction of the OH^- ion concentration in the pore solution within mortar specimens as a result of the reduced permeability of blended cement mortars (compared to control specimens) and the incorporation of alkali hydroxides by pozzolanic reaction products. The lowering of the OH^- ion concentration is in turn responsible for the reduced expansivity of alkali-reactive aggregates.

These results prove the suitability of the NaOH bath test method as a means for evaluating the effectiveness of SCMs against alkali-silica reaction.

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AN APPRAISAL OF UK GREYWACKE DEPOSITS AND CURRENT METHODS OF AVOIDING AAR

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ABSTRACT

AAR involving greywacke aggregates has been known world-wide for sometime. However, problems related to this aggregate type have only recently been generally recognised and accepted within the UK. This paper presents the findings of a regional survey of UK greywacke deposits, using laboratory and field studies, which was undertaken to establish the likely potential for and extent of further problems. The results from this survey show that about half of the deposits tested would be considered potentially reactive in use. The report also presents findings, continuing the theme from the previous conference, which show that fly ash and ggbs have continued to suppress the deleterious effects of ASR when compared to equivalent opc only concretes, although final evaluation of these materials is not yet possible as expansion continues.
keywords asr, fly ash, ggbs, greywacke, UK.

INTRODUCTION

Greywacke, that ill-defined and highly variable rock of generic type, falls within the marginal areas of sandstone classification. It is essentially composed of a 'compositional jumble' of quartz and reworked rock fragments as clastic material set within a recrystallised clay and microcrystalline quartz matrix.

In common with other generic types of aggregates, such as silicified limestones or flint-based gravels, it is found that greywacke tends to have a reaction characteristic peculiar to its type (CAMNET ed Fourier 1991). Early theories that certain clay minerals within the rock matrix were the reactive constituents have largely been found inapplicable to the majority of reactive greywackes and there is now sufficient evidence to support the view that the reactive constituent is the matrix microcrystalline quartz.

The first recorded field case in the UK of greywacke alkali reactivity was identified in a small dam in the 1970's (Palmer 1978). Little attention was paid at that time to this isolated case as the main concern about AAR in the UK was focused on a number of affected structures which contained flint-based sand and gravels. Therefore, much of the UK guidance on AAR avoidance measures was based and demonstrated using the experience of field and laboratory concretes which contained flint. However, it was the discovery more recently of AAR in a number of older structures (pre -1975), located across the UK (figure 1), and containing reactive greywacke aggregates, which promoted this present investigation. This integrated investigation was undertaken to establish whether UK guidance and avoidance measures would need modification in light of the widening problems relating to greywacke.

Greywacke deposits (which in this study excludes those found as sands and gravels) have a wide geologic and geographic spread within the UK (figure 1) and tend to occur in the less populated areas of the country. Much of the production from the 20-30 quarries in England, Scotland and Wales and another 40 in Northern Ireland, is used as a good quality road surfacing material in asphalt/macadam. However, some greywacke is used in concretes, mainly in mass concrete structures such as dams constructed in the area of outcrop, where there may be few alternative sources of aggregates. For this reason it is important to ensure safe use of this aggregate type.

This paper reports firstly, on the findings of a regional study using standard aggregate testing techniques to establish an overview of the potential problem within the UK. Secondly, the paper updates the results presented at the previous conference (Blackwell et al. 1992) concerning the use of fly ash, and presents new results concerning the use of ggbs to mitigate the effects of AAR. The major structures shown in figure 1 have been discussed in detail elsewhere (Palmer, 1978, Blackwell and Pettifer 1992, Thomas and Blackwell 1992).

REGIONAL OVERVIEW

To obtain an overview of the potential AAR problem within the UK a 'low density' study of UK greywacke deposits has been undertaken. Greywacke samples were collected from sites indicated in figure 2. Most of the collection sites were either working or disused quarries, although occasionally material was collected from exposures such as road cuttings. These samples were subjected to a number of laboratory tests, including: the ASTM C289-87 Quick Chemical Test for potential alkali reactivity, expansion testing using the BS 812 pt 123(draft) (now DD218:1995) ASR-Concrete Prism Test, the ASTM P214 (draft) Accelerated Mortar Bar Test. As previous laboratory testing (Blackwell et al., (1992), Blackwell and Pettifer, 1992) had shown that measured expansion was broadly proportional to greywacke content i.e. no pessimum, all experimentation was undertaken using 100% test aggregate.

The ASTM C289-87 test requires no introduction. Similarly, the ASTM P214 (draft) method was, until final publication as ASTM 1260-94, used widely across the world. Briefly, in this test, expansion measurements are undertaken after mortar specimens have been stored in 1N NaOH for 14 days. The BS 812 pt 123 prism test uses 75 x 75 x (200-300mm) concrete prisms, cast using 700 kg/m^3 cement content with 20,10, & 5mm test aggregate. The required alkali level is 7 kg/m^3 Na_2O_e supplied from a high alkali cement (approx. 1% Na_2O_e). The prisms are wrapped in damp towelling, further wrapped in polythene and sealed over water in screw top polypropylene containers. The specimens are stored at 38°C/100% RH and measured periodically for 1 year. For purposes of this study the prisms were continued on test after 1 year.

EVALUATION OF FLY ASH (EQUIVALENT TO CLASS F) AND GGBS TO MITIGATE THE EFFECTS OF ASR

Current UK guidance for the avoidance of AAR (The Concrete Society, 1987) recommends that the risk of AAR could be minimised by **A) Limiting the mix alkali levels to 3 kg/m^3** by (i) use of a cement with relatively low alkali content , or, (ii) use of OPC/fly ash or ggbs combinations, in which 25 % or more by mass of cement is replaced. **B) Use of a *low alkali* cement** which is either an OPC with <0.6% Na_2O_e or OPC/ggbs combination, where 50% or more ggbs is used.

The aim of this part of the study is to assess the applicability of these recommendations to greywacke aggregates. Concrete mixes have been prepared using two greywacke aggregate samples previously shown to be alkali reactive in the

Maentwrog Dam (Maent) and Northern Irish road bridge (N.I.)respectively. Concrete prisms of a similar size to those used in the BS Prism Test (previously described), have been cast using OPC only, and using 25% & 35% fly ash and 50% and 70% ggbs partial cement replacement. The fly ash was selected to have a high alkali content (4% Na_2O_e) and therefore potentially represents the worst case for fly ash. A cementitious content of 450 kg/m^3 was used in all mixes and the water content controlled to produce a 'slump' of 30-60mm. This resulted in some water reduction in the fly ash concretes of between 7-10%.

Three sources of opc with varying alkali contents were blended to give mix alkali contents in the opc-only specimens in the range 3-5.18 kg/m^3. Higher alkali contents, up to 7 kg/m^3 Na_2O_e, were produced by dosing the mix water with K_2SO_4.
Where K_2SO_4 had been added to the opc-only mix the same level of addition was made to the corresponding mixes containing cement replacement materials. Full chemical analyses of these materials has been previously presented (Blackwell et al. 1992). The storage conditions were identical to those described previously for the BS 812 pt123 prism test.

RESULTS

The results from the standard tests in the regional study are presented in Table 1. The limits placed on the ASTM P214 method are: <0.1% expansion at 14 days (innocuous), 0.1-0.2% (inconclusive) and >0.2% (reactive). At the present time there are currently no officially published limits relating to the BS 812 pt123 (draft). However, a recent discussion document (presented to BSI B/502/6 1995) provides suggested criteria for interpretation of results; In summary these are: <0.05% at one year (non-expansive), >0.05-<0.1% (probably non-expansive), >0.1-<0.2% (possibly expansive) and >0.2%(expansive).

The latest expansion results from the evaluation of cement replacement materials, ranging in time from between 44-56 months for the prism tests, are presented in figures 3,4,5 and 6. The figures are plotted so that only the alkali from the opc (and salts where added in all specimens) is considered As this is an ad hoc test guidance on interpretation of results is not available. Traditionally, expansion values at the termination of expansion of >0.1% are considered *deleterious*, although values between 0.05 - 1% would be considered *significant*, as these values denote the onset of microcracking detectable in thin section. Expansion values <0.05% would be considered innocuous. If the curve of (significant) expansion against alkali level for the concrete containing the cement replacement material lies on the high alkali side of the opc-only curve the cement replacement material can be considered to be inert or 'consuming' alkalis. Conversely, if they lie on the lower alkali side of the opc-only curve they are contributing alkalis. The principles of this approach of appraising the effectiveness of cement replacement materials is explained in greater detail elsewhere (Hobbs, 1988).

DISCUSSION

Preliminary surveys of aggregate deposits using low density sampling on a regional scale, such as the one reported here, have been undertaken in other countries

(Oberholster et al 1978, Gratten-Bellew, 1990) where reactive greywacke aggregates occur. In such studies it is not the intention, and indeed it would not possible, to provide detailed analyses on individual quarries. Similarly, the results presented in table 1 should not be used in that manner. However, these studies do provide a basis for an overview of a rock type recognised only relatively recently in the UK as being potentially reactive. The results from laboratory testing clearly show that approximately half the greywacke deposits surveyed would be classed as 'potentially expansive' in use. There is some discordance between the results of the accelerated mortar bar test and BS prism test method. In general, the mortar bar test has proved a harsher test, classifying some greywacke as 'reactive' or 'inconclusive' which are classed 'non- or probably non-reactive' in use, by the prism method. There is no easy resolution to this discrepancy. Experiments reported from Canada suggest that the accelerated mortar bar test is perhaps too harsh and that its use should be restricted to a method for preliminary screening of aggregates rather than as a definitive method of test. Additionally, there are a few greywackes judged 'non-reactive' in the prism test at 12 months but which have subsequently continued to expand slowly so that at 30 or 42 month the expansion value would be of concern. This phenomenon of slowly expanding greywacke has previously been well documented in Canada (CANMET- ed Fournier 1991) and therefore some caution must be applied when interpreting results . Notwithstanding this, both test methods have correctly classified the material used in structures identified as being affected by AAR as being 'potentially reactive' This clearly provides a degree of confidence in these methods of test. The C289-87 Quick Chemical Test has indicated that all the greywacke tested, including where there has been field occurrence of AAR, would be classed as 'innocuous'. This test method should therefore be considered unsound with this rock type.

Our study of the geographical spread of structures, with good supporting evidence to confirm that the source of the greywacke aggregates was within the locality, supports the findings of the regional study that greywacke alkali reactivity is not related to one geographic area of the UK. However, reactivity also appears not to be related to any easily recognisable geologic or petrographic criteria of the greywacke and in this respect is somewhat of a enigma.

Results from the prism expansion tests on greywacke aggregates broadly support current UK guidance measures for minimising the risk of AAR. However, it should be noted that the alkali threshold for the onset of expansion at 4-4.5 kg/m ^{3} Na_2O_e is significantly less than the threshold for aggregates containing flint-bearing sands and gravels. This might have significance where concrete is placed in extreme conditions. However, it is apparent that to date a low alkali cement (<0.6% Na_2O_e) would have prevented deleterious effects of AAR.

The results from the trials using cement replacement materials show that these materials are so far having a beneficial effect. *Significant* expansions were only observed when these materials were used to replace opc in concretes where the opc equivalents mixes had alkali values of 6 kg/m^3. The only *deleterious* expansion has been found in cement replacement mixes where opc-only equivalent mixes had alkali values of about 7kg/m^3 (ie fly ash replacing a 450kg/m^3 cement content mix using a cement with 1.15% Na_2O_e with additional K_2SO_4). However, as expansion was first observed in cement replacement specimens after about three years, and as expansions are still continuing some caution is required in interpreting these results.

The data from this study therefore continues to support the current guidance regarding the use of ggbs and fly ash outlined in the Concrete Societies' document$^{(6)}$. In common with other countries there is only limited field evidence in the UK to validate such laboratory findings. However, the Nant-y-Moch dam as previously reported (Thomas et al. 1992) contains fly ash concrete with a 'reactive' greywacke aggregate combination and even after 35 years, 25% fly ash replacement was sufficient to prevent deleterious AAR. No structures with a ggbs concrete containing a 'reactive' greywacke aggregate has yet been identified.

CONCLUSIONS

Low density regional studies using field and laboratory methods have shown that greywacke alkali reactivity is not restricted to one geographic or geologic province. The survey has shown that approximately half the UK greywacke deposits tested would be considered 'potentially' reactive in use. Results from prism expansion tests indicate that greywackes react at relatively low alkali levels compared to flint. However, use of low alkali cements or with ggbs or fly ash appears beneficial from the results to date but expansions are continuing and final evaluation is not yet possible.

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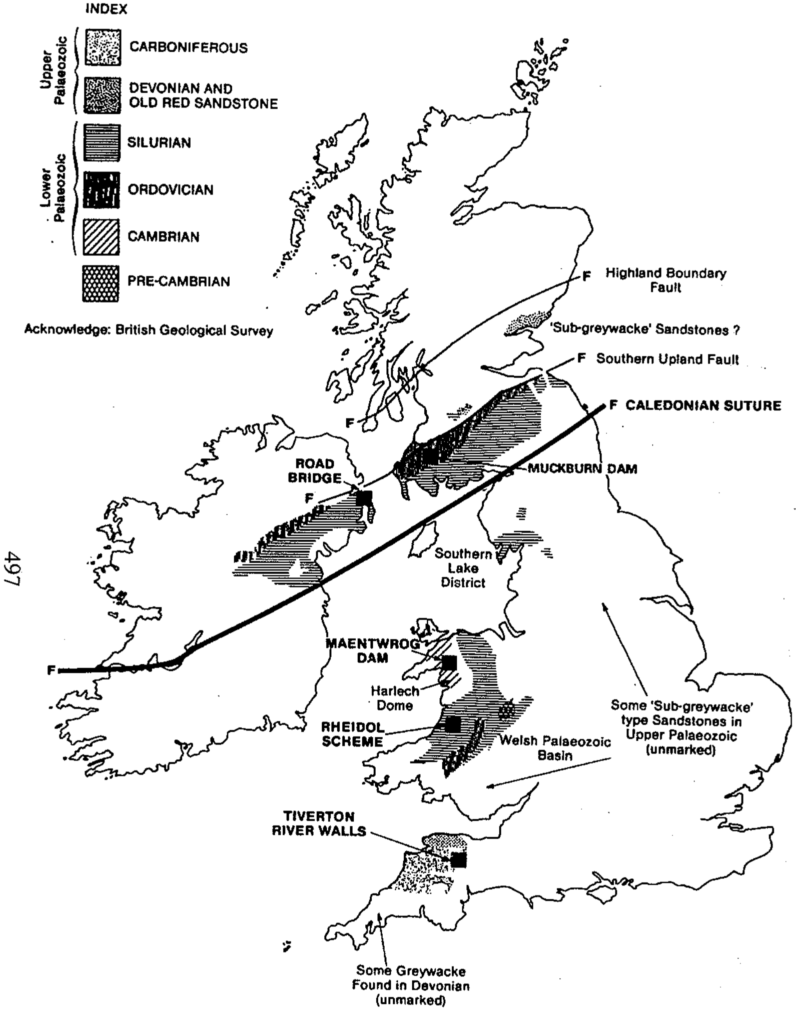

FIGURE 1: Geological map showing strata by geological age where greywacke are found in the UK. Also located are the greywacke containing structures which show distress due to AAR.

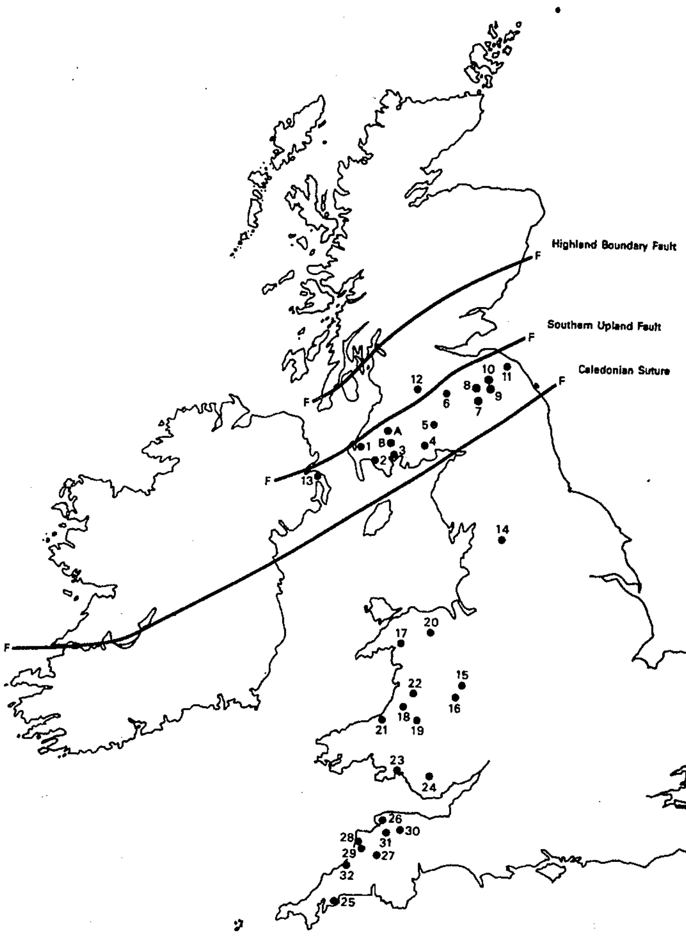

FIGURE 2: Regional survey of UK greywacke, showing sample collection locations (results of laboratory testing of these samples are shown in table 1.)

Sample	ASTM P214 Exp %	BS Prism Test			C289-87	Classification				
		12 m Exp %	30 m Exp %	42 m Exp %	RC:SC	ASTM P214	BS Prism Test			C289-87
							12 m	30 m	42 m	
Scotland and Northern Ireland										
A*1	N/A	0.31	N/A	N/A	N/A	N/A	E	E	E	N/A
B	N/A	0.12	N/A	N/A	N/A	N/A	PE	-	-	N/A
1	0.21	0.19	0.28	0.3	N/A	R	PE	E	E	N/A
2	0.26	0.22	0.35	0.35	N/A	R	E	E	E	N/A
3	0.45	0.18	0.19	0.20	N/A	R	PE	PE	E	N/A
4	0.32	0.15	0.18	0.19	N/A	R	PE	PE	PE	N/A
5	0.15	0.02	0.04	0.05	N/A	I	NE	NE	PN	N/A
6	0.19	0.05	0.008	0.09	N/A	I	NE	PN	PN	N/A
7	0.14	0.06	0.10	0.11	N/A	I	PN	PE	PE	N/A
8	0.11	0.01	0.01	0.02	N/A	I	NE	NE	NE	N/A
9	0.15	0.01	0.04	0.05	N/A	I	NE	NE	PN	N/A
10	0.10	0.005	0.02	0.03	N/A	I	NE	NE	NE	N/A
11	0.13	0.005	0.02	0.03	N/A	I	NE	NE	NE	N/A
12	0.16	0.005	0.03	0.04	N/A	I	NE	NE	NE	N/A
13*2	0.49	0.20	N/A	N/A	113:30.4	R	E	E	E	In
N England and Wales										
14	0.47	0.18	0.18	0.18	44:26.7	R	PE	PE	PE	In
15	0.19	0.04	0.09	0.11	55:32.9	I	NE	PN	PE	In
16	0.20	0.11	0.20	0.21	79:29.4	R	PE	E	E	In
17*3	0.25	0.16	0.16	N/A	36:24.6	R	PE	PE	PE	In
18*4	0.28	0.22	0.25	N/A	51:34.4	R	E	E	E	In
19	0.38	0.25	0.27	0.29	38:33.0	R	E	E	E	?
20	0.27	0.08	0.18	0.19	46:44.3	R	PN	PE	PE	?
21	0.25	0.06	0.11	0.12	78:37.7	R	PN	PE	PE	In
22	0.32	0.20	0.24	0.26	59:48.6	R	E	E	E	?
23	0.18	0.07	0.11	0.12	225:16.7	I	PN	PE	PE	In
24	0.31	0.07	0.09	0.11	146:60.0	R	PN	PN	PE	In
Devon and Cornwall										
25	0.21	0.05	0.06	N/A	67:35.3	R	PN	PN	N/A	In
26	0.06	0.03	0.06	N/A	143:24.4	N/R	NE	PN	N/A	In
27	0.35	0.03	0.06	N/A	100:45.5	R	NE	PN	N/A	In
28	0.11	0.03	0.04	N/A	103:0.6	I	NE	PN	N/A	In
29	0.09	0.02	0.04	N/A	130:190	R	NE	PN	N/A	In
30	0.15	0.31	0.34	N/A	68:43.2	R	E	E	N/A	In
31	0.29	0.30	0.31	N/A	14:58.1	R	E	E	N/A	In
32	0.23	0.05	0.06	N/A	165:34.6	R	PN	PN	N/A	In

N/A = Not Available; I = Inconclusive; N/R = Non-reactive; R = Reactive; In = Innocuous; E = Expansive in use;
PE = Possibly expansive in use; PN = Probably Non-Expansive in use; NE = Non Expansive in use
*1 = Muckburn Dam; *2 = N. I. Road Bridge; *3 = Maentwrog Dam; *4 = Dinas Dam and Nant-Y-Moch Dam (fly ash)

Table 1 **Summary of the laboratory test data for the UK Regional Survey of greywacke deposits**

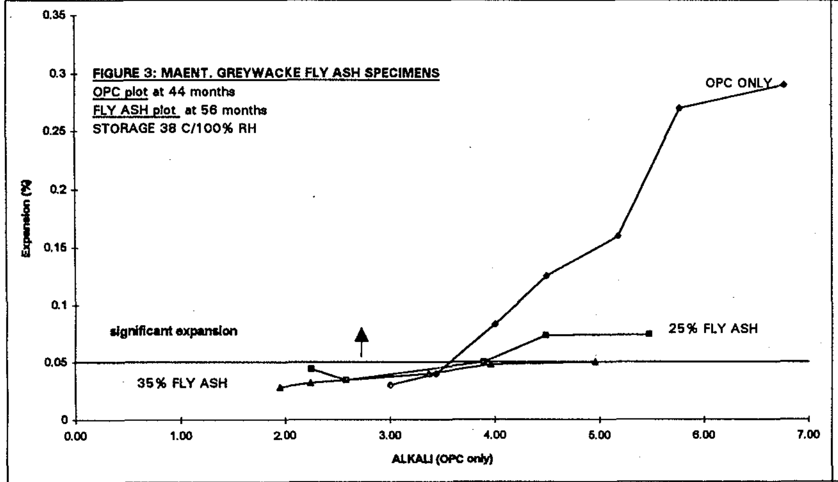

FIGURE 3: MAENT. GREYWACKE FLY ASH SPECIMENS
OPC plot at 44 months
FLY ASH plot at 56 months
STORAGE 38 C/100% RH

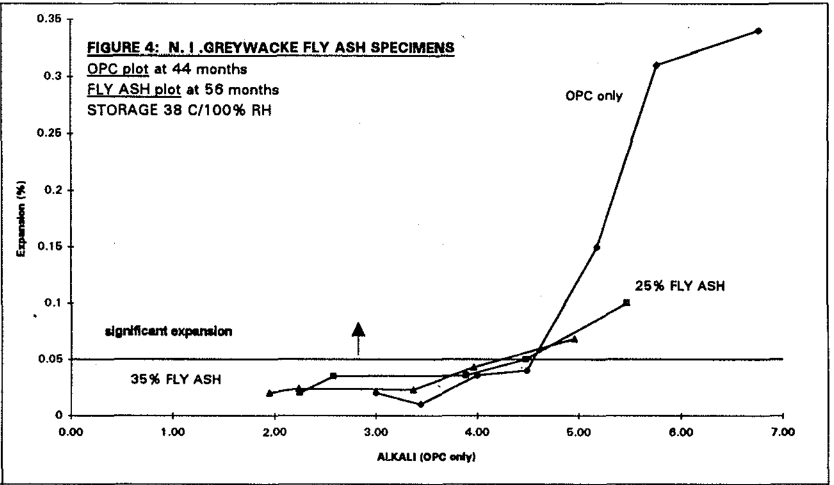

FIGURE 4: N. I .GREYWACKE FLY ASH SPECIMENS
OPC plot at 44 months
FLY ASH plot at 56 months
STORAGE 38 C/100% RH

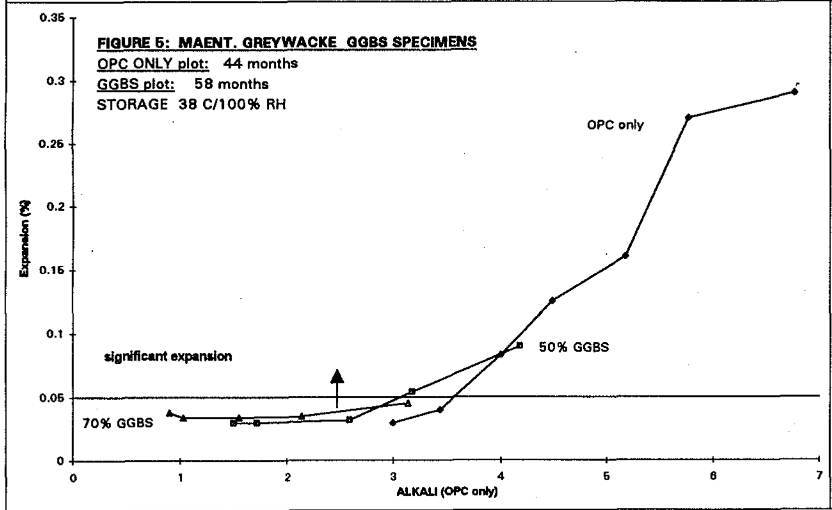

FIGURE 5: MAENT. GREYWACKE GGBS SPECIMENS
OPC ONLY plot: 44 months
GGBS plot: 58 months
STORAGE 38 C/100% RH

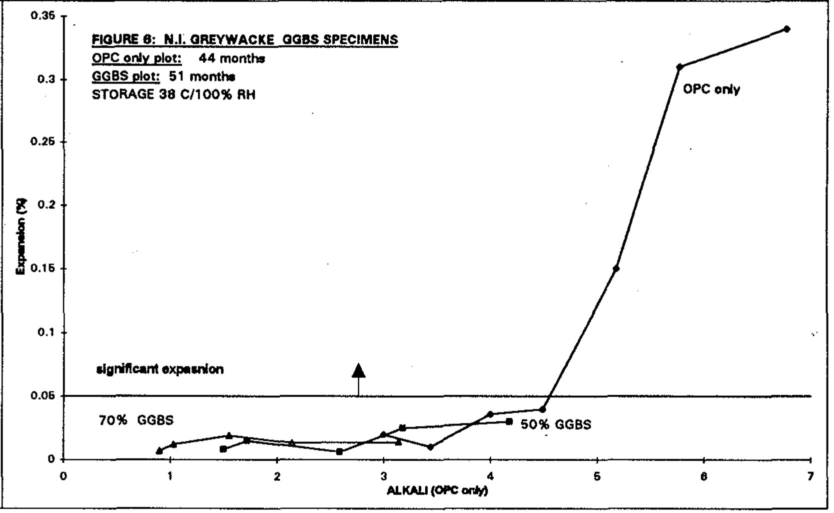

FIGURE 6: N.I. GREYWACKE GGBS SPECIMENS
OPC only plot: 44 months
GGBS plot: 51 months
STORAGE 38 C/100% RH

THE ASSESSMENT & MANAGEMENT OF ALKALI-SILICA REACTION IN THE GORDON RIVER POWER DEVELOPMENT INTAKE TOWER

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ABSTRACT

The Gordon River Power Development in south-west Tasmania includes an underground power station which receives water from Lake Gordon through a prestressed concrete Intake Tower and vertical shaft. The Intake Tower contains an 8.2m diameter steel cylinder gate to close off the Intake in an emergency or for maintenance work. The discovery in 1992 that the twelve guide shoes on this cylinder gate had lost precompression led to investigations which showed that the concrete Tower was expanding due to alkali-silica reactivity (ASR). This paper describes the testing and monitoring that has been undertaken in the assessment of the ASR and the consequences of the resultant expansion on the operation of the Intake. A prediction of the ultimate expansion has been made from accelerated laboratory testing of concrete prisms. This prediction is being used in the formulation of a long term management strategy for the Intake Tower so that the power station can continue to operate safely in the future.

Keywords: alkali-silica reactivity, concrete aggregates, hydroelectric power, maintainability, power plant intakes.

INTRODUCTION

The Gordon River Power Development of the Tasmanian Hydro-Electric Commission is located in the remote south-west corner of Tasmania. It was constructed between 1967 and 1977 and includes four dams, forming Lake Gordon and Lake Pedder, and the underground Gordon Power Station housing three 144MW generators. The associated infrastructure for the Power Development included an 85km sealed access road from the logging town of Maydena, the construction township of Strathgordon, other site access roads, and a canal connecting Lake Pedder to Lake Gordon.

The 140m high concrete arch Gordon Dam forms Lake Gordon whilst the 38m high Serpentine Dam (concrete faced rockfill), the 43m high Scotts Peak Dam (bitumen faced rockfill) and the 17m high Edgar Dam (concrete faced rockfill) extend the original Lake Pedder. Water from Lake Gordon reaches the underground power station via a 140m deep vertical intake shaft and a horizontal power tunnel and distributor manifold. Discharge from the power station is via a 1.6km long tailrace tunnel into the Gordon River.

The vertical intake shaft is concrete lined and has an internal diameter of 8.23m. On top of the shaft is a 76m high prestressed concrete Intake Tower, the foundations of which extend 32m into the Intake Shaft in the form of a thickened lining (Lea, 1982). Only the top 14m of this Tower projects above the normal full supply level of Lake Gordon. Near the base of the Tower an enlargement contains six equal rectangular openings which allow water to flow from the Lake into the Intake Shaft. Within the shaft these six openings can be closed off in an emergency by an hydraulically operated steel cylinder gate. This gate has guide shoes which bear onto vertical steel gate guides embedded in the concrete of the Tower. The discovery during routine checks in 1992 that the preload between the guide shoes and the gate guides had been lost led to investigations which finally confirmed the presence of alkali-silica reactivity in the concrete Intake Tower as well as in other concrete works of the Gordon River Power Development.

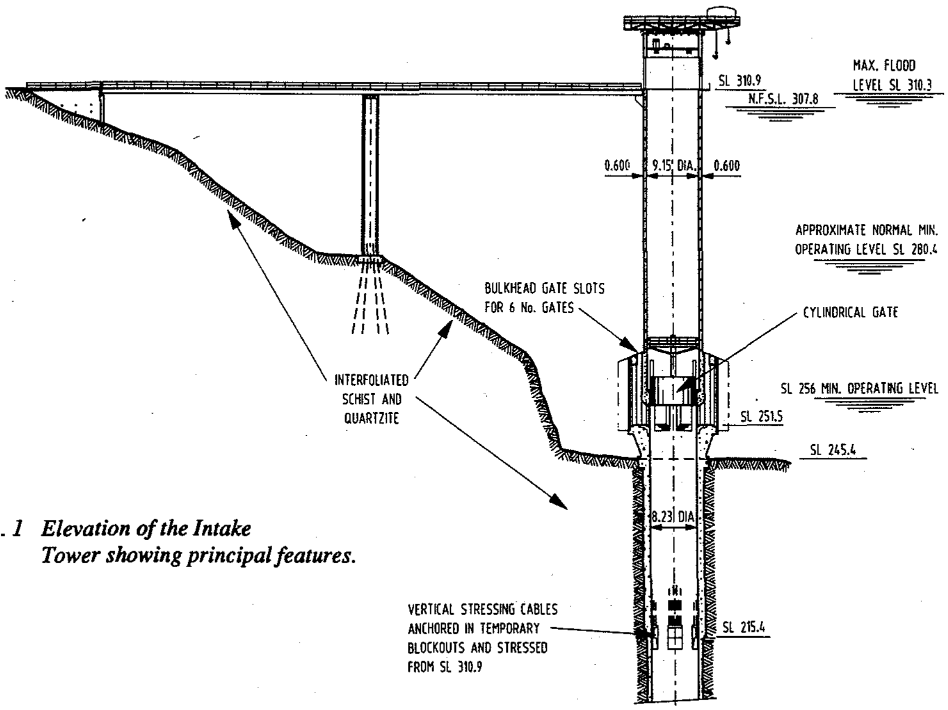

Fig. 1 Elevation of the Intake Tower showing principal features.

DESCRIPTION OF THE INTAKE TOWER

The Intake Tower is a heavily reinforced and prestressed concrete structure, Figures 1 &2. Where it projects from the Intake Shaft it is in the form of a conic frustrum, 6m high and increasing in outside diameter from 10.5m at the shaft collar to 15.5m. On top of this frustrum are six pillars, each 12m high and with a roughly elliptical cross-section of area about 8m². Between the bottoms of these pillars are the six water entry openings, each approximately 3.8m square. On top of the six pillars the Tower extends for 57m as a cylinder with an internal diameter of 9.15m and a wall thickness of 600mm.

At the outer perimeter of the six pillars there are trashrack screens to prevent the entry of timber into the Intake Shaft and inside these screens there is provision for the

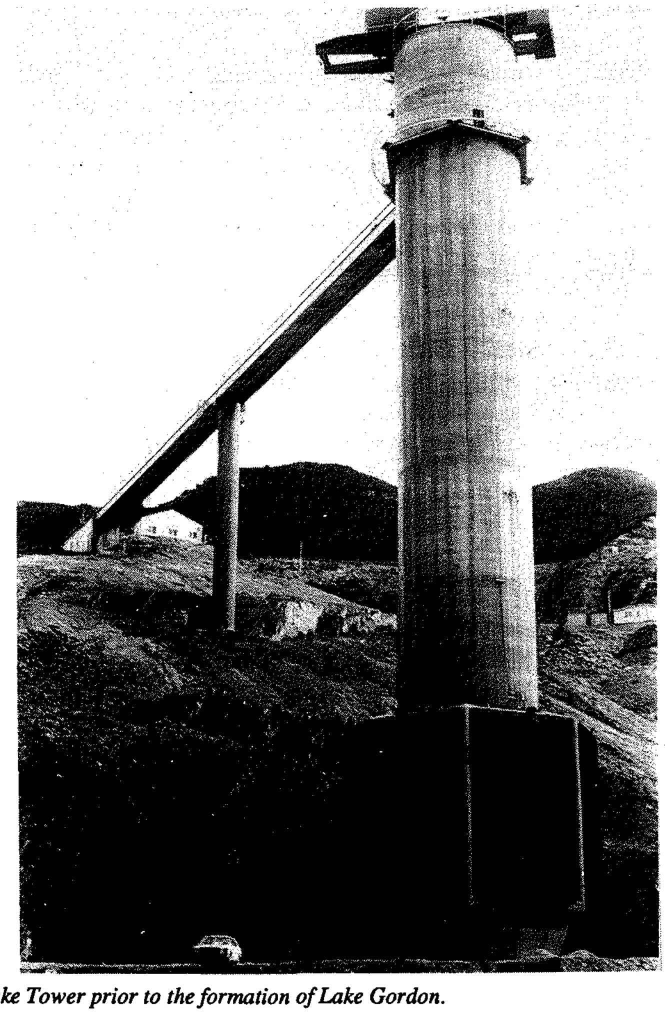

Fig. 2 The Intake Tower prior to the formation of Lake Gordon.

installation of six bulkhead gates which enable the Tower and shaft to be dewatered for maintenance work. A permanent rotating crane on top of the Tower enables the trashrack screens and bulkhead gates to be removed and replaced.

Prestressing of the Tower was by six groups of five prestressing cables, each about 97m long, extending from anchorage blockouts about 30m down the Intake Shaft, up through the six pillars to the top of the Tower. Each cable was stressed to an average load of about 2000kN. Reinforcement in the six pillars is typically 35mm diameter bars at 300mm centres, both ways at 75mm beneath the concrete surface and in the upper cylindrical section of the Tower, 25mm diameter bars at 300mm centres both ways and at both faces with once again 75mm concrete cover.

The steel cylinder gate is designed to close against station flow if there is some major leakage emergency in the underground Power Station. The gate is 8.22m outside diameter and 8.1m high. It is connected to a central hydraulic ram by a three arm framework, the hydraulic ram being supported by a similar three arm framework attached to the inside of the Tower. The gate seals at the top and bottom perimeters against hydraulically inflated circumferential rubber seals. These rubber seals are mounted in two circumferential steel housings embedded in the internal concrete surface of the Tower immediately above and below the six water openings.

The gate is guided by six vertical machined steel gate guides embedded along the internal edges of the six concrete pillars and in the tower above the pillars. Two sets of six guide shoes, at the top and bottom perimeters of the gate, bear onto the gate guides to maintain gate alignment during operation. Each guide shoe can be adjusted in a radial direction so that preload between a shoe and its gate guide can be changed. Such adjustment was used initially, together with the inherent flexibility of the gate shell, to round up the gate to within its diametrical tolerances.

CONCRETE SUPPLY FOR THE GORDON RIVER POWER DEVELOPMENT

Concrete for the Gordon River Power Development was produced at a batch plant established and operated by the HEC near the Gordon Dam. Aggregate was obtained from a quarrying and crushing operation using locally available quartzite rock and natural sand was obtained from McPartlan Pass about 30km from the batch plant. Cement was type A (GP) obtained in bulk from the Goliath Portland Cement Company in the north of Tasmania. Records of cement testing by the Goliath Company between 1969 and 1974 indicate total equivalent sodium oxide contents ($Na_2O + 0.66K_2O$) of between 0.53 and 0.58% for their type A cement.

The quartzite rock was tested in 1969 for potential reactivity of aggregates with cement alkalis in accordance with ASTM Test C227. These tests indicated no potential reactivity, the mortar bars made and tested in accordance with the ASTM procedure showing a shrinkage after one year of 120 microstrain and no surface cracking, mottling or exudation. Petrographic examinations at about the same time also indicated that the Gordon quartzite was largely free of potentially reactive minerals. The quartzite averaged 90% quartz with 10% of other minerals, mainly sericite. In the words of one petrologist at the time "the presence of minerals likely to participate in alkali-aggregate reaction in concrete was not recognised".

Fly-ash was used as a partial cement replacement for some of the concrete used on the Gordon Scheme, principally in concrete for the Gordon Dam. The main reason for its use for the dam was to increase concrete strength gain between 7 and 91 days, but reduction in heat evolution was an important secondary reason, contributing to temperature control and thus to the avoidance of cracking in the dam. The fly-ash was from Port Augusta in South Australia and was used generally to replace between 20% and 25% of the cement, although up to 35% replacement was employed for a period. The variation in the amount of fly-ash used was because of continuing testing and reassessment during the three year period of the Gordon Dam construction. Concrete containing fly-ash was used in some other Gordon structures, but was not used in the construction of the Intake Tower.

RECOGNITION OF ALKALI-AGGREGATE REACTIVITY IN THE GORDON CONCRETE

Some cracking in minor structures on the Gordon River Scheme was noticed in the late 1980's, that is about 15 years after construction, but it was not until 1991 that the characteristic map cracking of AAR was recognised. Subsequent petrographic examinations of drill core specimens from some affected structures and accelerated laboratory tests on mortar bar specimens by the CSIRO (Shayan et al, 1988) and on concrete prism specimens by the Queensland Department of Transport (Carse and Dux, 1990) confirmed the presence of alkali-silica reactivity in concrete made with the Gordon quartzite aggregate. The same concrete with fly-ash has not shown ASR in the accelerated tests and the Gordon Dam in particular has not shown any cracking that can be associated with ASR.

In October 1992 routine checks on the operation of the cylinder gate in the Gordon Intake Tower resulted in significant water loss past the inflatable rubber seals and a loss of preload between the twelve gate guide shoes, which are attached to the cylinder gate, and the gate guides, which are embedded in the concrete of the Intake Tower. Similar checks in 1988 had not revealed any problems with the cylinder gate operation. Subsequent survey measurements showed that whilst the cylinder gate had not changed from its manufactured dimensions, the concrete Tower, housing the six gate guides and the inflatable seals, had expanded by between 6 and 10mm in diameter representing a diametral strain of between 750 and 1200 microstrain.

A number of causes for the observed expansion of the Tower were considered including loss of prestress, temperature changes, creep of the concrete, a low lake level which had persisted for some years prior to 1992 and alkali aggregate reaction. These considerations were assisted by a three-dimensional finite element analysis of the tower in which prestress loads, a change in water level and a change in temperature to simulate ASR expansion were modelled. Likely losses of prestress and temperature changes were determined to produce negligible effects. The lower lake level and a reduction in concrete modulus to a third of its initial value to represent creep, each only produced diametral strains of about 100 microstrain, about a tenth of the observed diametral strain. It was therefore concluded that only alkali aggregate reaction could explain the magnitude of the observed expansion. Petrographic examination of drill cores taken from the Tower subsequently showed the presence of ASR in the concrete and together with the already identified ASR in other Gordon structures it was concluded that the expansion of the Tower was principally due to ASR.

Further investigations and measurements were made during power station outages in October 1993 and October 1994 from which a trend of continuing concrete expansion was deduced. The absence of significant cracking in the structure, and particularly the absence of map cracking, is attributed to the heavy reinforcement and the vertical prestressing in the Tower. Precise survey observations of the level of the top of the Tower suggested that little vertical expansion had occurred. In this context a vertical strain of 1000 microstrain, which would be similar to the observed diametral strain, would have increased the height of the 70m tower by 70mm which would have been detectable by the survey measurements. It can be noted that the prestressing loads produce an average vertical compression in the upper cylindrical section of the Tower of about 3MPa. The structural competence of the Tower was assessed by making accurate measurements of the deflection of the top of the Tower while the tower-top crane rotated a 10t stop-log into four positions around the Tower. The small deflections were consistent with a concrete modulus of 30GPa which is a value that would be expected for the 28MPa concrete.

After three years of assessment it has thus been concluded that the Tower is currently structurally sound and should remain so for its design life of about 100 years. However the expansion threatens the satisfactory operation of the cylinder gate.

PREDICTION OF STRAIN TRENDS IN THE INTAKE TOWER

The results of diametrical strain measurements in the Intake Tower, at the level of the inlets, for the 1992, 1993 and 1994 measurements are shown in Fig. 3. It is apparent that extrapolation of a curve of strain trend from these three measurements cannot be reliably made.

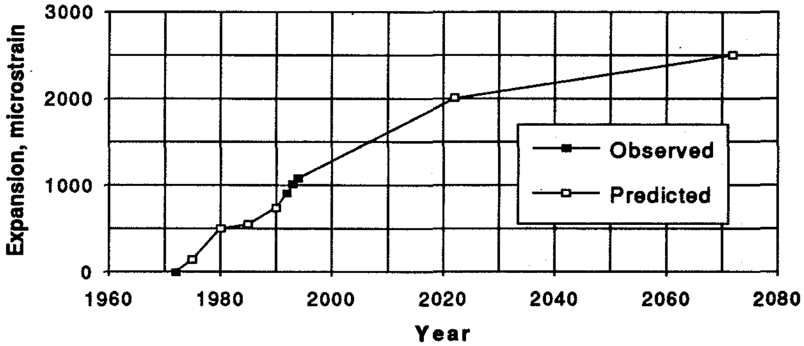

Fig. 3 Observed and predicted expansions for the Intake Tower

However, as part of the initial investigations into the ASR problem in the Gordon concrete, six concrete prisms were made by the Queensland Department of Transport and subjected to their accelerated ASR Test. In this test the prisms, 75mm x 75mm x 250mm long, are steam cured and then stored at 50°C and 100% relative humidity for an extended period and their length is measured every month. The prisms were made using Gordon fine and coarse aggregate, the same South Australian fly-ash used in the Gordon concrete in three of them, and Queensland Cement type GP fine. These tests commenced in October 1992 and were concluded in January 1994 when an expansion of 1100 microstrain had developed in the non-fly-ash specimens and 500 microstrain in the specimens with 20% fly-ash replacement of cement.

A regression analysis was performed on the concrete prism expansion results representing the concrete mix used in the Intake Tower. Using this relationship, Table 1 contains a prediction of the anticipated concrete expansion due to alkali-silica reaction within the Intake Tower over a life of 100 years and Fig. 3 presents this data graphically.

Table 1 Predicted expansion of the Intake Tower

Year	1972	1975	1980	1985	1990	1992	1993	1994	2022	2072
Expansion	0	140	500	550	740	912*	1008*	1082*	2000	2500
(Microstrain)						* Observed values				

From Table 1 the ultimate ASR strain is conservatively predicted as 2500 microstrain at a field age of 100 years i.e. 2½ times its present maximum value as measured in October 1994. It can be noted that the laboratory conditions are more severe than the field conditions with a laboratory temperature of 50°C (7-12°C in the field), Na_2O equivalent alkali level of 5.2kg/m³ (2.3 to 3.2 in the field) and the field concrete being heavily reinforced and prestressed. As a consequence 22 years in the field are required to reach the one year accelerated ASR strain.

MANAGEMENT OF THE ASR PROBLEM

The 432MW Gordon Power Station plays a major role in the Tasmanian electricity generating system. Its annual contribution of energy to the system over recent years has averaged 13% and has been up to 18%. Any extended loss of this power station could thus be serious for the Tasmanian economy. Whilst complete loss of the Intake Tower due to ASR is now seen to be very unlikely, even in the long term, the inability of the cylinder gate to operate correctly could have serious consequences in the event of a major water leakage in the underground power station. Such incorrect operation could be due to the gate seals not working. However, more seriously, incorrect operation could also be due to damage to the gate due to vibration because the gate guide shoes are loose or to damage to the support brackets for the three arm framework supporting the hydraulic gate operating ram due to tower expansion.

The short term management of the ASR problem in the Intake Tower has thus aimed at designing modifications to the inflatable rubber gate seals, to the adjustable gate guide shoes and to the support brackets for the hydraulic ram support framework. To carry out such design modifications the above deduced indications of the ultimate expansion of the Tower and the time trend for this expansion are crucial items of design data.

In the longer term the continuing management of the ASR problem will include periodic measurements of the expansion of the Tower to verify or modify the current predictions of ultimate expansion and its timing. Cracking in the structure will also be monitored and the likelihood of corrosion of reinforcing steel or of the prestressing cables will be assessed from time to time. Methods of sealing such cracks have currently been assessed but as it could be quite some years before corrosion becomes a potential problem, if in fact it ever does, new techniques or materials might by then have been developed.

CONCLUSIONS

Visual inspection, petrographic examinations and laboratory accelerated mortar bar and concrete prism tests have confirmed the presence of alkali-silica reaction in concrete made for the Gordon River Power Development using Gordon quartzite aggregate. One significant structure that has been affected by ASR is the 76m high Gordon Power Station Intake Tower. Although the 20 year old Tower is showing minimal cracking, which is attributed to its heavy reinforcement and prestressing, the ASR induced diametral expansion has affected the operation of the emergency cylinder gate that it contains. As a consequence the gate does not currently seal properly and further expansion could threaten its mechanical integrity.

The importance of the Gordon underground power station to the Tasmanian electricity generating system has required a management plan to be formulated to deal with the ASR problem in the Intake Tower. In the short term this plan requires modifications to be made to the cylinder gate so that the Tower expansion can be accommodated. In the longer term the continuing expansion of the Tower will be monitored, crack surveys will be carried out periodically and methods of sealing the Tower concrete to prevent corrosion of reinforcing steel via ASR cracks will be investigated in case such sealing becomes necessary in the future.

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EFFECT OF SILICA FUME ON REDUCING RISK OF FREE LIME EXPANSION IN CEMENTLESS CONCRETE

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ABSTRACT

Composition and technology of fine-grained cementless concrete based on high-calcium ash from thermal power plants were developed by the Siberian State Mining & Metallurgical Academy. The concrete does not contain any natural (rubble, gravel, sand) or artificial (claydite, aggloporite, polystyrenefoam) aggregates. However, high-calcium ashes often contain 5 to 20% free calcium oxide which may cause concrete determination by its expansion in the presence of moisture.

In order to prevent expansion, silica fumewaste product from Kuznetsky Ferroalloy plant - was introduced into the concrete. To provide a better silica fume - free calcium oxide interaction, the ash was ground to a fineness of 4000 cm2 /g and hot water was used for mixing the concrete. Besides, concrete placed in a formwork was heated using a 4+3+8+3 h cycle. The use of 10 to 20% silica-fume by weight of ash and double heat treatment of concrete resulted in a 5 to 20 MPa strength class of concrete with free calcium oxide content reduced from 10 to 20 to 1 to 3%.

Keywords: High-Calcium Ash, Silica Fume, Cementless Fine-Grained Concrete, Silica Fume - Free Calcium Oxide Interaction.

INTRODUCTION

The paper presents the results of the study of the effect of silica fume on eliminating the risk of expansion due to hydration of free lime in a cementless fine-grained ash-slag concrete. The ash-slag concrete is based on a high-calcium ash from the Abakanskaya thermal power plant, slag sand with a particle size of 0 to 5 mm being used as an aggregate.

High-calcium ashes have good binding properties (Savinkina & Logvinenko 1979), (Papayianni 1993), but containing free calcium oxide, they affect concrete adversely. This negative effect can be eliminated by binding free calcium oxide with silica fume.

Previous investigations (Pavlenko 1992), (Pavlenko & Oreshkin 1992) showed that double heat treatment of the concrete components provided optimum results: using hot water (60 to 80°C) for mixing concrete and its heating at 80 to 90°C after placing and compacting.

MATERIALS

Fly ash

Fly ash from the Abakanskaya thermal power plant is produced by combustion of powdered coal from the Irsha-Borodinsky basin. It is collected by electrostatic filters and then removed by water to landfills where it loses its binding properties.

Samples from all electrostatic filters were taken for study. Characteristics of the ash are given in Table 1.

Table 1. Physical properties and chemical analysis of fly ash and silica fume

Physical properties	FA*	SF*	Chemical analysis	FA*	SF*
Fineness, Blaine, m2/kg	245	2200	SiO2 (total), %	39.45	90.11
Specific gravity, mg/m3	2.4	2.2	SiO2 (free)	24.20	90.00
Residue on sieve No 008	7.5	--	CaO (total)	31.20	0.71
Bulk density, mg/m3	1.2	0.2	CaO (free)	8.96	--
Water demand, %	42	40	MgO	6.31	0.97
Hydraulic activity, mg/g	--	102	Al2O3	7.11	1.93
			Fe2O3 +FeO	10.79	1.82
			SO3	0.86	0.89
			MnO	0.18	0.205
			TiO2	0.9	0.015
			Na2O	0.4	0.87
* - Fly ash			K2O	0.7	1.16
** - Silica fume			Loss on Ignition	1.85	2.25

According to Ivanov (Ivanov 1986), ash from the Abakanskaya thermal power plant is referred to as a coarse, polydispersed ash (ash particles are agglomerated). To destroy the agglomerated particles and fused cover of the ash, it should be ground to a fineness of 400 to 450 m2/kg. Grinding improves physical properties and chemical activity of the ash and eliminates expansion of the concrete.

The coefficient of quality (Ignatova 1990), determining binding properties of ash (C=CaO+Al2O3+MgO/SiO2=1.13), indicates that ash from the Abakanskaya thermal power plant has good binding properties (C≥1). Besides, the ash has a great reserve of a potential activity: high content of free silica in the amorphous state (24.2%) increases the binding properties of the ash after grinding of the ash and its heat treatment. The grinding also increases the activity of minerals containing magnesium.

The ash contains negligible amounts of unburnt organic particles and sulfur compounds (1.85 and 0.85, respectively).

Silica fume

The waste product from the Kuznetsky Ferroalloy plant is a superdispersed powder of a light grey color with a high content of an amorphous silica which, under certain conditions (grinding of ash and double heat treatment), reacts with a free calcium oxide of the ash and protects concrete from deterioration during its performance.

The data on properties of silica fume are given in Table 1.

X-ray diffraction and differential thermal analysis show that the silicon dioxide is in the amorphous state and reveals hydraulic activity when interacted with lime. The hydraulic activity of the silica fume (102 mg CaO/g) was determined by the amount of lime absorbed by the silica fume from the saturated solution at 85°C.

Slag sand

The study of slag from the Abakanskaya thermal power plant was performed in accordance with the requirements of State Standard 26644-85 (Gosstroy USSR, 1986). The ordinary granulated fuel slag was graded to receive two grading fractions: 5 to 10 mm (slag rubble - 7.5%) and 0.14 to 5 mm (slag sand - 92.5%). The material above 5-mm particle size was tested for stability of the structure (silicate and ferruginous decomposition). The losses in weight of the slag (three samples) were maximum 6.7% (versus 8% standard) and 4.6% (versus 5% standard) for silicate and ferruginous decomposition tests, respectively. Frost resistance of the slag (three samples) was 96-102 cycles and after its grinding to 0 to 5 mm particle size it increased to 150-235 cycles. Thus, to achieve the 100 percent utilization of the slag, it should be ground to sand at a roller crusher.

The fineness modulus of the slag sand was 2.5 to 3.1. Its bulk and true densities were 1580 and 2200 kg/m3, respectively.

The chemical analysis of the slag sand is given in Table 2.

Table 2. Chemical analysis of slag sand

Oxides	Quantity, % by mass	
	According to State Standard 26644-85	
SiO2 (total)	not standardized	56.47
SiO2 (free)	not standardized	15.57
CaO (total)	not standardized	29.92
CaO (free)	not more than 1	0
MgO	not standardized	3.50
Al2O3	not standardized	8.16
FeO+Fe2O3	not standardized	9.63
MnO	not standardized	0.17
P2O5	not standardized	0.05
SO3	not more than 3	0.11
Loss on Ignition		
for dense slag	not standardized	0
for porous slag	not more than 3	0

The results of the tests show that the sand produced from the slag of the Abakanskaya thermal power plant may be used as a replacement for natural aggregates (crushed stone and sand) in fine-grained concretes and mortars.

EXPERIMENTAL DETAILS

Mixture proportioning

A lot of formulas and methods for proportioning concrete mixtures are available (Bazhenov 1975; Batrakov 1990; Buzhevich 1970; Stork 1971), however, none of them can be applied to a cementless concrete. It is attributable to the fact that high-calcium ash, being a multicomponent system, includes both binding (minerals) and inert substances and, to some extent, may serve as a microfiller.

Besides high-calcium ash, concrete was to contain silica fume to bind free calcium oxide and slag sand, to increase its strength and improve its deformation properties. High-calcium ash and silica fume were considered to be a single binder while slag sand was an aggregate.

A series of tests were made to determine mixture proportions for cementless concrete intended for load-bearing structures. First, the optimum ash-to-water ratio was determined (Fig.1) and then, the effect of the temperature of mixing water on a compressive strength of an ash concrete was studied (Fig. 2).

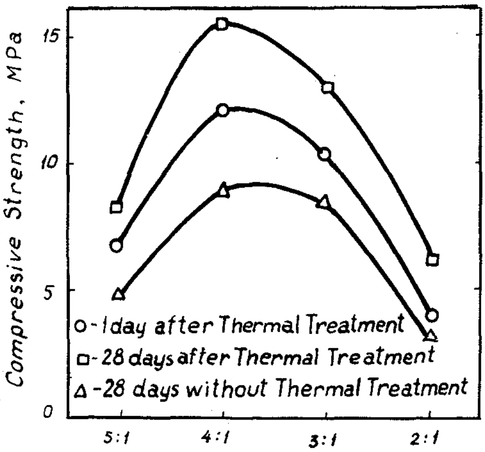

Fig.1 Compressive strength of cementless concrete versus ash-to-water ratio (A/W)

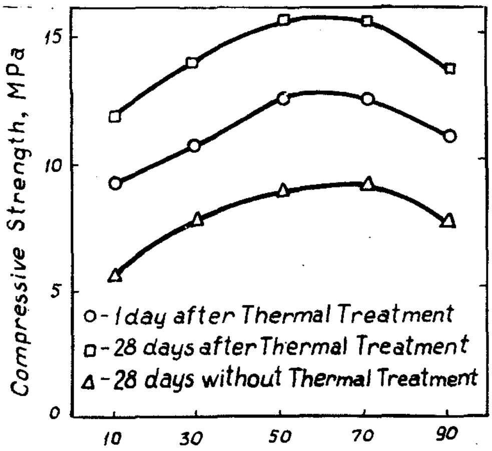

Fig.2 Compressive strength of cementless ash concrete versus the temperature of mixing water (t°C)

As can be seen from the data, the best results were obtained with the ash-to-water ratio of 4:1 and with the temperature of mixing water of 60 to 80°C. However, a slight expansion of cube specimens at their open surface was observed but without visible cracks.

To eliminate expansion, silica fume was introduced into the mixture having the optimum mixture proportions (a/w=4:1; the temperature of water is 70°C) in the quantities of 5, 10, 15 and 20% by mass of the ash. Both unground and ground ashes were used, their fineness being 245 and 440 m2/kg, respectively. The data on the compressive strength of concrete are given in Fig.3.

It is evident that concrete containing 10% silica fume had the highest strength. The strength of the concrete containing ground ash was higher than that of concrete with unground ash.

The next stage of the tests was to determine the maximum amount of an aggregate (slag sand) in a cementless concrete mixture based on ground ash and containing 10% silica fume (Fig.4).

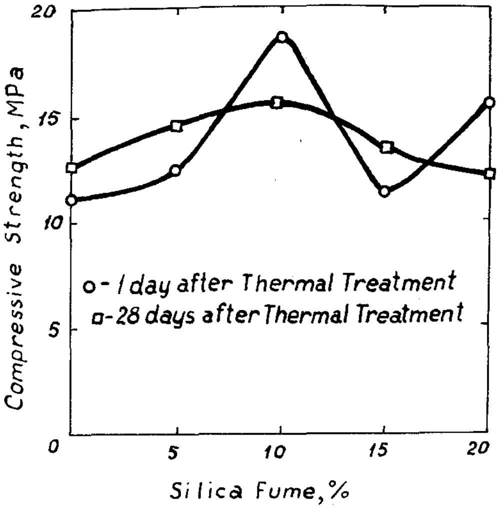

Fig.3. Compressive strength of cementless ash concrete versus the amount of silica fume

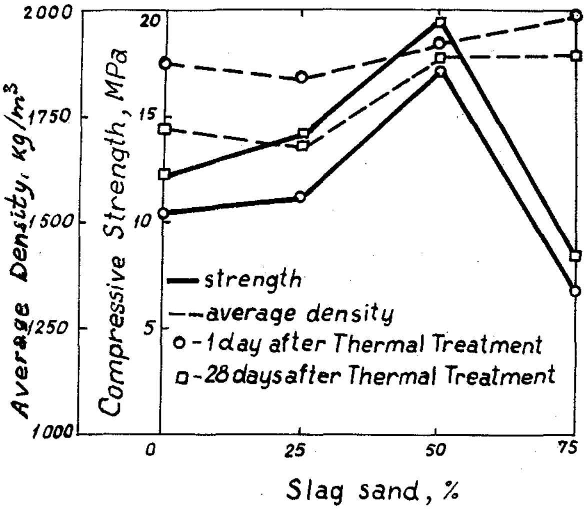

Fig.4. Compressive strength of cementless ash concrete versus the amount of slag sand

As can be observed from the data given in Fig.4, the strength of concrete increased with the introduction of up to 50% slag sand into the ash concrete mixture containing 10% silica fume.

After processing the results of the study by a computer, the optimum mixture proportions for a cementless fine-grained ash-slag concrete were developed (Table 3).

Table 3. Optimum mixture proportions

	Quantities, (kg/m3)				Mixture slump (cm)	Average density (kg/m3)	Compressive strength (MPa)
	Fly ash	Silica fume	Slag sand	Water			
For load-bearing structures	650	70	720	480	8-10	1920	15-20

To improve properties of the concrete developed, to predict its service life, to eliminate the reaction between free calcium oxide and water and to prevent expansion of the concrete, the effect of silica fume on the process of binding free calcium oxide in the concrete was studied. Some procedures were performed to assist the process of binding free CaO by the amorphous silica and converting the free CaO into 2CaO. SiO2 and 3CaO. SiO2 or, in the presence of water and heat, into hydrosilicates. They were as follows: grinding of ash which destroyed agglomerated particles and fused cover of the ash, using hot water for mixing concrete and subsequent heating of the concrete after its placing using a 3+9+3 h cycle (Pavlenko 1995).

Electron microscopy, X-ray diffractometry, chemical and differential thermal analysis were used to identify the reaction between silica fume and free CaO. The election microscopy revealed this reaction (Fig. 5, 6).

Fig. 5. Reaction of hydration between free CaO and silicon dioxide. Electron micrograph x 14600. At 24 h.

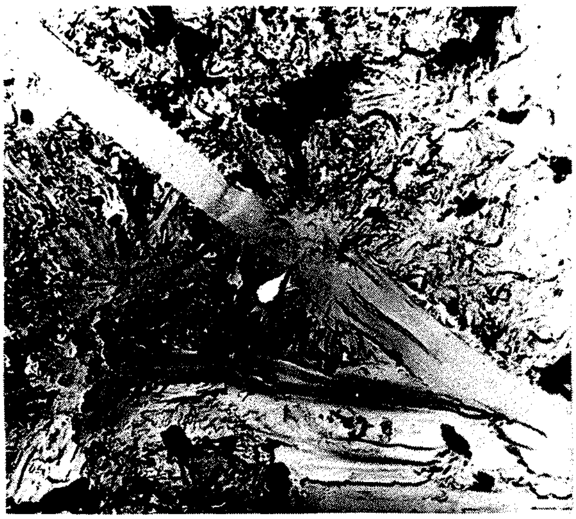

Fig. 6. Large crystals of calcium hydrosilicate. Electron micrograph x 12400. At 180 days.

X-ray analysis of the materials and concrete also indicated that the formation of hydrosilicates of calcium occured not only due to the hydration of minerals but also due to binding free CaO by silicon oxides and their interaction with water.

If we take the free CaO content in fresh concrete mixture as 100% (free CaO will be equal to 2.78% of the total mixture), its content will amount to 0.69, 0.51, 0.32 and 0.28% at 24 hours, 28, 90 and 180 days, respectively.

Differential thermal analysis and dynamic weighing of specimens were used to determine the degree of ash hydration, the amount of new formation being defined by loss in weight.

RESULTS

The reduction of free calcium oxide in the cementless ash-slag concrete from 2.78 to 0.28% due to the reaction between CaO and silica fume increased its strength by 20%. The use of slag sand resulted in a 60-7-% increase in the concrete strength. The expansion of concrete in the presence of free lime was eliminated.

The concrete developed was used for the production of small blocks at Abakanskaya thermal power plant and produced satisfactory performance.

CONCLUSIONS

The study of the effect of silica-fume on diminishing risk of expansion due to hydration of free lime in cementless fine-grained concrete confirmed good performance of the concrete consisting entirely of industrial waste products (high-calcium ash, slag sand and silica fume) prepared with hot water (60-80°C) and after its curing at 80-90°C.

The concrete developed and technology of its preparation, placing and heat treatment may be used for the production of bricks, small blocks and also for the construction of one-, two-storey cast in-situ houses.

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INLUENCE OF AGGREGATES AND MINERAL ADDITIVES ON THE COMPOSITION OF THE PORE SOLUTION

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ABSTRACT

Four different mineral additives (granulated blast furnace slag, silica fume, synthetic cristobalite and sericite schist) were added, as partial replacement for fine aggregate, to mortars made with sands of various mineralogical compositions (non reactive quartz, reactive quartzite and opal). The pore solution was extracted by the high pressure extraction method and the evolution of concentrations of OH^-, $SO4^{--}$, K^+, Na^+, Ca^{++} and silicate ions, measured at 7, 28, 60 and 90 days, was examined.

Opal was observed to lead to a very low alkali and hydroxyl ion level, indicating a very rapid consuption of alkalies to form a gel. In the case of quartz and quartzite, the alkali level was found to increase rapidly giving a peak at early age, than to decrease and finally to stabilize at values somewhat lower than that in the neat cement paste.

Keywords: ASR, pore solution, mineral admixtures

INTRODUCTION

Alkali-silica reaction (ASR) is a heterogeneous reaction between a solid phase represented by aggregates containing reactive species of silica and a liquid phase represented by the alkaline concrete pore solution. Apart from the ambient conditions (relative humidity, temperature...), the occurence of ASR depends mainly on the ratio between the reactive silica and the Na_2O equivalent content of the surrounding solution. There is a pessimum ratio around 4.5 at which the observed expansion is maximum (Dent Glasser & Kataoka 1981, Wen. 1989, Barret *et al.* 1992).

Mineral additives such as granulated blast furnace slag, pozzolanic fly ash, condensed silica fume and natural pozzolans can be used to limit the risk of ASR (Duchesne & Bérubé. 1992, 1994). These additives not only act by a simple dilution effect, but also by their ability to combine the alkaline oxides and thus reduce the OH^- concentration in the pore solution. Their use in concrete can also lead to a finer capillary pore structure and a lower diffusivity and permeability. The result is usually a reduction of the observed expansion and deterioration.

EXPERIMENTAL

Materials

The chemical, physical, and mineralogical characteristics of the cement (OPC), the fine aggregates and the mineral additives are presented in tables 1, 2a and 2b.

Table 1 : Chemical composition of materials

Weight %	OPC	Qs	Q	O	c	gbfs	s	csf
L.O.I.	0,9	0,1	3,1	6,8	0,2	1	3,5	2,6
SiO_2	19,9	99,1	90,5	92	99	35	57,4	95,1
Al_2O_3	4,4	0,6	1,4	0,7	0,2	15	19,8	0,2
Fe_2O_3	2,4	0,1	0,3	0,2	0,1	1	7,7	0,3
CaO	62,3	0,1	3,4	0,1	0,5	40	0,9	0,9
MgO	4,7	-	0,1	0,1	0,1	4,5	2,9	0,4
SO_3	3,6	-	-	-	-	0,2	0,2	0,3
Na_2O	0,3	-	0,3	-	0,1	0,5	1,6	0,2
K_2O	1,1	-	0,8	-	-	1	4,1	0,6
TiO_2	0,2	-	-	0,1	-	0,5	0,8	-
MnO	-	-	-	-	-	0,5	0,1	-

Qs = quartz sand, Q = quartzite, O = opal, c = synthetic cristobalite, gbfs = ganulated blast furnace slag, s = sericit schist, csf = condensed silica fume

Table 2a : mineralogical and physical characteristics of cement and mineral admixtures

	Material	Mineralogical and physical
CEMENT	OPC	Class CEM I according to the french standard NF P15-301 C3S = 59.6 ; C2S = 12.7 ; C3A = 7.6 ; C4AF = 7.4 $Na_2O + 0.658\ K_2O = 1.2$ Fineness =350 m^2/kg
ADMIXTURES	Slag	Granulated blast furnace slag Glass > 95 % Fineness =280 m^2/kg
	Silica fume	Condensed silica fume Specific area (BET) = 22 m^2/g
	Cristobalite	Obtained from burning of silex Contains 85 to 90 % of cristobalite Fineness = 300 m^2/kg ; average grain size = 25 μm
	Sericit schist	Ground natural rock containing 25% quartz, 14%albite, 42% illite, 14 % chloriteand minor minerals (calcite, magnetite, apatite...) Fineness = 300 m^2/kg, average grain size = 25 μm

Table 2b : mineralogical and physical characteristics of the sands

SANDS	Quartz sand	Natural well crystallized quartz sand. Non reactive SiO2 = 99 % Granarity : 250-600 μm
	Quartzite	Strained quartz with microquartz and phyllit at the grain boundaries Potentially reactive rock Granularity : 250-600 μm
	Opal	Amorphous silica with some quartz Potentially reactive rock with pessimum effect Granularity : 250-600 μm

Preparation and curing of mortars

The composition of the micro-mortars is given in table 3.

Table 3 : composition of mortars

Materials	Standard mortar	Mortar with additive
OPC	774 kg/m^3	774 kg/m^3
Sand (250-600μm)	774 kg/m^3	694 kg/m^3
Mineral additive	-	80 kg/m^3
W/C	0,6	0,6

Five mortars were prepared with each of these two fine aggregates: four were made by replacing 10% by weight of the fine aggregate with a different mineral additive (granulated blast furnace slag (gbfs), condensed silica fume (csf), synthetic cristobalite (c) and natural sericit schist (s)) and one was made without any mineral additive. This fine aggregate replacement procedure was selected in order to keep the amount of cement (which is the main source of alkalies) constant. Pure cement paste and a series of five mortars made with a non reactive quartz sand (Qs) were also prepared as reference mixtures. The selected water/cement ratio was relatively high in order to facilitate the pore solution extraction.

For each of the 15 micro-mortars, three cylinders (h = 9 cm, Ø = 4.5 cm) were prepared using the following procedure: homogeneization of the dry materials, addition of the required amount of water, energetic mixing (10 000 rpm), filling up the moulds under mechanical vibration to avoid heterogeneity and bleeding. The moulds were sealed and kept for 24 hours at 20°C and 95% R.H. After demoulding, the mortar specimens were stored at 38°C and 100% R.H.

Extraction of the pore solution and chemical analysis

The pore solution of the mortars was extracted by the high pressure extraction method (Longuet *et al.* 1973) at 7, 14, 28, 60 and 90 days. About 4 to 12 ml of liquid were collected and immediately neutralized. The chemical analysis was carried out by ICP and ionic chromatography.

TEST RESULTS

Evolution of pore solution composition

The results of the pore solution analyses are illustrated in Figures 1 to 7.

With regard to the evolution of the OH^- concentration (Fig.1), the mortars can be clearly divided into three groups :

- the first group includes all mortars prepared with the non-reactive quartz and the reactive quartzite, without silica fume. In this group the OH^- concentrations are the highest. They vary between 500 to 600 mmoles/l, i.e. close to the concentration observed in the neat cement paste. These results thus show that the slag, the synthetic cristobalite or the sericit schist did not significantly reduce the OH^- concentration;
- the second group includes the mortars containing silica fume and made either with the non-reactive quartz or the reactive quartzite. For these mortars, the OH^- concentration lies between 250 and 350 mmoles/l, which is still rather high probably because of the alkali content of the csf itself. Nevertheless these results confirm the well known effect of silica fume in reducing the alkalinity of the pore solution;
- the third group includes all mortars prepared with the opal aggregate for which the OH^- concentration decreases from about 180 mmoles/l at 7 days to 75 mmoles/l at 90 days. In these mortars, the reaction with the opal aggregate seems to be the prevailing one and there is no apparent influence of any of the mineral additives with respect to the OH^- level. Nevertheless complementary expansion tests carried out on the opal mortars have shown an obvious reduction of expansion with csf when compared with the other additions, indicating that, in fact, there is a rapid consumption of OH^- by the csf.

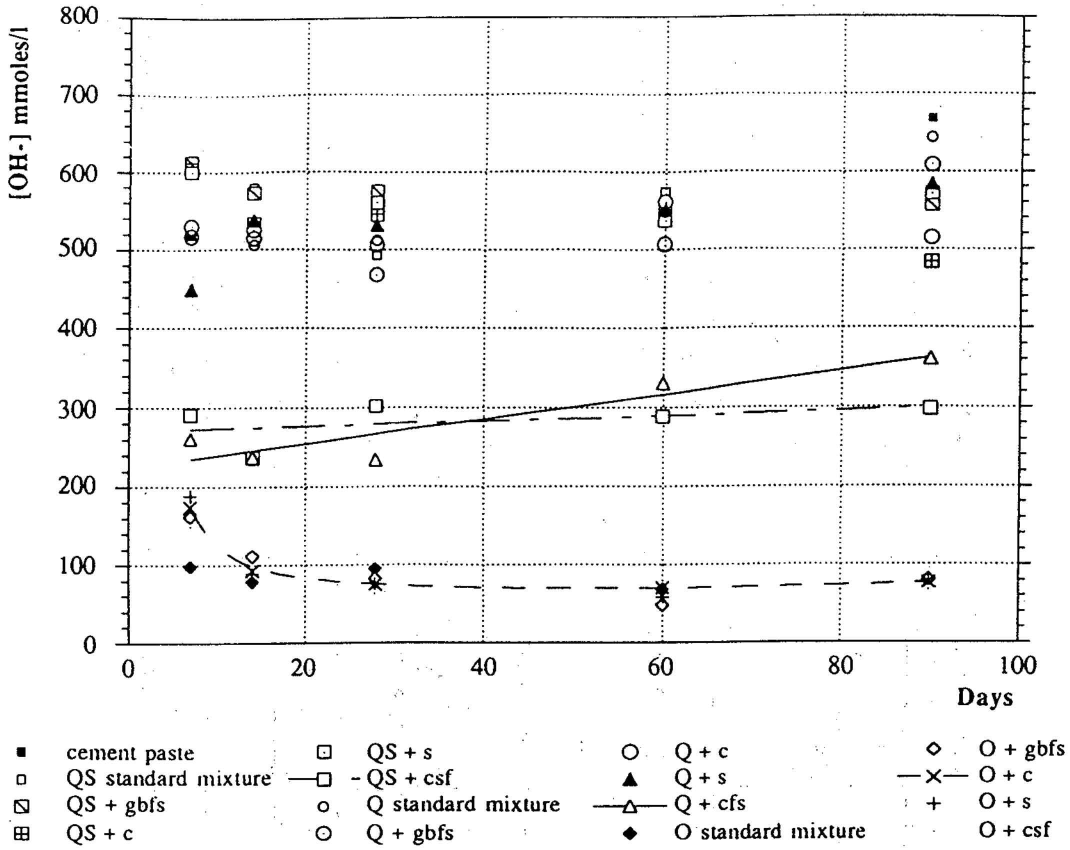

Figure 1. Evolution of OH$^-$ concentration versus time

With regard to the evolution of alkaline oxides in the pore solution (Figures 2 to 7), four groups can be distinguished:

1) neat cement paste which shows a normal increase of the alkali concentration between 7 and 90 days. The value rises from 100 to 330 mmoles/l which is quite normal.

2) all mortars prepared with the non-reactive quartz (with the exception of that containing silica fume) which exhibit a very characteristic peak of concentration at 14 days. At this point in time, the alkali concentration ranges between 400 and 500 mmoles/l and is much higher than that of all the other mixtures including neat cement paste. The concentration drops to approximately 200 mmoles/l at 28 days, close to the value for the quartzite mortars.

3) all mortars prepared with the reactive quartzite. These mixtures do not exhibit a characteristic concentration peak at 14 days, but there appears to be a slightly higher value at 7 days, which would tend to show that there is a peak before 7 days. The concentration decreases slowly from about 260 mmoles/l at 7 days (a value also much higher than that in cement paste at 7 days, 100 mmoles/l) to 200 mmoles/l at 90 days. In the presence of silica fume, the concentration decreases from 150 to 100 mmoles/l.

4) all mortars prepared with the opal aggregate for which the concentration is very low and approximately constant between 7 and 90 days at a mean value of about 20 mmoles/l.

The concentration of SiO_2 which is normally low in cement systems with high calcium content, was also determined. It was found to be in all cases less than 1 mmole/l, even in the presence of silica fume and opal, despite the obviously high amount of silica that is disssolved, particularly in the mixtures containing opal. The small amount of silica in the pore solution is due to the presence of calcium which leads to the precipitation of a siliceous calco-alkaline gel. It results from a dynamic equilibrium between the dissolution and the precipitation processes. Work by Thuret *et al.* (1992) has shown that in the absence of calcium, the silica concentration in a 1N NaOH solution can be about 1.6 moles/l after 1 day, but that the addition of a diluted $Ca(OH)_2$ solution leads to gelification.

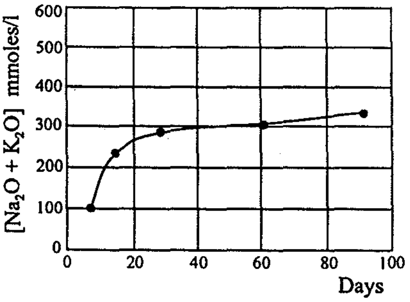

Figure 2. Neat cement paste.

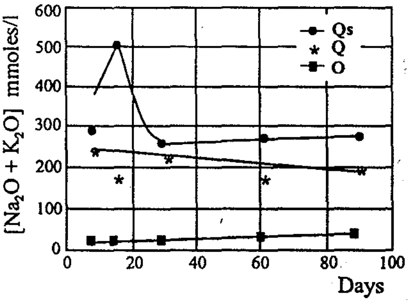

Figure 3. Mortars without mineral admixture.

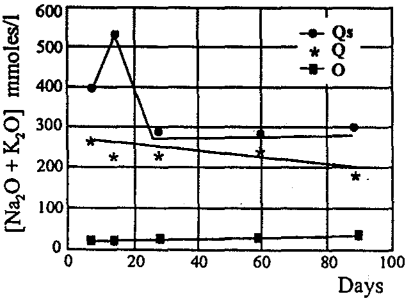

Figure 4. Mortars containing blast furnace slag.

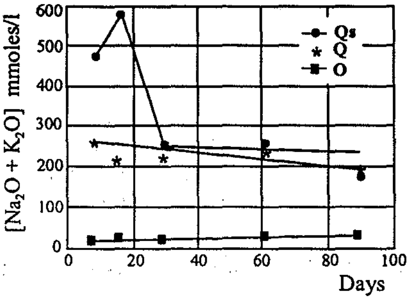

Figure 5. Mortars containing synthetic cristobalite.

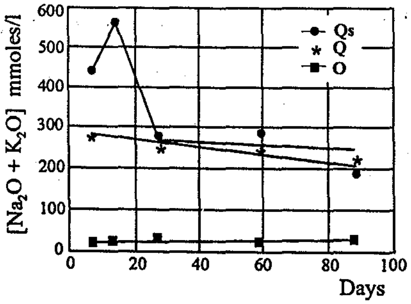

Figure 6. Mortars containing sericit schist.

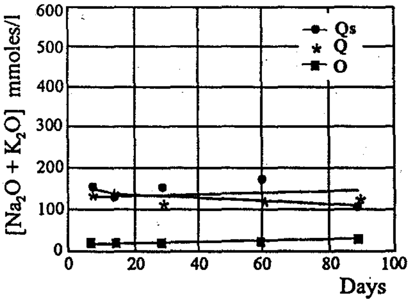

Figure 7. Mortars containing silica fume.

The CaO concentration was observed to be approximately constant at around 1 mmole/l for the cement paste and for all mortars prepared with the non-reactive quartz and the reactive quartzite. With opal, the concentration was higher at a value around 10 mmoles/l, except for the mixture containing silica fume where the values during the 90 day period varied between 3 to 5 mmoles/l. The reason for this is the lower alkali concentration which allows a higher amount of Ca^{++} in the pore solution, the solubility of calcium being very low in the presence of alkalies. Figure 8 illustrates the variation of the CaO concentration with the composition of the mixtures which can be divided into three groups :

- first group : cement paste, quartz sand and quartzite mortars ;
- second group : opal mortars with silica fume ;
- third group : opal mortars containing slag, sericitschist and cristobalite and opal mortars without addition.

The difference between the second and the third group is probably due to puzzolanic activity of silica fume wich partly consume the free calcium ions.

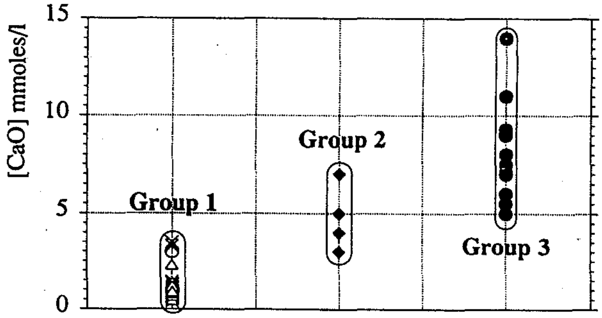

Figure 8 CaO concentration of the pore solution in the three groups of mortars

The opposite trend is observed with the concentration of the SO_4^{--} ions which clearly decreases in the presence of opal from an average value of 10 to 18 mmoles/l to about 0.2 to 1 mmole/l. Also due to solubility equilibrium, precipitation of SO_4^{--} within the solid phases is increased.

As shown by Shayan (Shayan *et al.* 1993) ettringite formation is promoted by silica fume addition. Due to their ability to consume alkalis which leads to higher Ca^{++} concentration in the pore solution, opal aggregates can act in the same way. Higher Ca^{++} concentration reduces ettringite solubility which in turn leads to a decrease of SO_4^{--} concentration in the pore solution.

DISCUSSION

In a recent publication on the mathematical modeling of ASR (Sellier *et al.* 1995), it was explained that the global kinetics of ASR is governed by the diffusion rate of the alkalies either in the reactive silica or in the mortar matrix, and that the chemical process ends when the diffusion of alkalies towards the reaction sites is no longer possible (generally when all the alkalies have been chemically bound in the silica gel). It was further shown in this publication, by mathematical calculations, that the probability of the presence of alkalies at a given reactive site increases rapidly at first, due to the slow diffusion of the alkalies in the silica, and then rapidly decreases as the alkalies diffusing from the mortar to the reaction site are chemically combined in the gel being formed.

This phenomenon can be linked to the variation of the amount of alkalies in the pore solution for the mixtures prepared with the quartz aggregate (Figures 3 to 6) which shows a similar trend of a rapid increase immediately followed by a decrease (thus indicating that, even with innocuous quartz, some reaction with the alkalies is occurring). The same type of phenomenon should of course also be observed for the reactive aggregates, but the peak is expected to occur earlier, probably in the very first minutes or hours for the very reactive opal, and during the first few days for the quartzite which is less reactive than opal but more than quartz. It should be noted in this respect that, as previously indicated, there seems to be a higher concentration of alkalies at 7 days in at least three of the mixtures containing quartzite (Figures 3 to 5). Furthermore, it is clear from the test results that the presence of aggregates (such as quartz and quartzite) tends to cause an increase in the alkali content of the paste in the first days of hydration, which again shows (as the model briefly described indicates) the influence that aggregates can have on the kinetics of the processes involved.

CONCLUSION

Tests were performed to determine the compositon of the pore solution in mortars prepared with three different fine aggregates (non reactive quartz, reactive quartzite and opal) and four different mineral additives (granulated blast furnace slag, condensed silica fume, synthetic cristobalite and sericit schist). The composition of the pore solution was found to be influenced both by the type of aggregates and the mineral additives.

Opal was observed to lead to a very low alkali and hydroxyl ion level, indicating a very rapid consumption of alkalies to form a gel. In the mortars made with quartz, and perhaps also with quartzite (although, in this case, more tests with measurements made at early ages will be required to investigate the phenomenon), the alkali level was found to increase rapidly at first (to values higher than in the neat cement paste), then to decrease, and finally to stabilize at values somewhat lower than that in the neat paste. This phenomenon shows that, despite its innocuity, the quartz consumes some of the alkalies. It also indicates that the dissolution kinetics is linked to the kinetics of the gel forming reaction.

Under the particular test conditions that were used, slag, cristobalite and sericit schist had no influence on the pore solution composition, probably due to the low replacement level in the case of slag and to poor reactivity in the case of synthetic cristobalite and sericit schist. Silica fume, however, had a strong influence, its use causing a significant decrease of alkali and hydroxyl ions in the solution.

Limiting ASR by the use of mineral admixtures is a complex problem in which different parameters must be taken into account, and, as shown in this paper, particularly the reaction kinetics of the aggregates and of the mineral additives. Further investigation using both laboratory tests and mathematical calculations (with models such as that proposed by Selier *et al.*) should allow a better understanding of the phenomenon.

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REVIEW OF METHODS USED AT HYDRO-QUÉBEC TO PREVENT ALKALI-AGGREGATE REACTIONS IN CONCRETE STRUCTURES

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ABSTRACT

The methods currently used at Hydro-Québec to prevent alkali-aggregate reactions (AAR) are generally derived from recommendations from the Canadian standards CSA A23.1 "Concrete Materials and Methods of Concrete Construction" and CSA A23.2 "Methods of Test for Concrete". These methods can be outlined as follows:

- Selection of non reactive aggregates (selective quarrying or aggregate beneficiation/dilution)
- Control of alkali content per cubic metre of concrete
- Use of mineral admixtures

This paper summarizes alkali-aggregate reactions and their background, as well as describing the methods of prevention used at Hydro-Québec. It also presents a case where silica fume has been used to prevent AAR in a new dam.
Keywords: Alkali-silica reaction, mineral admixtures, silica fume.

INTRODUCTION

Hydro-Québec is the largest energy producer in Québec operating on a territory of 1.5 km^2 and servicing 2,900,000 customers including 13,000 industrial accounts. It owns and operates more than 480 dams and 54 hydroelectric power plants, the most powerful being LG-2 with 5,328 MW. The total capacity is 30,434 MW and total energy produced per year is 142,778 GWh (1994).

By the year 2000 some 150 of Hydro-Québec's concrete dams will actually be at least 40 years old; about 100 of them will be 60 years old. The present condition of these structures reflects the effect of numerous factors that have brought on aging, premature or not. These factors include initial shrinkage causing latent microcracking, expansion and contraction tied to temperature cycles, larger infiltration entailing increased uplift, leaching and deposition of calcium salts that clog joints and drains, hydric swelling, freezing and thawing cycles, erosion, sulphatation, and alkali-aggregate reactions (AAR) (H.Q. 1993).

While the present condition of a dam often reflects a combination of several of these factors, deterioration caused by swelling due strictly to AAR may be critical. The well-known case of the Beauharnois hydroelectric development is a prime example of significant swelling due to AAR (Albert and Raphaël. 1987 and H.Q. 1992). Unfortunately, the Beauharnois structure is not Hydro-Québec's only dam where AAR has been discovered. Bérard and Roux (1986) name more than 20 dams where products of AAR are in evidence. A more recent study reports some 30% of Hydro-Québec's concrete dams are affected by AAR and 6% show no sign of reactivity. Because their condition is judged satisfactory, the remaining 64% have not been investigated in any great detail.

As these statistics may cause concern, it must be emphasized that not one concrete structure anywhere in the world has collapsed because of AAR alone[1] although, a few have been demolished by way of prevention (Hobbs. 1988). There are, however, several cases of Hydro-Québec concrete structures that have undergone work to thwart the effects of concrete swelling due to AAR or other factors. In some cases this has meant major rehabilitation to remove part of the old concrete and apply a layer of new concrete. This paper outlines the background of the AAR phenomenon and provides an overview of the preventive measures currently used at Hydro-Québec.

ALKALI-AGGREGATE REACTIONS

Description

Concrete consists for the most part of Portland cement, aggregates (stone and sand) and water. In hardened concrete, the hardened cement paste contains voids that are filled to some extent with a very basic alkaline liquid known as pore solution. Some mineral phases are chemically unstable in the presence of this solution and, when given conditions prevail, react to form products that occupy more space than the original phases. This results in concrete swelling, the phenomenon is known as alkali-aggregate reaction (AAR).

The details of the chemical reaction are well covered in the literature. The three following conditions are essential for AAR to occur:

- The aggregate must be alkali-reactive.
- Concrete made with reactive aggregates must have a minimum alkali content.
- The concrete must be exposed to an average relative humidity exceeding 75%.

When these three conditions are satisfied, an alkali-aggregate reaction occurs after a certain period of time. While it is generally more than five years before cracking sets in, the AAR may continue over several decades once it has started.

Alkali-silica reaction - Background

The first discoveries regarding the alkali reactivity of certain mineral components go back as far as 1916, when E.A. Stephenson, a geologist, reported that the reaction between feldspar and sodium carbonate produces a heavy gelatinous precipitate gel (Meissner. 1941). In 1923, J.C. Pearson and G.F. Loughlin, aggregate researchers, identified "dangerous aggregates" with regard to the performance of concrete (Blanks. 1941). In 1935, Professor R.J. Holden, F.M. Lea and C.H. Desch provided an initial understanding of how gels form in the alkali-aggregate interaction process (Leps. 1995). A little later, the American T.E. Stanton (1940), a materials and research engineer, California Division of Highways, observed concrete swelling and cracking in Parker dam (California) and attributed them to chemical reactivity of the aggregates. Since then, several American dams have been reported to be affected by AAR (Parker, Stewart Mountain, Gene Wash, Copper Basin, Buck, American Falls, Coolidge, Owyhee, Hiwassee, Chickamauga and Fontana dams) (Leps. 1995). In Canada, it was not until 1953 that E.G. Swenson identified the first case of AAR, in the concrete of a Montréal bridge (Grattan-Bellew. 1992). Cases of structures damaged by alkali-silica reaction have since been reported practically wherever concrete is

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[1] There is nevertheless a California pipeline that collapsed because of AAR. (Haavik and Mielenz 1991)

used. The physicochemical mechanisms that enter into these reactions have been extensively investigated, and much progress has been made in elucidating these phenomena.

The alkali-silica reaction is the only type of AAR to occur in Québec, although with a wide variety of rocks belonging to three major geological families are present in the province -- Appalachians, St. Lawrence Lowlands and Precambrian. The Precambrian Shield north of the St. Lawrence Lowlands includes granitoid rocks containing fairly deformed quartz of igneous or metamorphic origin. This family of rocks usually exhibits a slow/late-expanding reaction characteristic of the sub-category of slow or late-expanding alkali-silica/silicate reaction.

PREVENTION AND REPAIR METHODS USED TO INHIBIT AAR

Methods to prevent AAR

In the past, Hydro-Québec technical specifications did not all prescribe special measures for dealing with AAR, but over the last ten years this has become common practice for hydraulic structures (Pedneault et al. 1992). In the case of small-scale projects involving small structures such as insulator foundations, however, it is not always possible to obtain certification of the aggregates from the concrete suppliers, which means that only sometimes were engineers able to choose suppliers who are using aggregates known to be nonreactive. Engineers can now refer to a list of quarries classified in terms of alkali reactivity by the Québec Transportation Ministry (Tremblay and Vézina. 1993).

The most recent specifications by Hydro-Québec refer to Canadian standards CSA A23.1 "Concrete Materials and Methods of Concrete Construction" and CSA A23.2 "Methods of Test for Concrete" to prevent AAR in concrete.

The proposed approach consists first in evaluating the aggregate in light of available data under real conditions similar to those to which the concrete will be exposed. In the absence of valid field experience with structures ten years old or more or if the aggregate has already shown potential alkali reactivity, the aggregate is rejected or laboratory tests are required. These are mainly expansion tests that produce results within a time ranging from a few days to one year or even two years in some cases.

If the test results point to potential reactivity, the aggregate must be rejected for use in Portland cement concrete. There are three main alternatives:

- Selective quarrying or aggregate beneficiation (dilution).
- Control of alkali concrete per cubic metre of concrete.
- Use of mineral admixtures.

Selective quarrying or aggregate beneficiation

Where possible, dilution of reactive aggregates with nonreactive aggregates in the quarry or selective quarrying of areas of nonreactive rock are methods often used to prevent AAR.

Hydro-Québec's most recent technical specifications usually stipulate aggregates used for concrete must conform to CSA A23, which sets out procedures for selecting concrete aggregates to inhibit AAR. The specifications sometimes go further than the standards, prohibiting use of certain types of aggregates that have exhibited alkali reactivity in Hydro-Québec structures located in the same region as the source of those aggregates (Chouinard. 1993). There is also SEBJ's[2] standard technical specification AA-80N-001 93, which makes mention of Appendix B of CSA A23.1, which states

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[2] SEBJ is a Hydro-Québec subsidiary (Société d'Énergie de la Baie James).

aggregates from each source must be checked for alkali-aggregate reactivity and each aggregate source must be approved by SEBJ.

Control of alkali content per cubic metre of concrete

One recommended means of inhibiting AAR when using reactive aggregates is to control the alkali content of the mixtures, given that there is typically little or no expansion below a certain alkali content in concrete mixtures containing reactive aggregates.

The use of low-alkali cements reduces the alkali content of concrete mixtures. Technically, a low alkali cement has a content less than 0.6% Na_2O equiv. (Na_2O equiv. = Na_2O + 0.658 K_2O). This is the total alkali content of the cement (ASTM C 114-85, atomic absorption or flame spectrophotometry).

SEBJ recommended the use of low-alkali cements for the structure of the James Bay megaproject. Starting in Phase 1, completed in 1982, the maximum alkali content prescribed in the technical specifications was 0.80% Na_2O equiv. For Phase 2 (1982-1996), this was reduced to a maximum of 0.70% and an average below 0.65%. The decision was aimed at preventing any risk whatever of alkali reactivity of the rocks used at James Bay. The rocks in question were granitoids containing fairly deformed quartz and similar to those classed as slightly reactive in some old dams (Rapides Farmers and Chelsea).

The new 1994 version CSA A23.1 (Appendix B) recommends a 3.0 kg/m^3 limit as a means of preventing AAR. Caution will be urged regarding mass concrete since problems have already occurred with alkali contents as low as 2.0 kg/m^3. The same is true for highly reactive aggregates or structures exposed to external sources of alkali, such as de-icing salt or sea water.

Most concretes poured thus far at James Bay probably have alkali contents below this 3.0 kg/m^3 limit, that is to say, about 2.5 kg/m^3. Evidently, simply limiting the alkali content of cement was sufficient to control the alkali reactivity of the aggregates used at James Bay, since no case of reactivity has been identified to date[3] (Verville. 1993).

Use of mineral admixtures

CSA A23.1-M90 states that silica fumes, fly ash, ground granulated blast-furnace slag and pozzolan used in sufficient quantities can prevent or reduce swelling due to alkali-silica reaction or the slow/late expansion such reactions produce. The new 1994 version of this standard is more explicit, mentioning several Canadian structures where mineral admixtures have been used to limit expansion due to AAR. One example is the Lower Notch dam in Northern Ontario, built with a reactive argillite (Sturrup et al. 1983) and 20% fly ash. No deleterious expansion has been observed after some 20 years (Hooton. 1990). In Britain, some dams built more than 50 years ago with reactive aggregates and fly ash are still in excellent condition (Thomas. 1992).

Here .again, the standard advises caution, particularly about tests to evaluate the effectiveness of mineral admixtures. It mentions that no test is needed if the concrete to be used contains less than 3.0 kg/m^3 alkalies and, further, that no additive is effective against alkali-carbonate reaction.

Hydro-Québec for the first time used at Témiscouata dam a mineral admixture to specifically inhibit alkali reactivity in a hydraulic structure. More specifically, this dam,

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3. The oldest structures at James Bay are about 18 years old.

rebuilt in 1993-1994, probably marks the utility's first utilization of condensed silica fume to inhibit aggregate reactivity. The case history of this dam is presented in the next section.

SEBJs technical specifications state use of silica fumes, pozzolan and calcium carbonates is prohibited, except as authorized in writing by SEBJ. As mentioned earlier, the preferred approach at James Bay was systematic selection of aggregates and use of low-alkali cements (Verville. 1993).

Témiscouata dam (Blanchette, Massad and Nadon 1992)

The Témiscouata dam was built in 1933 near Dégelis in Témiscouata, about 70 km southeast of Rivière-du-Loup. It includes a 188-m spillway equipped with 24 gates and a 153-m earthfill dam on the right bank, for a total length of 341 m. This structure regulates the flow of Madawaska River, a tributary of Saint-Jean River, which in turn supplies several power stations in New Brunswick.

The coarse and fine aggregates in the concrete of this development are of natural origin and consist of sedimentary rock, including pelites, greywackes and sandstone rich in quartz. Both grades of aggregates exhibit alkali reactivity resulting in slow concrete swelling followed by polygonal microcracking. Cracks are more extensive in emerged areas, such as the top of the piers and the deck. The downstream apron is also affected, albeit to a lesser extent.

In the 1960s concrete swelling caused all the piers to move in the direction of the earthfill dam, with the right end of that structure resting against the rock. The total displacement observed at the last pier on the fill side was about 225 mm. The possibility of rebuilding the dam was contemplated at that time. In the final analysis, the service life of the structure was extended by creating joints sealed with two asphalt boards installed every two piers (Munger. 1993). Most of these joints are now partly closed, resulting in extrusion of the bituminous sealer.

A subsequent measure consisted in recasting all of the pier and deck concrete (end of 70s, beginning of 80s). Fine polygonal cracking is now showing in this concrete. After a general review completed in 1993, it was decided to rebuild the Témiscouata development in 1993-1994 (phases 1 and 2).

To prevent AAR-related problems, the choice of aggregates for the new concrete was based on the practices recommended by CSA A23.1. The two local gravel pits were subjected to petrographic analyses, accelerated mortar and concrete prism expansion tests (South African and modified South African tests), concrete prism expansion tests at 38°C and modified chemical test ASTM C289. These tests revealed the potential alkali reactivity of the two aggregates and the sand. The option of using fly ash referred to in CSA A23.1 was considered, then dropped for technical reasons. The final choice involved using the lithic gravel deemed least reactive, together with cement with condensed silica fume (CSF) and a nonreactive sand from the Québec City area. This choice was based on results of accelerated expansion tests on concrete prisms placed in NaOH 1N at 80°C. These tests were conducted for various combinations of sand-aggregate-CSF cement (modified South African test) (Blanchette. 1989).

SUMMARY AND CONCLUSION

The methods used by Hydro-Québec to prevent AAR are in line with the main recommendations of Canadian standards CSA A23.1 "Concrete Materials and Methods of Concrete Construction" (Clause 5.5 and Appendix B) and CSA A23.2 "Methods of Test for Concrete," namely:

- Selection of nonreactive aggregates (selective quarrying or aggregate beneficiation/dilution).
- Control of alkali content per cubic metre of concrete.
- Use of mineral admixtures.

Over the past ten years Hydro-Québec has consistently tightened its technical specifications for the selection of concrete aggregates, at least for its larger structures (Chouinard. 1993). There is some room for improvement as regards smaller structures, e.g., concrete requirements for insulator foundations in distribution substations have not always taken account of the AAR factor, until recently in any case.

The list of Québec quarries classed by alkali reactivity (Tremblay and Vézina. 1993) would greatly help Hydro-Québec's engineers, specialists and technicians in selecting concrete aggregates.

In addition to selection of nonreactive aggregates and use of low-alkali cement, Hydro-Québec has used a mineral admixture to counter AAR at the Témiscouata dam rebuilt in 1993-1994. This was the first time Hydro-Québec used condensed silica fume to prevent AAR in a water-resources facility where selected aggregates have a slight potential alkali reactivity. The concrete behavior is monitored by Hydro-Québec by conducting expansion laboratory tests to validate this choice and correlate laboratory testing and structure behavior.

Mineral admixtures are not permitted for James Bay structures, unless specifically authorized by SEBJ. The preferred approach is selection of nonreactive aggregates and use of cements with an average alkali content of 0.65%.

Based on a general impression gained through conversations with Hydro-Québec engineers, not many measures have specifically addressed AAR to date, except perhaps for the Beauharnois dam and few other dams. In coming years, however, Hydro-Québec will carry out several major rehabilitation projects on its structures, making it essential for those in charge to use the most recent guides and specifications to cope with the AAR problem. Concurrently, research should continue so as to:

- Further elucidate the mechanisms of AAR;
- Improve quantification of the effects of AAR on concrete structures, including mathematical modeling;
- Improve prevention and repair methods for structures affected by AAR.

As Québec's leading user of concrete, Hydro-Québec has to remain at the leading edge of concrete technology to incorporate breakthroughs, as they emerge, in as many of its structures as possible.

Over the past ten years, Hydro-Québec has consistently tightened its technical specifications for the selection of concrete aggregates, at least for larger structures. However, there is some room for improvement as regards smaller structures; concrete requirements for electrical apparatus foundations in distribution substations have not always taken AARs and their effects into account, until recently in any case. The 1994 edition of Hydro-Québec standard SN 30.2 "Fourniture et mise en oeuvre du béton" contains specific sections which are either new or updated to reflect recent development in the field of AAR.

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EFFECTIVENESS OF GRANULATED BLASTFURNACE SLAG IN PREVENTING ALKALI-SILICA REACTION

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ABSTRACT

In a programme covering a wide range of mixtures, three portland cements and two ground granulated blastfurnace slags (ggbs) have been used to investigate the relationship between alkali content and ASR expansion. Length changes were monitored, for several years, on concrete prisms made with a reactive natural aggregate; the prisms were moist stored at 20°C and 38°C. Storage at 38°C was found to be an accelerated test which correlated well with storage at 20°C. At 20°C the rate of expansion was some four times slower than at 38°C. Nonetheless, there was very good consistency between the two temperatures in classifying mixtures as either 'expanding' or 'non-expanding'. Current indications are that the magnitude of ultimate exposure is less with 20°C storage than 38°C.

The mixtures containing ggbs, tolerated much greater alkali contents in the concrete, without expansion. This effect was more pronounced for higher proportions of ggbs. The results of the programme are discussed in relation to various rules which have been proposed to take advantage of the effectiveness of ggbs in preventing ASR.

KEYWORDS: Accelerated tests, alkali-silica reactions, expansion, regulations, slag

INTRODUCTION

Ground granulated blastfurnace slag (ggbs) is used as a within-mixer addition or as a component of a factory-blended portland blastfurnace cement. Its ability to prevent Alkali-Silica Reaction is well established worldwide (1). Evidence of the effectiveness of ggbs in preventing ASR has come from numerous laboratory investigations which, almost without exception, have confirmed its ability to prevent deleterious expansion at high levels of slag content. In general the tendency for expansion reduces with increasing ggbs content. Additional and possibly even more convincing evidence, has come from experience of actual structures. Countless structures throughout the world have been constructed using portland blastfurnace slag cement. It is the world's second most commonly used cement type and has been extensively employed for over one hundred years. However, ASR in concrete containing ggbs is virtually unknown to us.

Regulations and advice on the avoidance of ASR vary from country to country (2). In those countries with relevant regulations and where slag cements are available, rules have evolved for the use of ggbs as a preventative measure against ASR. Table 1 summarises these. The common factor of these approaches is that, at slag percentages greater than about 50%, more alkali is permitted in a slag cement than is allowable in a portland cement (typically nearly twice as much).

Recommendations for slag levels lower than about 50% are less well developed.

In the UK, the approach parallels a procedure for normal portland cement concretes, which limits the calculated alkali content of the concrete to less than 3.0kg/m^3. (It should be noted that calculations are based on average alkali contents and statistical fluctuations may lead to actual contents up to about 3.5kg/m^3). For normal portland cement, the full alkali content (expressed as the sodium oxide equivalent) is included in the calculation, but for any slag present, only a proportion of its alkali is included. There are two views in the UK on the appropriate proportion of the slag alkali to include. A Concrete Society Technical Report (3) recommends that only the water-soluble alkali content of the ggbs need be counted as the reactive alkali contribution. The water-soluble alkali in ggbs is normally very small, typically less than 0.05%, and the calculated contribution from this is of little significance. However, Building Research Establishment Digest 330 (4) recommends that the contribution should be one half of the acid-soluble alkali content of the ggbs. The acid-soluble alkali content of ggbs is essentially the total alkali content, and one half of this is significant (typically 0.3%, expressed as sodium oxide equivalent).

The present programme of research was designed to try to resolve this conflict of advice. Expansion measurements are being carried out on concrete prisms containing natural UK aggregates. The chosen aggregates were a combination of flint-bearing sand and crushed limestone coarse aggregate; these were selected because ASR damage in UK structures has occasionally resulted when high-alkali portland cement has been used with such a combination. The following parameters have been varied:

1) Cementitious content	(420kg/m^3 to 1000kg/m^3)
2) percentage ggbs	(0% to 70%)
3) alkali content of portland cement	(0.54% to 1.15%)
4) alkali content of ggbs	(0.58% and 0.83%)
5) added alkali	(0 to 3.8kg/m^3)
6) storage temperature	(20°C and 38°C)

Nearly three hundred specimens, corresponding to 136 different combinations of the above are being monitored.

Since a primary aim of the programme was to test the hypothesis that ggbs contributes one half of its total acid-soluble alkali to the alkali-silica reaction, the matrix of mixture designs was based on this assumption. However, a sufficiently wide range of mixtures was included to enable alternative hypotheses to be tested.

METHOD OF TEST

Prisms (75 x 75 x 285mm) were cast with stainless steel inserts for length measurement. Expansion testing followed the method given in the draft British Standard (5), with the following exceptions and additions:

Two pairs of specimens were cast from each combination; one pair was stored at 20+/- 2°C and the other at 38 +/- 2°C.

The specimens were wrapped in moist towelling or cloth, and then placed inside 500 gauge polythene tubing which was sealed at each end; this was then placed into a protective net and the specimens were stored horizontally.

The specimens were initially measured and weighed at 1 and 28 days. For the first two years, the 38°C specimens were measured at monthly intervals and the 20°C specimens at 3 monthly intervals. Subsequently the time span between measurements has been increased.

The concrete mixtures were designed to produce nominal reactive alkali contents generally either 5, 6, 7 or 8kg/m^3, using 3 combinations of portland cement alkali contents and 2 ggbs alkali contents. These high alkali contents, which resulted in very high cement contents were chosen to insure that at least some of the mixes containing slag expanded which would not have been the case if lower alkali levels were used. It was assumed for the purpose of calculation that 50% of the acid-soluble alkali in the ggbs was available for reaction. Details of the materials are given in Table 2 and the concrete mix proportions are detailed in a previous report (8). Mixtures that required in excess of 1000kg/m^3 of cement used a total cement content of 1000kg/m^3 and potassium sulphate was added to bring the total alkali to the required level. After the programme had commenced, it was found that the measured alkali content of some of the cementitious materials did not coincide exactly with the values originally indicated by the manufacturers. It is for this reason that the actual alkali contents of the mixtures do not always precisely align with the nominal values.

In all the concretes, 25% of the flint-bearing sand (ex Thames Valley) was included by volume of the total aggregate based on a mixture with a cement content of 700kg/m^3. The combination of aggregates chosen has a wide 'pessimum' and it has been separately reported that similar expansions are obtained with between 20 and 40% Thames Valley sand (6) and with between 15 and 35% Thames Valley sand (7).

The results for the mixtures which have shown any significant expansion (greater than 0.05%) have previously been reported (8).

The prisms were visually examined and the crack type described, the crack width was also measured using a crack microscope.

Table 1 - Requirements under which slag cements can be used as precaution against ASR (Equivalent to low-alkali cement)

	% ggbs	Maximum sodium oxide equivalent of cementitious phase	Maximum sodium oxide equivalent of Portland cement component only
Germany	>50	1.1%	-
	>65	2.0%	-
Belgium	>50	0.9%	-
	>50	1.1%	-
France	>60	1.1%	-
	>80	2.0%	-
UK	>50	1.1%	-
Japan	>40	-	0.8%
	>50	-	-
South Africa	>40	-	-

'-' indicates no requirement

Table 2 - Alkali content of cementitious materials

	Sodium oxide equivalent content	
	Nominal	Measured
Portland cement 'H'	1.20%	1.15%
Portland cement 'M'	0.87%	0.87%
Portland cement 'L	0.60%	0.54%
GGBS 'h'	0.87%	0.83%
GGBS 'l'	0.45%	0.58%

DISCUSSION

Crack Measurement and Visual examination

The smallest expansion at which cracking was noted was 0.06 and 0.09 % for the 20°C and 38°C specimen respectively. However while the 20°C specimen exhibited cracking in numerous combinations at below 0.1% expansion only two of the 38°C series exhibited cracking at below 0.1% expansion. The remainder exhibited cracking at above 0.09%. With the exception of where single large cracks were noted, the 38°C specimen exhibited larger crack widths than the 20°C specimen.

Expansion Criterion

An expansion of 0.1% was taken as a limit, above which the specimen was considered to have expanded. If the appearance of cracking is taken as an indication of deterioration, then the value 0.1% is valid of specimens cured at 38°C but may not be valid for specimens cured at 20°C.

In the 38°C specimens the shortest time to reach this expansion was 2 months, the longest time to date is 45 months. In the 20°C specimens, the shortest time to reach 0.1% expansion was 9 months. The specimens were cast over a period of time and are currently between 4.5 and 6 years old.

Effect of Temperature

Storage at 38°C accelerates ASR expansion and is advantageous for obtaining results within a realistic timescale in the laboratory. Fig 1 compares testing at 38°C with results for 20°C, which is more representative of normal ambient temperature.

For the mixtures which have expanded, the age at which they achieve 0.1% expansion at 38°C is plotted against the age to achieve this expansion at 20°C. The points with an arrow are mixtures which have achieved 0.1% at 38°C but not as yet at 20°C.

It can be seen that there is good correlation between 20°C and 38°C; the specimens at 20°C take about four times longer to reach 0.1% than those at 38°C and the

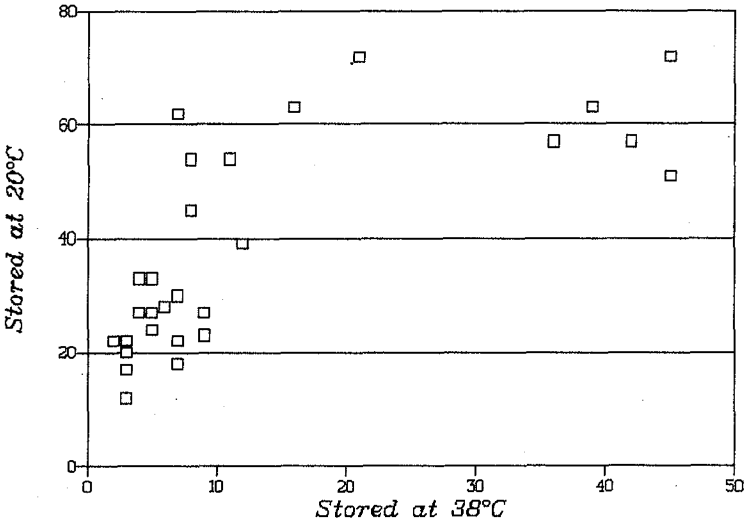

Fig 1 Months to 0.1% expansion

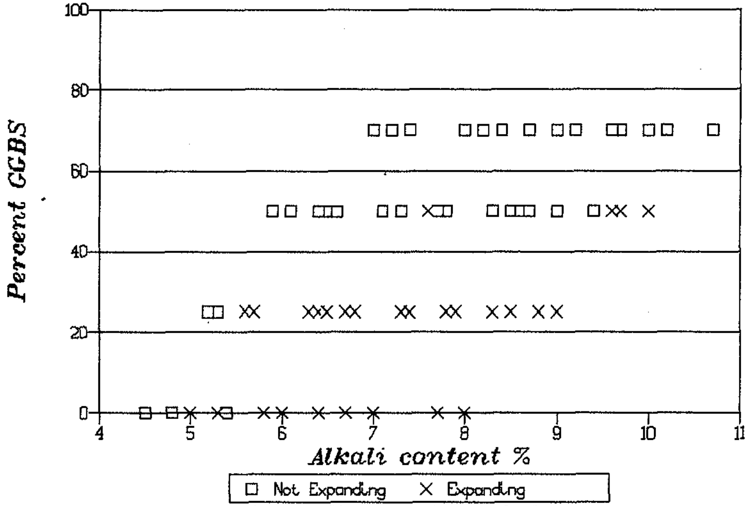

Fig 2 Reactive alkali calculated assuming full contibution from GGBS.

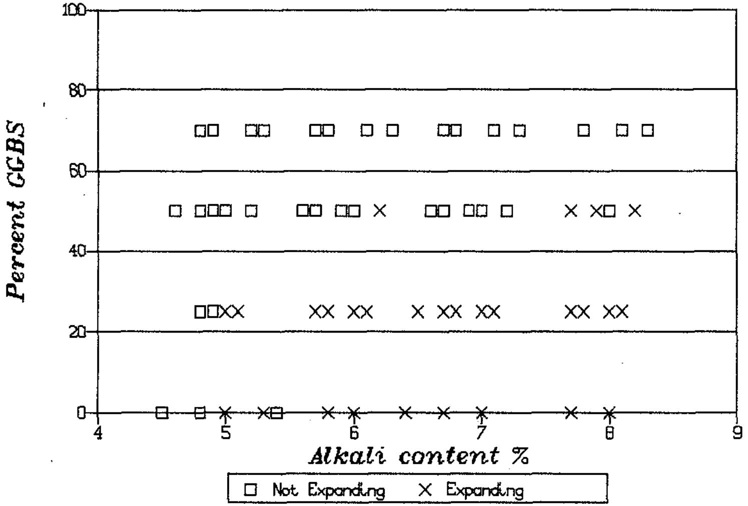

Fig 3 Reactive alkali calculated assuming half contibution from GGBS.

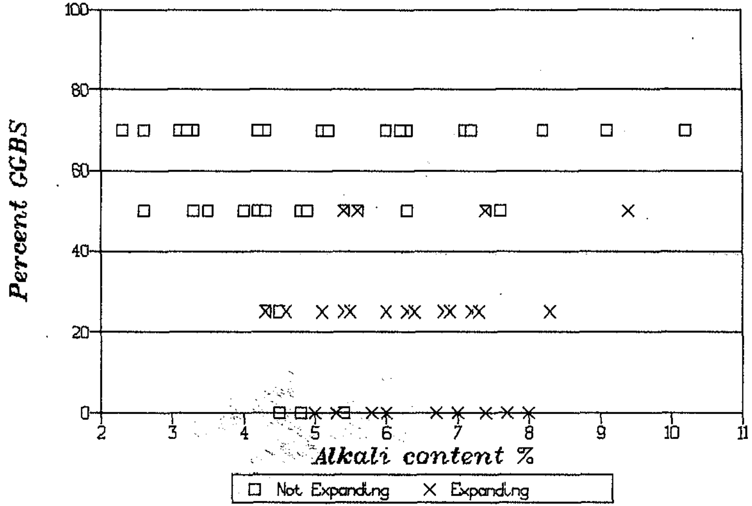

Fig 4 Reactive alkali calculated assuming no contibution from GGBS.

mixtures which have not expanded at 38°C have not expanded at 20°C either.

The magnitude of ultimate expansion varies for the various mixes. However, generally the ultimate expansion at 38°C is greater than that achieved at 20°C.

Cementitious Content and Alkali Level required for Expansion

Of the mixtures which expanded, the one with the lowest cementitious content was a mixture without ggbs, containing 458kg/m^3 of high-alkali (1.15%) cement. It is worthy of note, that in terms of cementitious content, cement alkali and aggregate combination, this mixture is similar to concrete mixtures known to have suffered ASR damage in actual UK structures. For the mixtures without ggbs, the lowest alkali level for expansion was 5.0kg/m^3 under laboratory conditions.

The mixtures with 25% ggbs had increased resistance to expansion. The lowest cementitious content to expand was 595kg/m^3 and the lowest concrete total-alkali content was 5.6kg/m^3. Even greater resistance was shown by the 50% ggbs mixtures. The lowest cementitious content to expand was 856kg/m^3 and the lowest concrete total-alkali content was 8.5kg/m^3. None of the 70% ggbs mixtures have expanded, even with cementitious contents of 1000kg/m^3 and concrete total-alkali contents over 10kg/m^3.

Reactive Proportion of Slag Alkali

The results were used to test the hypothesis that a fixed proportion of the total alkali in the ggbs might be taken as its contribution to the 'reactive' alkali in concrete.

Fig. 2, 3 and 4 are graphs of calculated 'reactive' alkali content of concrete against % ggbs and the squares indicate specimens which have not expanded and the crosses, indicate specimens which have expanded. The 'reactive' alkali content is the sodium oxide equivalent in kg/m^3 and was calculated assuming: Figure 2 : full contribution from ggbs, Figure 3 : 'half' contribution from ggbs, Figure 4 : no contribution from ggbs.

If the 'reactive' alkali content is actually that calculated, then it should be possible to draw a straight vertical line separating the expanding from the non-expanding specimens . This is not possible on any of the three graphs which suggests that the concept of taking a fixed proportion of the alkali in the ggbs as reactive, is not valid. The assumption of a full contribution from the ggbs underestimates the effectiveness of ggbs in all cases. The assumption of no-contribution from the ggbs underestimates the effectiveness at ggbs levels above 35%; at 25% there can be a slight overestimate.

However, it should be noted that this slight overestimate is of the same order as the reproducibility in measuring the alkali content of OPC combined with the uncertainty in defining the critical alkali content for the normal portland cement mixes.

Benefits of ggbs

As has been discussed elsewhere (6), it is unlikely that the benefits of ggbs are simply due to a restricted availability of its alkali. Indeed, when ggbs hydrates, its alkali will inevitably be released into the pore solution and thereafter be indistinguishable from that released from portland cement.

Several investigators have attributed the reduced ASR susceptibility of ggbs concrete, to modifications in the chemistry and permeability of the cementitious paste. They suggest (1 that the reduced permeability to alkali and/or the increased ability of the cementitious paste to bind alkali, result in ggbs concretes being able to tolerate high levels of alkali without expansion. As shown in Table 2 this is an approach taken by Germany, Belgium and France in their Regulations.

Fig 2, which takes into account the full contribution of alkali from all sources (including the ggbs), demonstrates this increased tolerance to alkali. At 50% ggbs, no expansion has occurred with mixtures containing as much as 8kg/m^3 of alkali. At 70% ggbs, no expansion has occurred even with approaching 11kg/m^3 of alkali. By comparison, normal portland cement mixtures expanded with as little as 5kg/m^3 of alkali.

UK Rules for the Avoidance of ASR

It appears that the concept of taking a proportion of the ggbs alkali as reactive, is not valid. This being the case, it will never be possible to sensibly determine the correct proportion to consider as reactive. The basic philosophy of UK rules, therefore, needs to be reconsidered. Nonetheless, for UK materials, the assumption of a zero (or water-soluble only) contribution of alkali from the ggbs appears to err on the safe side, particularly for a ggbs percent of 35% or more.

CONCLUSIONS

1. The concrete prism test produced expansion in the laboratory with a mixture similar to that known to have suffered ASR damage in the UK, in actual structures.

2. Storage of prisms at 38°C gave similar expansion behaviour to storage at 20°C but in a shorter timescale (about four times faster).

3. Storage of concrete prisms at 38°C is an accelerated test which appears to correlate well with practical situations.

4. The results are not consistent with the concept of taking a fixed proportion of the alkali in the ggbs as reactive. At 25% ggbs, an assumed contribution of either 0% or 50% of ggbs alkali would fit within the accuracy of the results. At 35% and 50% ggbs, an assumed contribution of 0% of ggbs alkali is more appropriate. At 70% ggbs there is an apparently negative contribution.

5. The concretes containing ggbs, tolerated greater alkali contents without expanding. This effect was more pronounced for higher proportions of ggbs.

6. UK Design Rules for the use of ggbs to avoid ASR, appear to be based on an incorrect hypothesis, and underestimate the effectiveness of ggbs, particularly at higher replacement percentages.

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LONG-TERM EFFECTIVENESS OF FLY ASH IN PREVENTING DELETERIOUS EXPANSION DUE TO ALKALI–AGGREGATE REACTION IN CONCRETE

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ABSTRACT

A non-reactive and several reactive aggregates were used in concrete specimens with and without two low calcium fly ashes at binder (cement + fly ash) alkali levels ranging from 0.46 to 2.5% and a binder content of 500 kg/m^3 and fly ahs/binder ratio of 0.25. The specimens were stored either at 23°C (fog room) or 40°C, 100% RH. Some specimens were steam cured at 75°C for eight hours, and then transferred to 40°C, 100% RH. The expansion behaviour of the specimens was monitored over nearly six years, and showed that the effectiveness of the fly ash in preventing deleterious AAR expansion depended on the alkali content of the concrete. At the highest alkali content of 12.5 kg Na_2O equiv./m^3, the fly ashes only had a delaying effect (one to several years), whereas at 6.9 kg Na_2O equiv./m^3 they eliminated deleterious AAR expansions. Generally, for more highly reactive aggregates, and at the 2.5% alkali level, fly ash was less effective at 40°C than 23°C because the rate of AAR expansion was much higher at 40°C. At lower alkali levels and for less reactive aggregates, the temperature was not important. Fly ashes were also effective under steam-curing conditions. A measurable amount of chemical shrinkage occurred in the first few months in concretes containing fly ash and high alkali contents, although some of these concretes later expanded and cracked as a result of aggregate reactivity.

It is concluded (based on six-year results and the shapes of expansion cures) that the two fly ashes can be used to prevent deleterious AAR expansions in practical situations.
Keywords: Alkali–aggregate reaction; fly ash; steam curing.

INTRODUCTION

The effectiveness of fly ash and other mineral additives in preventing the deleterious effects of alkali–aggregate reaction (AAR) has been known for a long time, and recent reviews (Berry & Malhotra 1986; Hobbs 1986; 1987; 1989; Helmuth 1987) have summarised most of the previous work. The controversy on the effectiveness of fly ash in suppressing AAR expansion is related to the composition of the fly ashes and cements used, as well as to the degree of reactivity of the aggregate employed.

For instance, a range of fly ashes and other mineral admixtures were found to stop any deleterious expansion in concrete caused by slowly reactive aggregates when 20 or 30% of cement (by mass) was replaced by fly ash (Nixon & Gaze 1983; Nixon *et al.* 1986; Pepper & Mather 1959; Stark 1978). However, Hobbs (1981; 1986; 1987; 1989) used highly reactive opal as aggregate, in its most expansive combination with cement. In such cases, even very effective fly ashes may not reduce the expansion to safe levels.

Other factors that may influence the results include the level of replacement and the fineness of the admixture (Dunstan 1981). The amount of fly ash needed to suppress AAR also depended on the Ca content of the fly ash. Lee (1989) suggested that for each fly ash a critical Na_2O/SiO_2 ratio determined maximum expansion due to AAR. Nagataki *et al.* (1991) found that suppression of AAR expansion depended on the amount of soluble alkali and amorphous SiO_2 content of fly ashes and their fineness, but not on their total alkali content.

Thomas *et al.* (1991) noted very little evidence of AAR in concrete specimens containing a reactive aggregate and a high alkali fly ash, when the level of replacement was 20% or more. Blackwell *et al.* (1992) showed that a fly ash of 4.0% Na_2O equiv. prevented expansion of concrete containing a reactive greywacke and 7.0 kg Na_2O equiv./m^3. However, their results are only short term (15 months). Thomas *et al.* (1992) provided long-term (30 years) evidence on the effectiveness of a high alkali fly ash in a dam structure to prevent damage due to AAR.

Hobbs (1994) also found that expansion due to AAR was influenced by the interaction between the level of alkali in cement (0.6–1.2% Na_2O equiv.), fly ash (3.0–3.9% Na_2O equiv.), and the level of replacement, determining the total alkali content of the concrete. The influence of mineral admixtures on the alkalinity of the pore solution of concrete and the consequences of this on AAR also depend on the alkali content of the cement (and concrete), as well as the available alkali content of the admixture and its replacement level (Nixon *et al.* 1986; Canham *et al.* 1987; Farbiarz & Carrasquillo 1987). Generally, low alkali mineral admixtures are more effective than high alkali ones in reducing pore solution alkalinity and expansion caused by AAR, and their effect is greater than a mere dilution of high alkali cements (Oberholster & Westra 1981; Diamond 1983; Canham *et al.* 1987). However, the variability of fly ashes in this respect has clearly been shown by Diamond (1981; 1983). Duchesne and Berube (1994) found that expression and analysis of the pore solution is the best method of estimating the contribution of fly ash alkali to the pore solution of concrete.

Australian fly ashes have been found to be effective in reducing AAR expansion (Shayan 1990; 1992), although the reported results related to short-term monitoring (one year). This paper provides long-term data for nearly six years, showing that at very high alkali contents (12.5 kg Na_2O equiv./m^3) AAR expansion may not be prevented, although the fly ashes employed have an excellent tolerance for all practical levels of alkali, even as high as 7.0 kg Na_2O equiv./m^3.

EXPERIMENTAL WORK

Materials

The aggregates used (all from Queensland) were a non-reactive basalt, a reactive basalt, two river gravels, a greywacke, a quartzite and an ignimbrite, all being reactive in laboratory testing. A detailed petrographic description of the aggregates has been given elsewhere (Shayan 1992). Two cements, designated D and N, were used with alkali levels of 0.79 and 0.46% Na_2O equiv. The fineness of the cements and their compositional features were similar. Two low calcium fly ashes, designed (1) and (2), were used with SiO_2 contents of 72.6 and 57.7% respectively. Glass content comprised 80 and 93% of fly ashes (1) and (2) respectively, which also contained alkali contents of 0.21 and 1.35% Na_2O equiv. Cement D and fly ash 1 were used in combination with all the aggregates except the ignimbrite for which cement N and fly ash 2 were used. This choice was governed by the geographical location of sources. The sand used in this work has been shown (Shayan *et al.* 1994; and unpublished work) to be non-reactive in concretes containing less than 7 kg Na_2O equiv./m^3, but it caused moderate expansions in concrete containing 10 kg Na_2O equiv./m^3 and deleterious expansions in concrete with 12.5 kg Na_2O equiv./m^3. This sand is regarded as non-reactive by local practice.

PROCEDURES

Concrete prisms measuring 75 × 75 × 285 mm were used for the evaluation of expansion potential of the various aggregate–cement–fly ash combinations. The concrete mixture contained 500 kg cement per m^3 concrete, with a water/cement ratio of 0.40. To evaluate the effectiveness of the fly ashes in controlling AAR, in some mixes 25% of the cement was replaced by fly ash on a mass basis. The water/binder ratio was kept at 0.40, and this caused a reduction of compressive strength of the concretes containing fly ash.

Three levels of cement alkali were employed in this work, viz. the native level of alkali as well as cement alkali levels of 1.38 and 2.5% Na_2O equiv. achieved by adding NaOH to mixing water. Considering the cement content of the concrete, these cement alkali levels correspond to concrete alkali contents of 6.9 and 12.5 kg Na_2O equiv./m^3. The corresponding mixes containing fly ash also had the same alkali contents. The addition of alkali also resulted in a considerable reduction in compressive strength of the concrete, as previously observed for mortar and cement paste (Shayan & Ivanusec 1989). The preconditioning, storage and measurement of length change for specimens at 40°C, 100% RH, was described earlier (Shayan 1992).

Similar specimens were stored at 23°C in a fog room and measured the same way. Another group of specimens were steam cured at 75°C for eight hours and, after initial length measurement on demoulding, were transferred to storage conditions of 40°C, 100% RH for monitoring their expansion behaviour.

RESULTS AND DISCUSSION

Expansion curves for concrete prisms containing each of the aggregates tested, and stored under the storage conditions of 40°C, 100% RH, are presented in *Fig. 1.*

The expansion and cracking of concrete prisms made with the non-reactive basalt (without fly ash) at 2.5% alkali level (*Fig. 1*), is attributed to the reactivity of the sand component at this high alkali content, as shown by Shayan *et al.* (1994). All the other aggregates caused relatively rapid deleterious expansion and cracking of concrete at both the 1.38 and 2.5% alkali levels.

The quartzite aggregate was the most rapidly reactive among those tested and caused cracking in concrete prisms at the lowest alkali content (3.95 kg/m^3, i.e. no added alkali). However, all of the reactive aggregates may react in field concretes of this alkali content due to probable redistribution of alkali and local high concentrations that could be produced as a result of the moisture movements and drying out of concrete.

Figure 1 also shows that 25% mass replacement of cement by fly ash has been very effective in eliminating the AAR expansion in concretes that contained the 1.38% alkali level (6.9 kg Na_2O equiv./m^3). However, it only delayed the expansion and cracking at the highest alkali level of 2.5% (12.5 kg Na_2O equiv./m^3), although this alkali level would be outside the usual range of concrete alkali content. This was not evident from the short-term results (Shayan 1992) that showed only the ignimbrite aggregate caused cracking in the fly ash concrete at the 2.5% alkali level. These results emphasise the need for long-term monitoring of specimens to verify the effectiveness of a certain treatment. Field concretes containing fly ash have been reported to undergo AAR expansion and cracking, and Hobbs (1987; 1994) mentions two such cases in the USA and Japan.

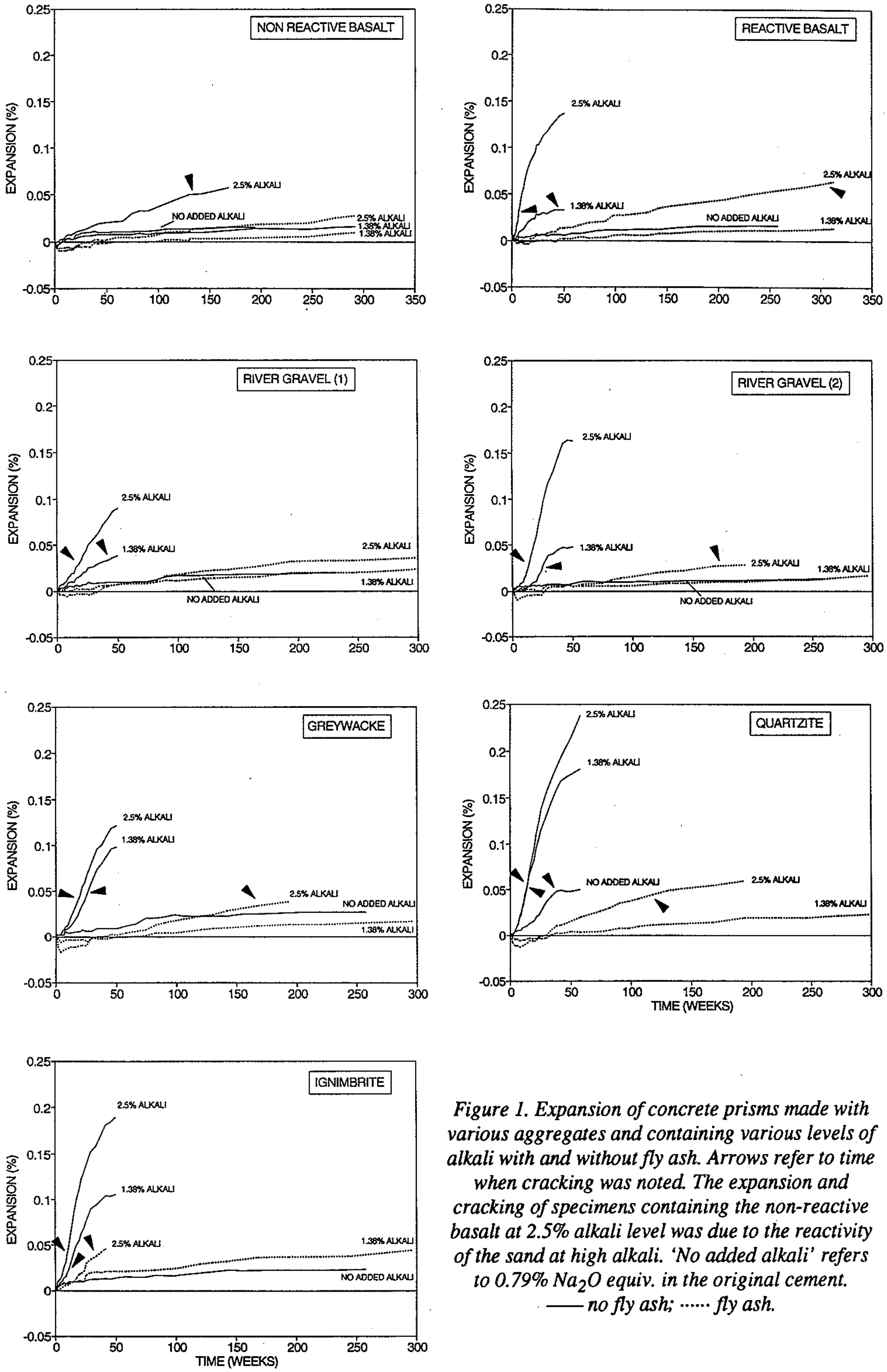

Figure 1. Expansion of concrete prisms made with various aggregates and containing various levels of alkali with and without fly ash. Arrows refer to time when cracking was noted. The expansion and cracking of specimens containing the non-reactive basalt at 2.5% alkali level was due to the reactivity of the sand at high alkali. 'No added alkali' refers to 0.79% Na_2O equiv. in the original cement.
—— no fly ash; ······ fly ash.

In earlier work (Shayan 1990; 1992), it was shown that the accelerated mortar bar test (Shayan *et al.* 1988) could predict the effectiveness of fly ash in suppressing AAR expansion in concrete. From the above results, it is clear that the predictions only relate to certain ranges of alkali content in concrete. This is to be expected because mortar bars immersed in 1 M NaOH solution would have equilibrium pore solution concentrations around 1 M. Therefore the prediction is limited to this upper concentration value. In concrete specimens at the 1.38% alkali level, the pore solution concentration would be about 1 M NaOH, and the prediction of the accelerated test agrees with long-term concrete expansion results. At the 2.5% alkali level, which gives a pore solution concentration of 1.4 M in the concrete, the prediction of the accelerated test is no longer valid.

Figure 2 compares the storage conditions of 23°C (fog room) with 40°C, 100% RH for two aggregates, at the 1.38 and 2.5% cement alkali levels. The other aggregates show similar trends. The expansion curves for the concretes without fly ash show that expansion at 40°C, 100% RH is much more rapid, but that the same magnitude of expansion is reached at 23°C (fog room) at much later times, varying from about one year to a few years. None of the specimens made with fly ash and stored at 23°C (fog room) showed deleterious expansion up to the age of six years, regardless of the level of alkali, whereas most of those made at 2.5% alkali level and stored at 40°C, 100% RH have cracked. This effect was more evident for the more highly reactive aggregates. The difference is probably related to the much faster rate of AAR expansion of such aggregates at 40°C, 100% RH.

Figure 3 represents the behaviour of steam-cured (75°C) fly ash concretes made at 1.38% binder alkali level and stored at 40°C, 100% RH, and compares the effects of the

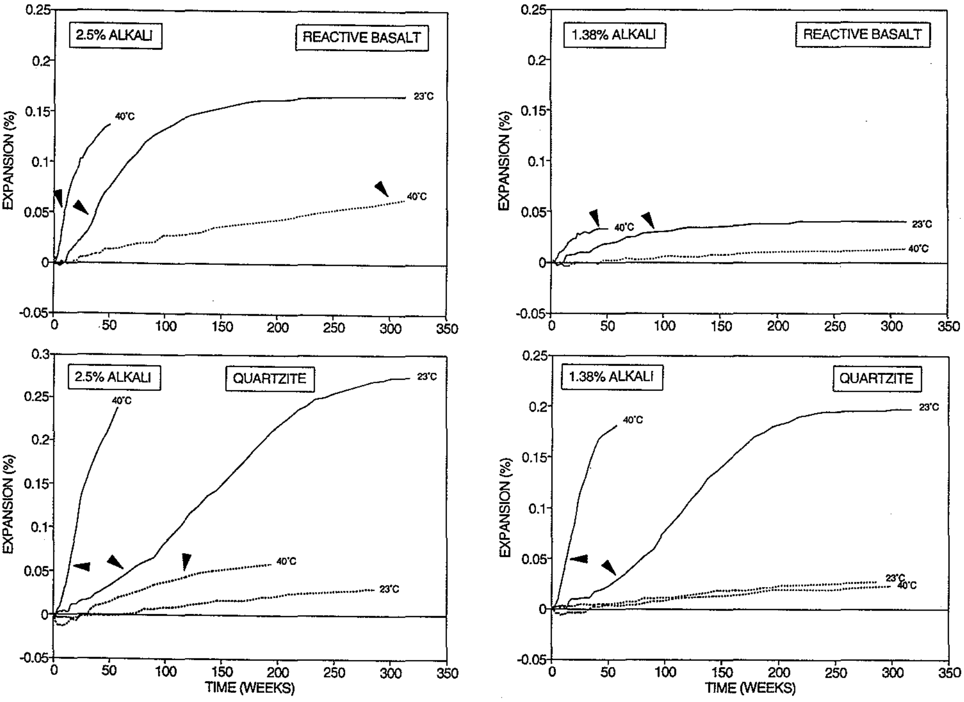

Figure 2. Expansion of concrete prisms made with reactive aggregates with and without fly ash at alkali levels of 1.38 and 2.5%, and stored at 40°C, 100 RH, or 23°C (fog room). Arrows refer to time when cracking was noted. —— no fly ash; ······ fly ash.

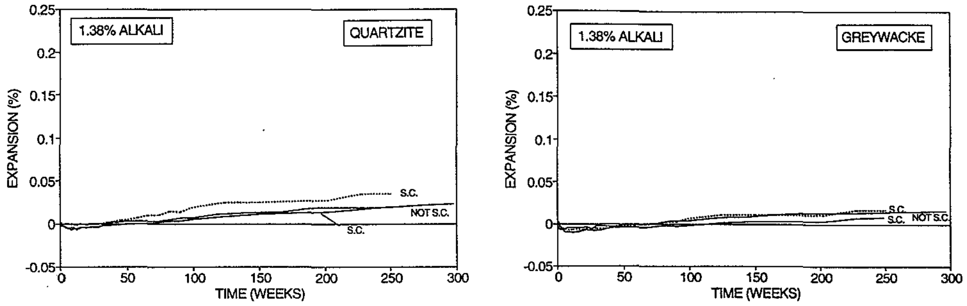

Figure 3. Expansion of concrete prisms made with reactive aggregates and the two fly ashes at 1.38% alkali level. Steam curing is designated by 'S.C.' on the curves. The fly ashes behave similarly despite large differences in total alkali contents. —— no fly ash; ······ fly ash.

two fly ashes under these conditions. Steam curing does not influence the effectiveness of fly ash in suppressing AAR expansion, and that fly ash (2) which contained a much higher total alkali content caused only a slightly higher expansion than fly ash (1). The higher glass content of fly ash (2) probably contributes to its effectiveness.

The effectiveness of various mineral admixtures in reducing deleterious expansions caused by AAR has been attributed to various factors. These include reduction in permeability (Smolczyk 1975; Bakker 1981), formation of various hydration phases which bind the alkali ions (Tenoutasse & Marion 1986; Quillin *et al.* 1993; Hogan 1985), and the consumption of the $Ca(OH)_2$ produced by cement hydration (Tang & SU-Fen 1980; Chatterji & Clausson-Kaas 1984; Bhatty & Greening 1986). The reaction of fly ash and $Ca(OH)_2$ produces hydrated phases with a low Ca/Si ratio and a high affinity to bind alkali ions. Results of Qian *et al.* (1994) are also in agreement with this mechanism.

An interesting observation made as a result of this work is that the specimens containing fly ash, particularly those also containing high alkali contents, exhibited an initial shrinkage during the first few months before some expansion took place. This phenomenon is not due to drying shrinkage because it was also observed in large blocks immersed in water. It is suggested that the observed shrinkage in early age is due to the chemical attack of alkali on the glassy phase of the fly ash and its dissolution and formation of a denser CSH-type hydration phase. Therefore, this phenomenon could be regarded as a chemical shrinkage such as that taking place when concentrated NaOH solutions react with unstable silica (Knudsen 1986). Further studies are needed to explain the mechanism of this shrinkage and its implications for the use of fly ash as an AAR preventive mineral admixture.

CONCLUSIONS

Long-term results presented in this work have shown that the two Australian fly ashes studied have been effective in preventing deleterious AAR damage in concretes with alkali contents as high as 7.0 kg Na_2O/m^3, but they produced only a delaying effect in concretes containing 12.5 kg Na_2O/m^3. The delay was between two and six years, depending on the type of aggregate. A measureable chemical shrinkage occurred in the first few months in the presence of fly ash in the latter concretes, although some of them later expanded and cracked due to AAR.

For highly reactive aggregates and at high alkali contents, the fly ashes were more effective in preventing AAR expansion under the storage conditions of 23°C (fog room) than at 40°C, 100% RH, due to the much faster rate of AAR under the latter conditions.

Although the accelerated mortar bar test (1 M NaOH, 80°C) can be used to predict the long-term effectiveness of fly ash in suppressing deleterious AAR expansions, this prediction only applies to concretes having alkali contents which would produce pore solution concentrations around or below 1 M NaOH.

As field concretes contain usually less than 7.0 kg Na_2O equiv./m^3, the two fly ashes should be effective in suppressing AAR in practical applications.

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EFFECT OF REACTIVE AGGREGATE POWDER ON SUPPRESSING EXPANSION DUE TO ALKALI-SILICA REACTION

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ABSTRACT

A condition under which alkali — silica reaction (ASR) damages occur is put forward and based on this a powder of an identified reactive aggregate was used to inhibit the deleterious expansion of mortars due to ASR containing the same aggregate sand with five fractional particle sizes. The results show that the powder could reduce the expansion and the further reduction of expansion could be accomplished with increased amount of the powder. For example, when alkali level remains 3.5% $Na_2O_{equi.}$ in mortars, the expansion after autoclaving was reduced by 43.6%, 59.5%, 77.1%, 89.5% and 95.4% from the control value of 0.46% as 30, 40, 50, 60 and 70% of powder was used to replace the same amount of cement, respectively. This implies a very promising approach to economically and effectively preventing mortars or concrete from the serious occurrence of ASR. The effect of the powder on setting times of cement and on strengths as well as on the flowability of mortars were also studied.

Keywords: ASR, reactive aggregate powder, suppression of expansion, preventing ASR

INTRODUCTION

The number of reports on the occurrence of ASR in concrete constructions all over the world is growing, and the losses due to this harmful reaction in rebuilding or rehabilitating these damaged constructions are also accordingly on the increase. Therefore, an inhibition of the reaction is calling so much attention that extensive studies on this subject have been undertaken from phenomena to mechanisms.

The following conditions under that ASR damages occur are put forward by the present authors: ① the presence of reactive aggregate particles with detrimental radii, generally considered within the range of 0.15-0.60 ㎜ (Ono et al. 1986); ② the presence of high level alkalies and therefore their concentration around these aggregate particles and thus ③ in-situ alkali-aggregate reaction occurring in certain restrained interfacial space and consequently the increased volumes or the swelling of the reaction products. Among the above three ② and ③ play a very important part in the expansion process. So excluding the reactive aggregate from use is the safest way but this is not always feasible, however, controlling the other conditions might more economically and feasibly reach the destination for preventing the damage.

To control the in-situ reaction or the swelling of reaction products, a variety of chemical and mineral admixtures have been selectedly studied.

Since Li^+ ions can penetrate into the structures and substitute some parts of Na^+, K^+ ions in N(K)-C-S-H gels produced from the ASR and consequently anti-swell, the use of Li-containing chemicals as inhibitors for ASR has resulted in a large volume of findings since the constructive work of McCoy et al (McCoy et al. 1951). Their research, using pyrex glass and a high-alkali cement (1.15% Na_20 equivalent), revealed that different Li-containing chemicals have significant reductions in expansions, for instance, 88% by adding 1.0% LiCl of cement, 91% by 1.0% Li_2CO_3, 98% by 1.0% LiF etc., respectively, after curing for 8 weeks. Sakaguchi et al (Sakaguchi et al. 1989) studied the effects of lithium carbonate, nitrite and hydroxide on expansions of mortars, and verified their effectiveness too. In our previous work, we also confirmed that $LiNO_2$ reduced ASR expansion most effectively at a Li/Na ratio of 0.8 when alkali content is greater than 2.0%, while smaller ratios would be desired for lower alkali levels (Bian et al. 1995).

In view of the conditions used by different authors, we conclude that the effectiveness of lithium compounds on suppressing expansion depends in a complicated way on the variety of chemicals used, alkali levels in cement and concrete, Li/Na molar ratios (thereby the amount of compounds used) and probably the ratio of alkali to available silica in reaction.

Another common method for preventing mortar or concrete damage due to ASR is the utilization of blending materials, though there are some debates on their effectiveness (Duchesene et al. 1994a &b). Duchesene et al (Duchesene. 1994a) conducted investigations into the effect of blending materials (silica fume, fly ash, slag etc.) against ASR. They observed a pessimum effect for concrete containing about 5% of silica fume or about 20% of very high alkali fly ash, and found that even as much as 10% of silica fume of high alkali content may not limit the concrete expansion to a satisfactory level (e.g.<0.04% after 2 years). On the other hand, 35 to 40% of slags as well as about 20% of low alkali fly ashes (<3.5% $Na_2O_{equi.}$) may result in good results. Based on this, they further concluded that at higher alkali contents both in blending materials and in concretes a much greater amount of substitution for cement is needed, which might be accomplished with lower quality of concrete. Qian et al (Qian et al. 1994a) concluded that the main reason for mineral admixtures alleviating ASR is the relieving of corrosion of reactive aggregate from alkali owing to the adsorption of OH^- ions on the surfaces of admixture particles, resulting in a reduced amount of alkali attacking the surfaces of aggregates, and that the higher the acidity of the admixtures, the lower the expansion.

Based on the foregoing observations, there seems to be a need for an auxiliary admixture in order to effectively suppress the unexpected expansion through transforming an in-situ reaction to an innocuous one or through anti-swelling. The ground reactive aggregate powder might be selected as such a kind of admixture as it usually contains reactive silica with higher acidity. Furthermore, in view of the compatibility between aggregate and admixture, it can reasonably be deduced that the utilization of such a powder could expectantly prevent mortars or concretes from suffering ASR.

STARTING MATERIALS AND PROCEDURES

Starting materials

Cement: An ordinary portland cement (OPC) with low alkali level (0.46% Na_2O equi.) was used as a cementitious material (Blaine's value 3360 cm^2/g).

Aggregate: An andesite composed mainly of bronzite andesite was crushed into five fractions and used to fabricate mortars (Bian et al. 1995). It was previously identified as a potentially reactive aggregate by ASTM C289 with its Sc of 732 and Rc 177 mmol/l. It was also demonstrated that α -cristobalite is the primary reactive component of the andesite by XRD.

The chemical analyses of the materials are shown in Table 1.

Table 1 Chemical compositions of starting materials (%)

	LOI	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Na_2O	K_2O	TiO_2	P_2O_5	MnO
OPC	2.1	20.9	5.1	3.1	64.2	1.5	1.9	0.20	0.42	0.26	0.10	0.11
Andesite	1.9	66.2	16.1	3.4	3.3	2.0	0.0	3.50	2.60	0.23	0.10	0.06

Admixture: Powders of the same aggregate with different Blaine values of 2900, 5640, 7800, 8045 and 11330 cm^2/g were used as admixtures to replace part of the cement. The replacements accounted for 30, 40, 50, 60 and 70%.

Solid NaOH was used as an additional reagent to adjust the Na_2O content in mortars to desired levels of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0 and 3.5% wt. against cement mass.

Procedures

Mortar specimens with the dimensions of 4 × 4 × 16 cm, equipped with pins at two ends, were cast with a cement/aggregate ratio of 1: 2.25 and a ratio of water to cement of 0.50. Immediately following their establishments of initial lengths (L_0, μ) after curing for 24 hours in a fog room, the mortars were demoulded, and were autoclaved for 4 hours at a gauge pressure of 0.28 MPa with 1-hour interval of temperature-rising. They were then moved into the same fog room for another 24-hour-curing and their augmented lengths (L, μ) established. Thereafter, they were put into a container at 45 ℃ for long-term measurements.

The effects of aggregate powder on setting times of cement and on flowability as well as on the strength of mortars made of Japanese standard sand (Toyoura sand) were also studied.

RESULTS AND DISCUSSIONS

Expansion characteristics

The effect of amount of powder on expansion

The effect of reactive aggregate powder (Blaine's value 7800 cm^2/g) on the expansion of mortars are substantiated. Fig.1 gives the relation between expansion and aggregate powder amount incorporated in mortars.

It could be seen that for any alkali level, the expansion of mortars decreased remarkably with an increasing amount of powder, especially at high alkali levels. For example, at the highest alkali content of 3.5% Na_2O in this study, the expansion decreased by 43.6%, 59.5, 77.1, 89.5 and 95.4%, from 0.46% of the control specimen for 30, 40, 50, 60 and 70% of powder replacement, respectively. It is obvious that the effectiveness of reactive aggregate powder on suppressing the excessive expansion depends strongly on the alkali contents available. Generally about 50% of the powder could depress the expansion below 0.1% with alkali greater than 3.0% Na_2O equivalent, and 35% of the powder could reach the same level when 2.5% of alkali was available and less amount for lower alkali levels. Therefore, the lower the alkali level, the lower the replacement.

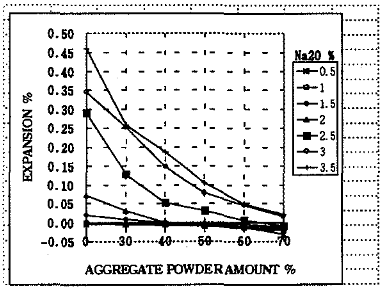


Fig.1 Relation between expansion and aggregate powder amount

The reason for such a powder suppressing expansion can be attributed to the disruption of the in-situ reaction and can be interpreted as follows: First, the powder substitutes some parts of cement, resulting in the reduction of available alkali levels coming from the cement, and also diluted the total alkali concentration in mortars. Moreover, the replacement would increase the acidity of the whole binder, being able to "neutralize" part of the alkalies. Qian et al (Qian et al. 1994b) have confirmed that there is a good relationship between mortar expansion and the acidity of the binder. Increasing acidity of binders could decrease the expansion rate of mortar. On the other hand, the incorporation of powder changes the distribution of alkalies in the binder matrix and mortar, and also between the aggregate and their powder particles. The macro-homogeneously dispersed fine powder particles closely around the aggregate sand absorb more alkalies on their greater specific surface area. Meanwhile, the crushing and milling of aggregate could produce some broken bonds at the particle surfaces. Some defects in crystal lattices such as ≡ Si· (E' center) and ≡ Si-O· (unbridged oxygen hole center, NBOHC) could also exist in siliceous reactive components of aggregate according to recent studies of geochemistry (Ikeya. 1993). These highly reactive free radical groups of unpaired electrons (the so-called dangling bonds) or the broken bonds should be responsible predominantly for their alkali-silica reactivity. Thus for these reasons, the fine particles retain alkalies and react with them through solutions in capillary pores instead of at the interface between cement paste and aggregate, forming complicated products and retarding the alkali migration toward aggregates consecutively. Furthermore, the powders along with their reaction products in the bulk matrix could fill up pores and voids, again densifying the matrix and causing more difficult migration of alkali species. However, such a reaction in the pores could cause practically no expansion, though a very limited expansion is even observed which would also benefit the densification of the matrix. As a result, all of the above mentioned factors would contribute to the prevention of a detrimental reaction or to the suppression of excessive expansion.

Effect of fineness on expansion

The effect of fineness of powder on the expansion of mortar is schematically shown in Fig.2. Evidently there is a sensitive fineness range of 5640 ~ 8045 cm^2/g to the expansion within it the variation of fineness will lead to an evident change of expansion, and below or beyond this range the expansion becomes less sensitively in dependence of fineness, So in this sense that the fineness of aggregate powder should be of about 8000 cm^2/g is technically and economically meaningful.

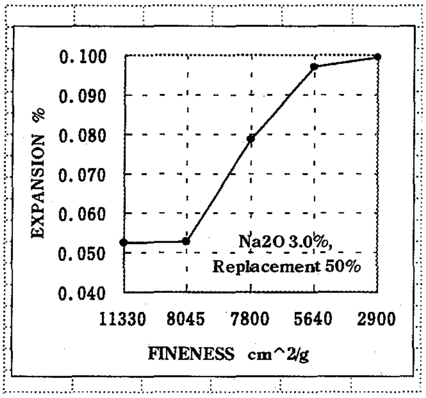


Fig.2 The relation between expansion and fineness of aggregate powder

Powder effect on other properties

Setting times and flowability

The effect of reactive aggregate powder (Blaine's value 7800 cm^2/g) on setting times of cement and on flowability of mortars were also studied at different replacements of OPC by 20, 30, 40 and 50% of powder. For the setting time test, the pastes were made with a constant ratio of water/binder (cement+powder) of 0.26, and for the flowability test the mortars were made with a ratio of binder/aggregate of 0.5, and water/binder ratio of 0.65. The results are shown in Table 2.

It is evident that the setting times decrease slowly when less than 30% of the powder is incorporated, and rapidly (especially for the initial setting) when 50% of the replacement is taken. All of these demonstrate that the powder could take part in the hydration reaction and improve the reaction rate. This phenomenon is somehow different from that when normal blending materials are used. On the other hand, once additional alkali are added to the binders, both setting times and flowability are reduced by a much bigger factor due to the hydration of cement which is usually accelerated by alkalies. In the case of higher alkali levels, the shortened setting time of NC40 by the powder compared to NC37 can also be ascribed to both the action between powder and alkali and the effect of powder retaining alkali.

Table 2 Effect of powder on setting times and flowability

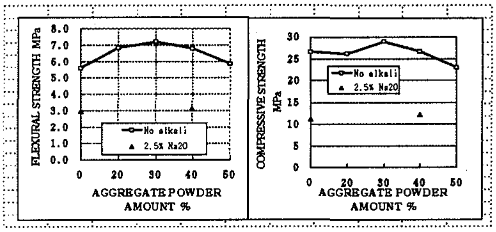
No.	cement %	powder %	Na_2O %	setting time (min.) initial	final	flowability mm
C37	100	—	—	147	269	219
C38	80	20	—	133	243	219
C39	70	30	—	120	226	214
C40	60	40	—	77	216	210
C41	50	50	—	59	167	202
NC37	100	—	2.5	11.5	50	167
NC40	60	40	2.5	8	38	172

However, the flowability of mortar is relatively less influenced by the powder while a relationship between the setting time of cement paste and the flowability of mortar also exists (the shorter the setting time, the lower the flowability) due to the rate of hydration.

Effect on strength

The same mortars as used in the flowability test were cast into 4 × 4 × 16 cm prisms and autoclaved under the same conditions as for expansion test and thereafter cured for 24 hours followed by strength determinations . The results are shown in Fig. 3.

It can be seen that the strengths are similar to or higher than that of pure cement mortar (no replacement) when replacement within 40% and that both flexural and compressive strength acquired the highest values when 30% of powder substituted the same amount of cement. This also verified that the



(a) (b)

Fig.3 Relation between mortar strength and aggregate powder amounts (a) flexural strength; (b) compressive strength reactive powder could take part in the hydration process and modify the strength development. However, elevating the alkali level in mortars decreased the strength markedly due to the decomposition of C-S-H by Na^+ ions, and even in this case (NC40) the incorporation of powder (40%) could also develop the strength by a factor of 5.4% and 9.9% for flexural and compressive strength, respectively, compared with NC37. This is because the reduction of Na_2O based on its reaction with silica resulted in the formation of gels and the newly formed gels, containing more Na^+ ions, could further react with Ca^{2+} ions which could benefit strength too.

CONCLUSIONS

1. The incorporation of reactive aggregate powder seems able to prevent mortars made of the same reactive aggregate sand from severe expansion, implying a promising approach to suppressing ASR.

2. Compared with existing approach, such as incorporating Li-containing chemicals and blending materials (silica fume, ground granulated blast furnace slag and pulverized fly ash), the local use of reactive aggregate powder to suppress ASR is more economical and effective.

3. The amount of powder used to replace part of cement depends mainly on the alkali levels in the mortar and also on the reactivity or the acidity of the powder. The higher the alkali level, the more powder is required and a greater effect can be observed.

4. The reason for aggregate powder inhibiting ASR is explained as follows: the alkalies are retained by and react with aggregate particles (in unconfined spaces) causing a reduction of alkali concentration near to the aggregate surface. Meanwhile, the powder along with its products fill up pores, resulting in a densification of the matrix and difficult migration of alkali species.

5. The fineness of powder affect its effectiveness in preventing ASR and an reasonably appreciable value of about 8000 cm^2/g is recommended.

6. The incorporation of such a powder could consistently shorten the setting times and modify the strength development within an adequate replacement range. Less effect on flowability of mortars could be observed, however.

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SUMMARY OF BRE RESEARCH ON THE EFFECT OF FLY ASH ON ALKALI-SILICA REACTION IN CONCRETE

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ABSTRACT

This paper summarizes the findings from a five-year research programme on the effect of fly ash (Class F) on ASR in concrete containing a range of U.K. aggregates. The study included expansion testing, pore solution analysis and microstructural examinations of laboratory concretes stored under a range of conditions and larger specimens under field conditions. In addition, field studies were made of fly ash concrete structures incorporating reactive aggregates. Only a small proportion of the data is presented here for the purpose of highlighting the principal findings of this investigation.

Collectively, the results from these studies do not support the concept of an "effective alkali contribution" from fly ash. The efficacy of Class F ash depends on the level of ash replacement and the reactivity of the aggregate, whereas the alkali level of the ash appears to have little effect. Higher levels of ash are required to control the expansion of more reactive aggregates (i.e. those that react at lower alkali contents in OPC concrete).

The implication of these findings for the specification of fly ash with reactive aggregate are briefly discussed.

Keywords: alkali-silica reaction, effective alkalis, expansion, fly ash, specifications

INTRODUCTION

Current U.K. guidelines (BRE, 1988; Concrete Society, 1992) for minimizing the risk of damage due to alkali-silica reaction (ASR) in concrete containing potentially reactive aggregate permit the use of fly ash to mitigate the effects of ASR. However, there is conflicting evidence regarding the efficacy of fly ash in this role and this is reflected in the lack of consensus advice. Much of the controversy is centered around the alkalis in the fly ash and whether they are potentially available for reaction. This is of particular concern when the total alkali burden of the concrete is being controlled to a specified maximum to avoid damaging reaction when potentially reactive aggregates are used. The Concrete Society recommends that the water-soluble alkali content of the fly ash is used when calculating of the total alkali content of the concrete. More conservative advice from the Building Research Establishment suggests that one-sixth of the total alkali content of the fly ash be used. Use of the "one-sixth rule" results in "available" alkali levels in the range 0.4% to 0.7% Na_2O_e for most U.K. ashes, which compares with values around 0.1% for water-soluble alkali.

In response to the conflicting advice regarding the role of alkalis in fly ash (and slag), the U.K. Concrete Society set up a technical sub-committee to review the subject of alkali contribution from fly ash (and slag). Under the guidance of this committee, further research was initiated to study the effect of fly ash on the expansion of concrete containing natural reactive U.K. aggregates and cristobalite. The programme on fly ash and cristobalite was carried out at the British Cement Association and results from the study have recently been reported by Hobbs (1994). A parallel programme using natural aggregates from U.K. sources was carried out at the Building Research Establishment (BRE). This paper presents a brief summary of some of the findings from the ongoing studies at BRE.

EXPERIMENTAL STUDIES

BRE carried out an extensive programme on the effect of fly ash on ASR in concrete containing reactive U.K. aggregates, which included the following studies: (i) expansion testing of concrete prisms stored under laboratory conditions, large specimens (up to 1m cubes) stored under field conditions and concrete specimens stored in various alkali salt solutions; (ii) pore solution analysis of concrete containing reactive flint; (iii) analysis of hydrate composition in concrete with a range of OPC, fly ash and aggregate combinations; (iv) field studies of concrete structures (with and without fly ash) constructed with reactive aggregates.

Five reactive aggregate sources were used in this study; these were: Thames Valley river sand (<5 mm), a crushed flint sand (<5 mm), a sea-dredged sand, a quarried siltstone (5-20 mm), and a quarried siliceous limestone (5-20 mm). Results are also reported for a greywacke aggregate from Wales. Greywacke has recently been implicated in a number of ASR-affected structures in the U.K. (Thomas et al. 1992) and its reactivity is currently the subject of an extensive investigation at BRE.

A number of Portland cements were used throughout the study to provide a range of alkali levels. Fly ash samples from four sources were included; all four of these ashes are marketed commercially for use in concrete.

RESULTS

Standard Concrete Prism Tests

The effect of fly ash on the expansion of concrete containing reactive flint (Thames Valley sand), siliceous limestone, siltstone and greywacke aggregate is shown in Figure 1. In these figures, the latest expansion data (at either 3 or 4 years) at 38°C is shown plotted against the alkali content of the concrete calculated on the basis of OPC alkalis only, i.e. disregarding the alkalis from the fly ash. From these relationships it is possible to estimate the "effective alkali contribution" from the fly ash using the approach developed by Hobbs (1986; 1994).

This *alkali contribution* from the ash is clearly dependent on the type of reactive aggregate and the level of ash replacement used. With flint aggregate, 25% ash is sufficient to prevent damaging expansion regardless of the alkali content of the concrete (within the range tested). Cracking was not observed after 4 years, even when the alkali content from the OPC was in excess of that required to induce cracking in control specimens. In other words, the ash has a positive role in suppressing expansion beyond that of merely diluting the cement. Other studies have shown lower levels of ash (e.g. 20%) to be equally effective (Nixon et al. 1987; Thomas et al. 1991). Similar behaviour was observed with the other sources of flint aggregate used in this study.

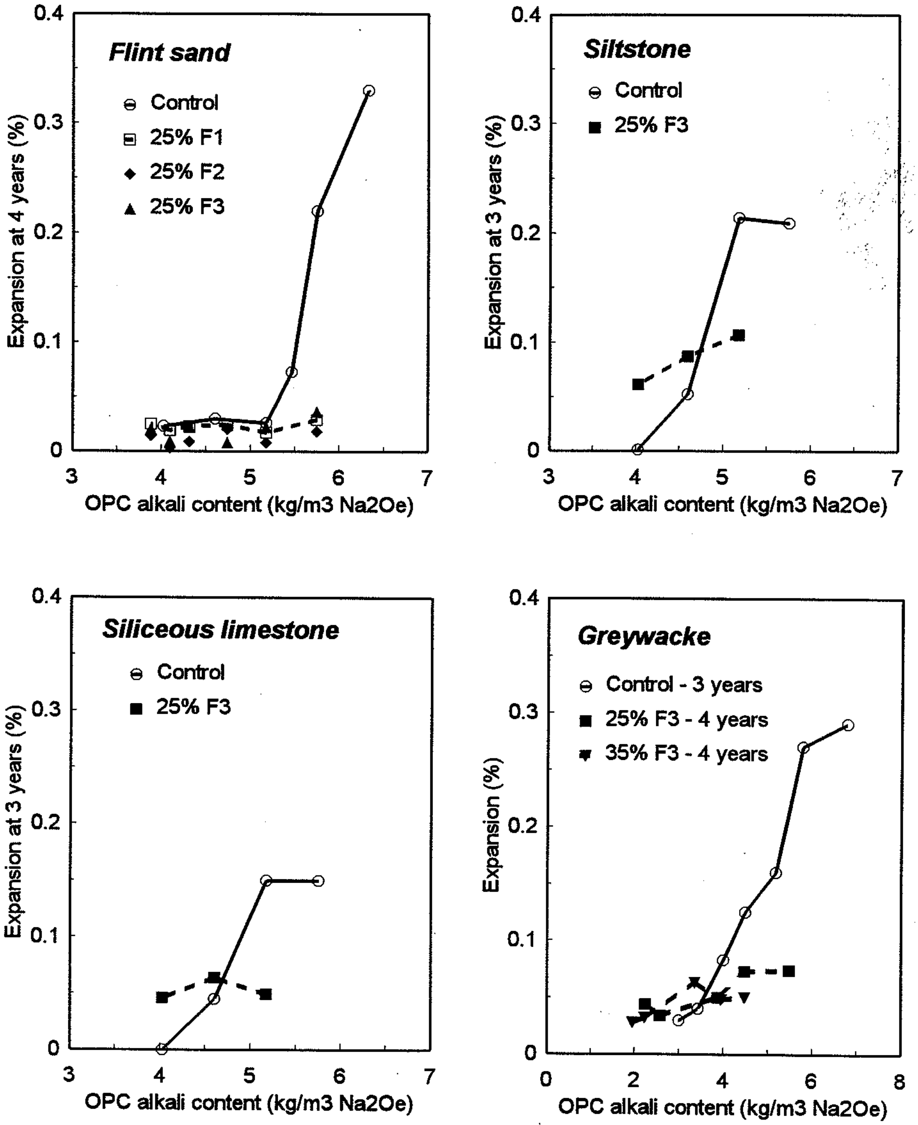

Figure 1 Expansion results for fly ash concretes

However, 25% fly ash was not effective in totally eliminating damaging expansion in concrete cast with the siltstone, siliceous limestone and greywacke aggregate. With these aggregates, significant expansion (> 0.05%) was observed in the ash concrete at lower OPC alkali levels than in the control specimens, indicating that the fly ash *contributes* alkalis to the concrete. At higher alkali levels, the ash appears to reduce expansion compared to control specimens at the same OPC alkali content. Indeed, examination of the relationships shown in Figure 1 suggests that the ultimate expansion of these fly ash concretes is not strongly influenced by the availability of alkalis from the OPC. Effective control of cracking with these aggregates may be achieved through the use of higher levels of ash, as shown by the results for 35% fly ash with greywacke aggregate.

Figure 2 shows the relationship between the *effective alkali contribution* estimated from expansion tests and the fly ash replacement level for the different aggregates; results from Hobbs' tests (1994) using artificial cristobalite as a reactive aggregate. This indicates that the quantity of ash required to control cracking increases with the reactivity of the aggregate. In this context, aggregate reactivity refers to the level of alkali necessary to induce cracking in the control specimens; i.e. more reactive aggregates induce cracking at lower alkali levels. The data is replotted in Figure 3 to show the *effective alkali contribution* as a function of the "threshold alkali content" of the aggregate.

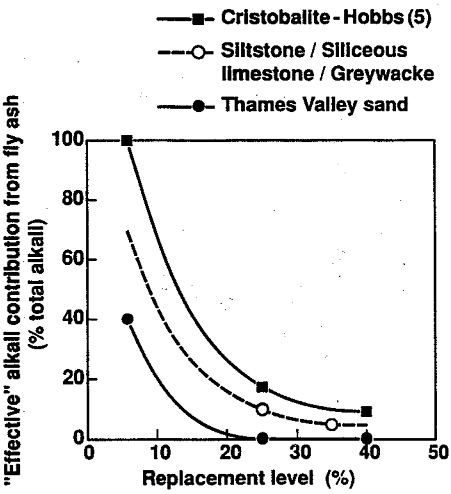

Figure 2 Effect of replacement level on alkali contribution from fly ash

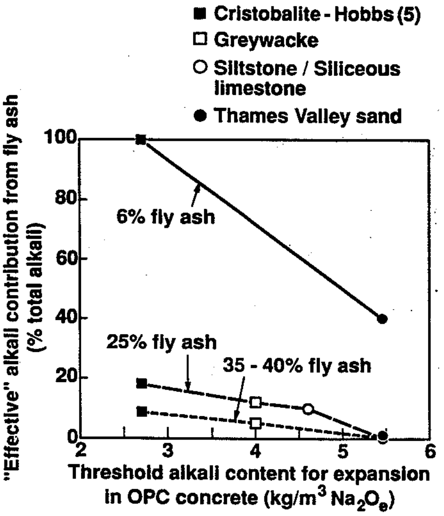

Figure 3 Effect of aggregate reactivity on alkali contribution from fly ash

Field Exposure of Large Concrete Blocks

Large concrete specimens (1.00 m or 0.35m cubes) of various cement-ash-aggregate combinations have been placed outdoors at BRE to study the effect of fly ash on ASR under field conditions. To date, some of the control specimens have cracked whereas all the fly ash concrete specimens are undamaged. However, considerably longer exposure periods are required before any conclusions can be drawn from these studies.

Pore Solution Studies

The pore solution composition of concrete, with and without reactive flint, was determined at various ages from 1 day to 2 years. The effect of reactive aggregate and fly ash is shown in Figure 4 for concrete cast with high alkali cement. For concrete without flint (i.e. 100% "inert" aggregate), the hydroxyl ion concentration of the pore solution at 1 day was lower in the fly ash concrete, the reduction being almost equal in proportion to the level of cement replaced. However, the ionic concentration in the fly ash concrete increases over the next 7 days and ultimately becomes very similar to that of the control. This indicates that fly ash has an affect beyond that of dilution and may be considered to "contribute" alkalis to the pore solution.

The addition of reactive aggregate has a marked effect on the pore solution, reducing alkalinity as early as 1 day after casting. The reductions continue for approximately 1 year and reach a steady value beyond this time. The behaviour of fly ash and control concrete was similar in this respect. Similar effects were observed for ashes with higher and lower alkali contents, indeed, in some cases the alkali concentration was higher in ash concretes compared to control samples with high alkali cement only.

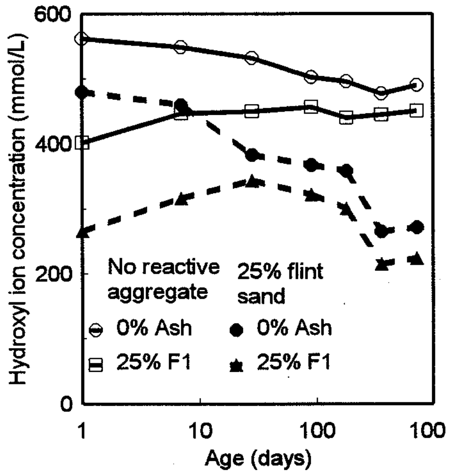

Figure 4 Pore solution evolution - effect of reactive aggregate and 25% fly ash

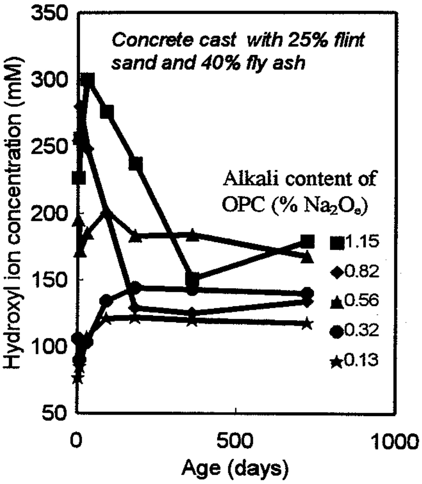

Figure 5 Pore solution evolution - effect of 40% fly ash

Figure 5 shows the effect of 40% fly ash on the pore solution of concrete cast with a range of Portland cements (0.13% to 1.15% Na_2O_e). The fly ash reduces the OH^- concentration when blended with high alkali cements and increases the concentration with low alkali cement. After 1 year, the OH^- concentration of all the concretes containing 40% ash lie in a fairly small range (120 to 190 mmol/L OH^-), suggesting that the ash has a buffering effect on the pore solution. The significance of the "buffering" effect will depend on the reactivity of the aggregate. If the reaction of the aggregate is not sustained at the buffered alkali level, then fly ash will be effective in suppressing the reaction and resulting expansion; this is the case with concretes containing reactive flint sand. If, however, the threshold alkali level for reaction of a particular aggregate is below the buffered alkali level, fly ash will not be effective in preventing reaction and may actually provide a reservoir of alkalis (i.e. its own) to fuel the reaction.

Alkali Immersion Tests

Concrete cylinders were immersed in a wide range of alkali salt solutions at various temperatures. Figure 6 shows the expansion of concrete specimens stored in 1N NaOH at 80°C. Concrete specimens with 40% ash showed no significant expansion and were uncracked after 2 years' exposure to this environment. Despite the lack of expansion, there was abundant evidence of reaction in these concretes; signs of reaction ranged from silica gel oozing from the surface of the specimens to complete dissolution of flint grains within the concrete. Samples examined by SEM/EDX showed distinct differences in the nature of the reaction and the reaction product due to the presence of 40% ash. The addition of lime at the mixing stage increased the expansion of all concrete containing reactive aggregate. The results from these studies are discussed in greater detail elsewhere (Thomas, 1995).

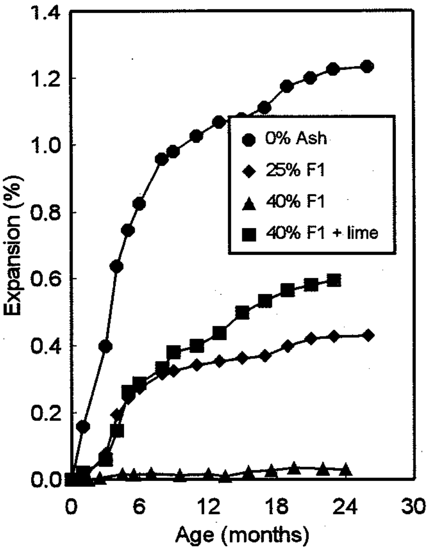

Figure 6 Expansion of concrete in NaOH solution - effect of fly ash and lime

Field Studies

Field data undoubtedly provide the most reliable indication of concrete performance. Fly ash has been used in concrete for many decades and, to date, there are no substantiated cases of damaging ASR in structures containing sufficient levels of Class F fly ash. However, in order to determine the positive effect of fly ash in suppressing ASR in a particular concrete structure, there has to be some means of assessing the likelihood of damage had the ash not been present. The use of fly ash in the Nant-y-Moch Dam in Wales, U.K. and the Lower Notch Dam in Ontario, Canada, has recently been documented by the authors (Thomas et al. 1992; Thomas, 1996). Both these structures contain reactive greywacke/argillite rock which has been implicated in damaging ASR in other hydraulic structures in the vicinity of the dams. However, both fly ash concrete dams are in excellent condition, despite the use of high alkali cement at Lower Notch.

DISCUSSION

The need to determine the alkali contribution of fly ash (and other supplementary cementing materials) stems from the now widely adopted use of maximum concrete alkali contents to reduce the risk of damage in concrete incorporating potentially reactive aggregates. Most fly ashes contain significant quantities of alkalis and there is a perception that these must be accounted for in the calculation of the total alkali content of the concrete. Other workers have attempted to determine the alkali contribution from supplementary cementing materials using leaching tests such as the "available alkali" test in ASTM C-311 and modified versions thereof (using cement in place of lime), or by analyzing pore solutions expressed from paste, mortar or concrete specimens. Hobbs (1986) developed an approach for estimating the "effective" alkali contribution from fly ash (and slag) using expansion data, and it may be argued that this is more relevant for specification purposes since, in practice, the concern is one of expansion and not pore solution effects. The term "effective" alkali contribution is used to distinguish the contribution calculated from expansion results from the proportion of alkalis actually released by the fly ash into the pore solution; these quantities are not the same.

The expansion tests reported in this paper were carried out to determine what proportion (if any) of the fly ash alkalis "effectively" contribute to the expansion of concrete containing natural U.K. aggregates. The results from this study and parallel studies at the BCA using cristobalite (Hobbs, 1994) demonstrate that the "effective" alkali contribution varies with fly ash replacement level and the nature of the reactive aggregate; other factors (e.g. fly ash chemistry and mineralogy) may also have an influence but were not adequately investigated in this study. Consequently, the adoption of a single value, such as one-sixth of the total ash alkali, is clearly inappropriate. This value is based on fairly specific circumstances (i.e. 25% ash with opal or cristobalite) and is too conservative for concretes with less reactive aggregates (e.g. flint) or higher ash replacement levels, and may actually preclude the use of ash in these situations.

The determination of "effective" alkali contributions from fly ash, whether from expansion or pore solution studies, for use in specifications where the alkali content of the concrete is to be limited, presupposes that availability of alkalis for reaction is the governing factor controlling ASR expansion in concrete containing ash. Such an argument is not sustained by the studies discussed in this paper.

The results and discussion presented here refer specifically to the use of low-calcium fly ash from bituminous coals (e.g. ASTM Class F). High-calcium, low-silica ashes are less effective in controlling expansion due to ASR, when used in the same proportion as Class F ashes, and may have to be used at higher replacement levels to suppress damaging alkali-silica reaction (Thomas, 1994).

SPECIFICATIONS

There is conflicting information from laboratory studies concerning the ability of fly ash to control ASR expansion. This is in contrast to the evidence from the field performance of fly ash concrete, which unanimously supports the use of fly ash as an effective “cure” for ASR. It is incongruous that when we have been unable to make the results of laboratory studies tally with field performance, we have based our specifications on the "artificial" tests in preference to the "real world".

The specification of fly ash for use with reactive aggregate should take cognizance of the need for increasing levels of ash as the reactivity of the aggregate or calcium content of the ash increases. The main problem with such a specification is the designation of aggregate reactivity since a well-defined basis for ranking aggregates according to their level of reactivity does not exist at this time. Knowledge of the reactivity of the aggregate is critical for optimizing the use of fly ash to control ASR and has application beyond the use of mineral admixtures as it would also allow more flexibility in selecting an appropriate alkali level for all concrete (with or without admixtures) based on the type of aggregate being used. Without knowledge of aggregate reactivity, specifications for fly ash (and slag) will have to either: (i) be overly conservative to cover all aggregate types; (ii) accept the chance of failure if a highly-reactive aggregate is inadvertently used; (iii) limit the use of fly ash to certain aggregate types (which implies some level of aggregate classification).

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SILICA FUME IN CONCRETE - 16 YEARS OF EXPERIENCE IN IÇELAND

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ABSTRACT

During the period 1961 - 1979 AAR caused serious damages to concrete structures (mostly to housing) in Iceland. In 1979 several actions were taken to prevent AAR damages in future constructions. Probably the most important one was using silica fume (SF) as a partial replacement in all Icelandic OPC and RPC as these cement types are used in over 95 % of all concrete houses. The SF is, with small exceptions, inter grounded with the cement clinker for better homogeneity. Approximately every third year a condition survey is carried out on houses in order to investigate if any AAR damages still occur. The latest and the most extensive survey, carried out in 1993, studied houses from the period 1977 - 1991. For the first time (in Iceland) a microscope was utilised in the study.

In this paper results from these investigations are presented. Assessment of the general quality of concrete in this period is made. Furthermore the homogeneity of the concrete and the distribution of SF is discussed and related to the way it is blended into the cement. AAR expansion of up to 12 years old mortar bar prisms according to ASTM C 227 are shown. Finally, a comparisons between ASTM C 227 and the recommended RILEM Concrete prism method are made in this paper.

INTRODUCTION

Serious damages were caused by AAR in concrete structures in Iceland in the years from 1961 to 1979. Preventive actions (possolan cement - non reactive aggregates) were taken for larger concrete structures. For house construction no actions were taken during this period, since these structures were not considered in any danger, partially due to low moisture content (Gudmundsson & Asgeirsson 1983). Icelandic Portland cement has a extremely high alkali content, currently about 1,65 % wt. as Na_2O eq., with the ratio of sodium to potassium oxide about 3:1 by weight (Gudmundsson 1995). The aggregates utilised in concrete are mostly volcanic in origin and some of which are very reactive with respect to AAR. The high reactivity of the aggregates is due to relatively high content of rhyolitic (acidic) material, altered basalt and the fact that some of the material is sea dredged and unwashed. In 1979 preventive measures were taken against AAR in concrete, those were: 1) blending silica fume (SF) into cement, 2) changing the criteria on reactive materials, 3) sea dredged material must be washed, and 4) use of reactive material was limited.

Tests on SF as a counteraction to deleterious AAR in concrete were first conducted in Iceland in 1972 (Gudmundsson 1975, Gudmundsson & Asgeirsson 1975). Since then the effect of SF on various properties of concrete has been studied (Gudmundsson & Asgeirsson 1979, Olafsson 1982, Olafsson & Helgason 1983, Asgeirsson et al. 1985, Asgeirsson 1986, Olafsson 1989, Wallevik 1990, Olafsson 1992, Gudmundsson & Moller 1992). In July 1979 the use of SF in commercial concrete started and has been used in almost all ordinary concrete ever since. Research had shown that a relatively small amount of SF was sufficient to suppress AAR expansions effectively in concrete with reactive Icelandic aggregates (Gudmundsson 1975, Gudmundsson & Asgeirsson 1975). Thus 7,5 % as cement replacement has been used since 1983, but before 5 % was used, due to some technical and practical problems in the cement production. By using so limited amount most of the undesired side effects such as increased water requirement, drying shrinkage etc. are mostly avoided. Instead of mixing SF into the concrete it was mixed with the cement itself, mostly by inter grinding it with the cement clinker. By doing it this way one reasoned that the homogeneity of the cement and the concrete as well was secured.

Annual freeze/thaw cycles in Iceland are relatively many, more so in the southern part (including the Metropolitan area) than elsewhere. Therefore, shifts from driving rain (near horizontal) to freezing conditions can take place within few hours. Due to these harsh climatic conditions a theory has been put forward that AAR damages in Icelandic concrete is not solely due to chemical reactions, but more to interactions between chemical reactions and freeze/thaw actions (Kristjansson 1985, Olafsson 1989).

The dominant laboratory method for assessment of the reactivity of aggregates used in Iceland has been the well known ASTM C 227 mortar bar method. This method has given indications of reactivity that seem to coincide with practical experience. The criteria set in 1979 for housing concrete was expansion under 0,1 % after 12 months.

CONDITION SURVEY

The preventive actions taken in 1979 have been followed up by condition surveys in order to investigate their effectiveness (Kristjansson et al. 1979). The methodology in carrying these surveys out has changed a little with time. The first ones were based on visual inspection, but in houses where map cracking was observed, cores were taken for laboratory inspection. In 1984 additional cores were taken from houses built in the period 1979 - 1983. The cores were initially inspected for AAR and again after 12 months storage at 100 % RH and 38 °C. In 1986 a similar survey was carried out on cores from houses build in 1984 and 1985, but this time the cores were reacted for 3 months in the AAR reactor. In 1993 an extensive study was made on concrete houses built in the years 1976 - 1990 (Sveinsdottir & Gudmundsson 1993). For each year approximately 15 houses were picked randomly and cores drilled from all of them. The cores were inspected for the first time by use of microscopy and thin sections. The purpose was to evaluate the quality of concrete in exterior walls of houses, like the AAR, w/c-ratio, air entrainment and SF distribution.

Table 1. Frequency of AAR damages in exterior walls of houses in Reykjavík from 1972-1982, survey 1983.

Year	Number of houses	Frequency of damaged houses (%)				Comments
		None	Small	Considerable	Severe	
1972-1973	60	15	45	35	5	
1974-1975	58	23	59	10	0	
1976-1977	61	48	48	4	0	
1978-1979	60	54	43$^{1)}$	3$^{1)}$	0	$^{1)}$from 1978
1980-1981	56	91	9$^{1)}$	0	0	
1982	8	100	0	0	0	

Table 1 shows the results of the survey carried out in 1983. As can be seen the frequency of considerable to severe AAR was up to 40 % of the houses built in 1972 - 1973, but decreases to 3 % in houses from 1976 and 1977, and after 1979 AAR can not be found.

In table 2 the result from the last survey is shown. A gelatinous reaction product was found in voids and cracks in 8 out of 16 houses from 1977 and in 6 out of 15 houses from 1978. After 1979 no gel was found in any samples, with the exception of 1981 where some gel was observed in few isolated voids. On the other hand some silica fume clusters were observed, somewhat contrary to expectations. These lumps are of similar amount each year. No trace of AAR were observed around these clusters.

Table 2. Alkali gel and silica fume clusters in concrete, survey 1993

Year cast	Number of houses	Alkali gel no. of houses	Clusters of silica fume (no/mm^2*10^{-2})	Comments
1977	16	8	0	
1978	15	6	0	
1981	16	2[1)]	13	[1)]trace
1983	15	0	10	
1985	16	0	10	
1988	13	0	11	
1990	16	0	11	

On figure 1 is shown a typical silica fume cluster in hardened concrete. Silica fume clusters are also relatively common in laboratory sample like mortar bars or similar samples made with relatively small mixers. Identical findings have been reported by (Lagerblad & Utkin 1995).

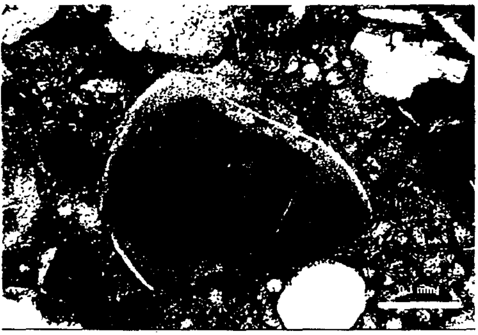

Figure 1. Photomicrograph of silica fume cluster in concrete, 200 x magnification.

DISTRIBUTION OF SILICA FUME IN HARDENED CONCRETE

The SF utilised in Icelandic cement is a by-product from the production of iron-silicon alloys, located within 10 km from the cement plant. Initially the SF powder is relatively lightweight, with a bulk density of about 0,24 g/cm^3. To be used in the cement production, it's density must exceed 0,6 g/cm^3. Therefore, the SF is compacted at the ferrosilicon plant. It is either compacted with air, which results in a relatively small **grains** (up to few mm in diameter); or it is palletised with water, which results in relatively large and hard SF **pellets**. Upon grinding the cement clinker, the SF is inter ground with the clinker. The pellets are added to the mill with the clinker, but the grains are added to the cement after the mill. Then the cement is "sieved" in a centrifugal sieve, the portion that is fully ground, passes through the sieve, but the remaining portion is sent back to the mill for regrinding. The layout of the grinding process is shown in figure 2 (Gudmundsson 1995).

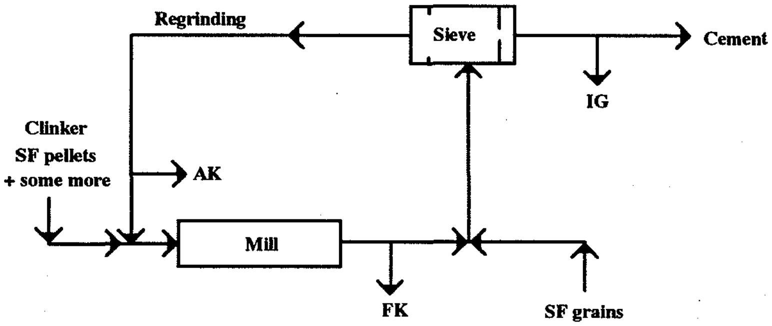

Figure 2. Schematics of the grinding process and location of sample sites and sample numbers.

In order to investigate the origin of silica fume clusters in hardened concrete, as shown on figure 1, few grinding experiments were carried out at the cement plant (Gudmundsson 1995). In the experiments the ratio of SF grains to SF pellets was varied. Samples were taken from the grinding process, location of the sites and the sample numbers are shown on figure 2.

The morphology of SF grains/pellets in these cement samples was measured. In the study only grains larger than 30 µm in diameter were considered. On figure 3 is shown the length of the grains vs. the width/length ratio (average of about 60 measurements). The SF in the AK samples are distinctly different in shape, being relatively large and round, while the SF in FK and IG samples are relatively small and more elongated.

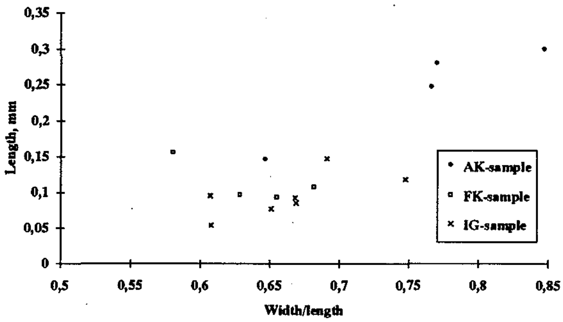

Figure 3. Morphology of SF in cement samples

The bulk composition of these samples was determined and the result of the chemical analysis was used to calculate the SF content in the cement samples. The result of the calculations is shown in figure 4.

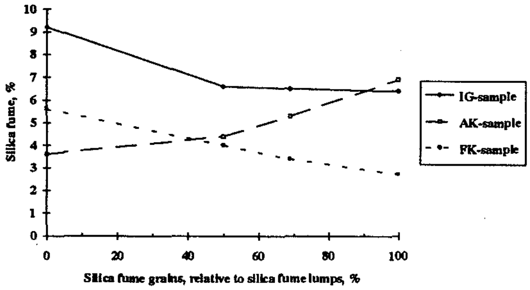

Figure 4. Calculated amount of SF in cement samples

On figure 4 is shown how the calculated % of SF varies as a function of the grain to lump ratio. The SF content of the rejects (sample AK) is proportional to the grain to lump ratio, the higher the SF grain content, the higher is the SF content. This suggest that the SF grains are too large when they are added to the mixture, and most of it is rejected by the centrifugal sieve. We therefore, suggest that the SF grains should be added to the mill with the SF pellets and the clinker, in order to avoid any danger of it passing through the sieve by an accident, which we fear is the common cause for SF clusters observed in hardened concrete.

EXPANSION OF MORTAR BARS - CONCRETE PRISMS

As mentioned earlier research on the effect of SF on alkali aggregate expansion started in 1972. The test method was mostly ASTM C 227 mortar bar method. Most of the research has been done at IBRI with Icelandic material. In 1980 some tests were made with Pyrex glass and three types of cement with Na_2O eq. from 0,86 - 1,39 at NBS (now NIST) in Gaithersburg, USA.

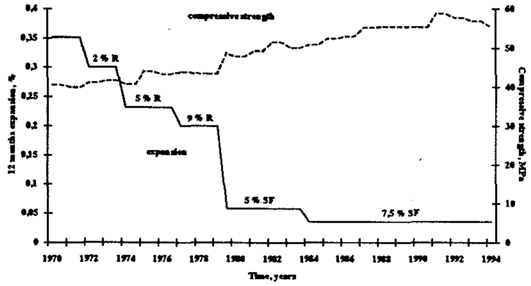

Figure 5. Average AAR expansion and compressive strength in Icelandic OPC. Modified and updated from Asgeirsson 1990. R = Rhyolitic glass, SF = silica fume

In figure 5 the development of Icelandic OPC with regard to AAR expansions and compressive strength can be seen, mostly as a result of the use of pozzolan (rhyolitic glass and silica fume). Research at IBRI has shown that the measured expansion when SF is used not only decreases substantially but is also to some degree delayed. Now we have 12 years old mortar bars that have been measured annually. These samples were made with OPC with no SF, with OPC with 7,5 % SF intermilled with the cement (ENV 197-1 CEM II/A-M 42,5) and OPC with 7,5 % SF added to the cement during mixing. The aggregate source is the same in all the samples, from Hvalfjordur, which has proven to be very reactive. The results are given in table 3 and figure 6. As expected the measured expansion is highest with OPC, but similar in the two SF blended cements. Even though the measured expansion for the SF blended cements, after 12 months, is only 50 % of the allowed limit, the one year allowed limit is reached after 4 years and increase slowly every year. After 12 years the measured expansion is about 60 % over the one year allowed limit. The consequences of these slow expansions are unclear and may not be a cause of any danger.

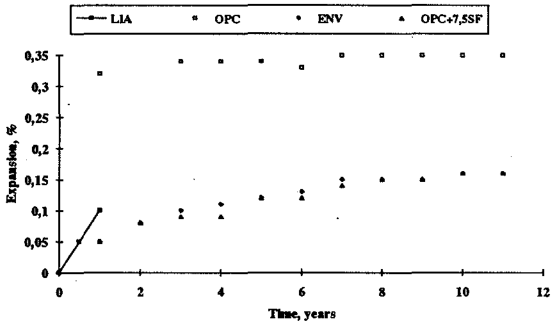

Figure 6. Measured expansion of various types of Portland cements, see the text for discussions. The solid line represents the limits set by Icelandic authorities (LIA).

As earlier mentioned has the ASTM mortar bar method given results that are in a good coherence with practical experience in Iceland. At the moment (1995) comparison measurements are under way between this method and the concrete prism method recommended by RILEM TC 106. First results indicate good coherence but final results will be presented at the conference.

Table 3. Expansion of mortar bars (%), ASTM C 227

Cement type	Year of casting	Expansion by years										
		1	2	3	4	5	6	7	8	9	10	11
ENV	1983	0,05	0,08	0,10	0,11	0,12	0,13	0,15	0,15	0,15	0,16	0,16
OPC+7,5 SF	1983	0,05	0,08	0,09	0,09	0,12	0,12	0,14	0,15	0,15	0,16	0,16
OPC	1983	0,32		0,34	0,34	0,34	0,33	0,35	0,35	0,35	0,35	0,35

CONCLUSIONS

The original purpose of utilising SF in Icelandic cement was to reduce or eliminate the risk for damaging AAR in concrete. After 16 years of experience this report shows that this main goal has been achieved. Through various conditions surveys it has been shown that in no case damage can be trace to AAR after 1979 when silica fume was first utilised. As expected, considering the limited amount of silica fume used, a trace of AAR gel is found in few samples.

The homogeneity of silica fume in the cement was not as good as expected since small clusters of SF is found in hardened concrete. Grinding experiments at the Cement factory indicate that the reason for this is that a part of the SF compacted with air is not as effectively inter ground with the clinker as the palletised part of the SF. Even though this does not have any harmful effect on the concrete, it is bound to reduce the efficiency of the SF replacement.

Criteria for aggregate reactivity based on mortar bars or concrete prisms methods may need to take into account that SF partial replacement prolongs the expansion, as well as reducing it. However, the actual effects of the prolonged expansion is not fully understood.

In summary one can state after 16 years of experience of SF in Iceland that one has been able to solve the AAR problem in an effective and economical way by utilising SF as a partial cement replacement. In the process the general quality of Icelandic cement and concrete has been improved.

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AUTOCLAVE TESTING OF CONCRETE WITH RESPECT TO AAR

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ABSTRACT

Concrete prisms and mortar bars have been subjected to periods of from 4 to 16 hours at 120°C and 0.1 MPa in an autoclave. The aggregate used was crushed granodiorite and quartz sand to which 15% of fused silica had been added. Alkali concentrations were made up to 4% by weight of cement using sodium hydroxide. In addition, some sodium and potassium silicates and silica gel were added to sets of bars. The purpose of this was to see if these reactants and reaction products influenced the rate and magnitude of reaction. The results showed that gel formation and cracking was generated by the autoclave procedure at a level comparable with two or more decades of field exposure. The added compounds took part in the reaction and where potassium silicate was added the gels became potassium-rich despite the presence of abundant sodium in the pore fluid. The technique could allow concrete designed for structural use to be tested directly at lower alkali concentrations than used here. The potential for further expansion in previously damaged concrete could also be evaluated and tests are in hand to evaluate this possibility, the first of which showed no further expansion.

Keywords: Autoclave, concrete, fused silica, sodium silicate, potassium silicate.

INTRODUCTION

This paper describes some tests carried out on concrete containing reactive aggregate (fused silica) to which various substances were added. The purpose of the work was to evaluate the effects of the additions on the magnitude of expansion and cracking. The concrete was made with the alkalies enhanced to 4% by weight of cement using NaOH solution and the concrete was heated in an autoclave to accelerate the reaction some days after curing. The period of heating varied from four to sixteen hours. The added materials were chosen on the basis that they were either adding to the reactants or to the potential products. They are listed in Table 1. Two suppositions were to be tested. Firstly it was postulated that the addition of reaction product or allied substances would reduce reaction by preventing further product formation. Alternatively, it was supposed that the added reaction product would break down by reaction with the cement paste to liberate additional alkalies so that reaction is enhanced.

Table 1 Compounds added to the concrete prisms (the admixtures)

A	Sodium silicate solution	$Na_2O/SiO_2 \sim 0.5$
B	Sodium silicate solution	$Na_2O/SiO_2 \sim 0.3$
C	Zeolite powder	$Na_2OAl_2O_32SiO_2$
D	Potassium silicate solution	$K_2O/SiO_2 \sim 0.25$
E	Sodium silicate powder	$Na_2O/SiO_2 \sim 0.4$
F	Sodium silicate powder	$Na_2O/SiO_2 \sim 0.5$
G	Silica gel powder	SiO_2
H	Sodium silicate powder	$Na_2O/SiO_2 \sim 1$

CONCRETE

The concrete was made with the following recipe.

Water	Approximately 180 kg/m^3
Cement	400 kg/m^3
Sand (0.073 to 3.18 mm)	450 kg/m^3
Fused silica (mostly 3 to 5 mm)	270 kg/m^3
Coarse aggregate (5 to 20 mm)	1080 kg/m^3

Cube tests of the control concrete gave a strength at 28 days of 48 MPa and a bulk density of 2380 kg/m^3. The coarse aggregate was a crushed granodiorite which was carefully washed and thoroughly dried. Similarly, the fine aggregate was a fine quartz sand which was washed and oven dried. The reactive part of the aggregate, the pure silica, was sieved to separate the fraction from 1 to 5 mm and this was used in the test. In practice, most of the material was in the size 3 to 5 mm. This was added to the sand to make a total of 15% of the total aggregate. The cement employed was an ordinary portland cement to BS12. Sodium hydroxide was added to the water to produce a final concentration in the concrete of 4% Na_2O equivalent by mass of cement (Criaud et al 1992, Fournier et al 1991, Saloman and Gallias, 1992, Tang and Han 1983a and b, and Tang et al 1987). It was necessary to carry out some trials with this in order to establish the correct recipe for the water content because the addition of the NaOH greatly stiffened the mixture. Having added the alkaline solution to obtain the necessary alkali to cement ratio, further water was added until the required workability was achieved. The mixes had to be blended and cast rapidly because it was found that the added materials acted variously as accelerators. The added ingredients were mainly added as solutions with the concentrations being used to make effectively 2 and 5% by volume (20 and 50 ml/L) of the total mixture added as part of the aqueous phase. The solutions of admixture used contained 14% by mass of Na_2O equivalent. The cement and solution or suspension was first blended and then the aggregates were added.

Prisms measuring 75 x 75 x 250 mm were cast with vibration and these were cured for between 14 and 19 days in a moist atmosphere. They were then transferred to an autoclave operating at working conditions of 120°C and 0.10 MPa. The apparatus is a Swing Clave autoclave measuring 65 x 75 x 100 cm. This is normally used for biological sterilisation. It requires an hour to reach the steady state condition and an hour to cool. Times referred to in Table 2 relate to the time at 120°C and 0.1 MPa. The tests were run overnight and samples were measured and examined the following day. All the prisms were studied using thin sections and with the electron microscope and with an attached energy dispersive analytical system.

Much gel and numerous microcracks were produced and the concrete after four hours resembled field concrete more than twenty years old and containing a highly reactive aggregate. There was some increase after 16 hours but the shorter time would have been sufficient to identify the potential for adverse reaction and to identify the reactive materials. It was found that after 16 hours a little of the siliceous sand and a few grains of the granodiorite also exhibited some reaction. Gel issued onto some of the prism surfaces and was removed and analysed using the energy dispersive system attached to the electron microscope. The prism lengths were measured using a Mitutoyo Digimatic Indicator type 1DF-130E at room temperature and the prisms were weighed at the time of measurement.

EXPANSION TEST RESULTS

The results obtained are given in Table 2. The controls gave an average expansion of 0.37% in four hours. This compares closely with the expansion created in the ASTM C1260-94 test (ASTM 1994) which gave about 0.40% for the same aggregate. Mortar bars were also made of the same materials with alkalies enhanced to the same extent as those in the concrete and these, after being treated in the autoclave for four hours, gave a mean expansion of 0.39% for the control.

The use of sodium silicate with an SiO_2/Na_2O ratio of unity gave an effectively identical expansion at 2% addition, but the expansion was reduced substantially with 5% addition. All the other added compounds showed a slight increase in expansion relative to the control. However, the expansion at 5% addition was in six out of seven cases less than that at 2% addition. Further, while the controls reached the cited expansion after four hours, others showed a little less expansion after four hours, but matched or exceeded the controls after a further twelve hours in the autoclave. This suggests that the admixtures influence the structure of the binder and retard the early stages of the reaction, but that ultimately the generation of microcracking leads to an acceleration of the reaction and rate of expansion.

COMPOSITION OF GELS

The petrographic examination of the concrete samples and analyses made with the electron microprobe showed a structure identical with those found in examples of concrete taken from the field. Cracks radiate from within the aggregate particles and often link nearby reactive grains. Gel occurs within the aggregate, in cracks in the paste, and in voids, and was found to issue on to the formed surfaces of the prisms in some instances. These gels have been analysed with the results given in Table 3. The analyses show that the gel compositions are similar to those found in field concrete exhibiting adverse reactions. The gels in the aggregate have high alkalies and low calcium, while those in the cracks in the paste have high calcium (Figure 1)(French

1995). Gel exudations, which have not had access to the cement paste, also have high alkalies and low calcium. Where the added alkali-silicate was potassic the gels produced were also potassium-rich so that clearly the added silicates are taking part in the reactions. It seems that the entry of the potassium ions may take precedence over the sodium ions since sodium was always in excess.

Table 2. Expansion of concrete prisms after a period in the autoclave

Samples	admixture (see table 1)	time in autoclave (hours)	expansion %
1	none	4	0.343
2	none	4	0.387
3	H 2%	4	0.366
4	H 5%	4	0.237
5	E 2%	4	0.418
6	E 5%	4	0.368
7	F 2%	4	0.392
8	F 5%	4	0.391
9	C 2%	16	0.465
10	C 5%	16	0.430
11	A 2%	16	0.510
12	A 5%	16	0.447
13	D 2%	14	0.528
14	D 5%	14	0.459
15	G 2%	14	0.457
16	G 5%	14	0.360
17	B 2%	14	0.567
18	B 5%	14	0.551

DISCUSSION

No substantial differences were found between the controls and the prisms to which compounds had been added. Observations of cracks generated in the aggregate, cracks produced in the paste, dissolution of the aggregate, gel infilling voids, and gel total quantity, show very little difference from one sample to another. The qualitative observations were scaled from 1 to 5 with 5 indicating the greatest degree of reaction. If these values are summed, then the samples were found to range from 18 to 25, indicating that the most damage occurred where the additional material was dispersed as silica gel. However, this was only marginally less reactive than where potassium silicate ($K_2O/SiO_2=1$) was employed.

Since the addition of potassium silicate to the mixture led to enhanced potassium in the gel it is evident that the admixtures have taken part in the reaction. Gel in the paste has a higher calcium content than that in the aggregate and this is interpreted as a result of reaction between gel and paste leading to release of alkali into the pore fluid. It seems likely therefore that the admixtures may also have reacted with the paste to liberate alkali metal ions into the pore fluid and hence have enhanced the potential for reaction. The conditions employed in the tests (120° and 0.1MPa) for between four and sixteen hours are very severe and led to some reaction with the granodiorite and quartz sand. These materials are regarded as very stable when used in field concrete. It therefore seems likely that designed concrete mixtures could be tested by this procedure but that lower alkali concentrations should be employed. Experiments are in hand to test these possibilities. It is also possible that the potential for further expansion for concrete already exhibiting some reaction could be evaluated rapidly. Four cores from structures exhibiting reaction were tested in this way all od which showed no further expansion.

Table 3. Chemical analyses of gel in various locations and some areas of paste.

Sample	2	2	2	2	2	6	6	6	6	6	6
Admixture (see table 1)	none	none	none	none	none	E 5%	E 5%	E 5%	E 5%	E 5%	E 5%
Location	B	B	B	C	B	D	D	D	C	B	B
SiO_2	84.4	73.8	83.6	79.2	78.9	45.4	47.2	66.7	68.9	76.9	83.7
Al_2O_3	0.0	0.0	0.2	0.3	0.2	0.3	0.0	0.2	0.0	0.0	0.1
Fe_2O_3	0.0	0.3	0.1	0.4	0.0	0.1	0.0	0.3	0.2	0.1	0.1
MgO	0.3	0.4	0.4	0.1	0.4	0.1	0.0	0.0	0.5	0.4	0.2
CaO	3.4	13.2	3.8	12.3	8.9	48.0	48.0	28.8	20.0	11.2	7.0
Na_2O	6.6	6.3	7.2	3.2	6.0	2.4	0.2	1.7	6.1	7.8	6.5
K_2O	5.4	5.7	4.6	4.4	5.2	0.7	0.3	1.0	4.2	3.4	2.2
SO_3	0.1	0.3	0.1	0.1	0.1	3.0	3.9	0.2	0.2	0.1	0.2

Sample	6	6	6	6	6	6	10	10	10	10	10
Admixture (see table 1)	E 5%	E 5%	E 5%	E 5%	E 5%	E 5%	C 5%	C 5%	C 5%	C 5%	C 5%
Location	D	C	B	B	B	A	E	E	E	B	C
SiO_2	68.0	77.7	60.8	85.3	85.9	87.6	37.0	23.2	25.1	83.8	83.8
Al_2O_3	0.4	0.0	1.8	0.0	0.1	0.0	6.0	7.2	7.5	0.1	0.0
Fe_2O_3	0.1	0.2	0.3	0.0	0.0	0.0	2.3	3.0	3.0	0.3	0.2
MgO	0.9	0.5	0.6	0.4	0.2	0.5	1.8	2.4	2.4	0.5	0.5
CaO	21.7	9.6	23.0	1.8	4.0	0.5	47.1	29.7	57.7	0.2	0.5
Na_2O	7.1	8.7	10.8	10.1	7.6	8.8	1.9	0.8	0.8	11.6	9.9
K_2O	1.3	3.3	1.2	2.0	2.2	2.2	0.7	0.3	0.3	3.3	4.9
SO_3	0.3	0.1	1.2	0.2	0.1	0.2	2.9	2.8	2.8	0.2	0.4

Sample	10	10	10	14	14	14	14	14	14
Admixture (see table 1)	C 5%	C 5%	C 5%	D 5%	D 5%	D 5%	D 5%	D 5%	D 5%
Location	B	B	D	B	A	B	D	C	B
SiO_2	81.3	76.3	83.6	81.5	87.0	81.6	78.7	65.2	68.7
Al_2O_3	0.1	0.2	0.2	0.1	0.2	0.2	0.1	0.7	0.2
Fe_2O_3	0.0	0.1	0.1	0.1	0.0	0.1	0.0	0.9	0.1
MgO	0.9	0.6	0.5	0.3	0.1	0.1	0.4	0.3	0.3
CaO	0.1	7.3	1.0	0.2	4.6	1.8	1.7	19.1	15.8
Na_2O	14.0	11.3	7.4	6.8	3.1	3.4	8.0	2.8	8.9
K_2O	3.2	3.9	4.4	10.6	4.9	12.8	11.0	10.9	5.6
SO_3	0.4	0.0	0.2	0.3	0.0	0.0	0.1	0.2	0.4

Sample	16	16	16	16	16	16	16	16	16	16
Admixture (see table 1)	G 5%	G 5%	G 5%	G 5%	G 5%	G 5%	G 5%	G 5%	G 5%	G 5%
Location	A	B	B	A	F	C	A	A	D	B
SiO_2	49.3	78.4	69.1	44.6	34.2	66.1	34.1	73.4	68.2	38.9
Al_2O_3	2.9	0.0	0.0	4.4	4.7	0.4	3.3	0.8	0.0	3.4
Fe_2O_3	0.5	0.0	0.3	0.0	2.8	0.0	0.2	0.0	0.2	0.1
MgO	0.0	0.1	0.2	0.3	2.0	0.4	0.2	0.3	0.4	0.0
CaO	43.5	11.3	18.8	16.8	52.3	16.8	55.5	11.6	17.8	53.5
Na_2O	2.0	5.2	6.0	0.9	2.2	10.7	1.3	5.7	7.8	1.4
K_2O	0.7	4.7	5.5	0.3	0.6	4.8	0.4	7.3	6.0	0.5
SO_3	0.7	0.2	0.2	1.9	1.0	0.8	4.9	0.6	0.3	2.0

Sample	18	18	18	18	18		13	13	17	17
Admixture (see table 1)	B 5%	B 5%	B 5%	B 5%	B 5%		D 2%	D 2%	B 2%	B 2%
Location	B	B	C	B	C		F	F	F	F
SiO_2	77.4	73.7	87.0	85.1	72.7		71.8	72.0	74.0	74.9
Al_2O_3	0.0	0.0	0.0	0.0	0.0		0.0	0.0	0.0	0.0
Fe_2O_3	0.2	0.1	0.1	0.2	0.1		0.0	0.0	0.0	0.0
MgO	0.2	0.2	0.2	0.6	0.5		1.0	1.0	1.1	1.1
CaO	10.7	17.4	0.9	4.8	15.5		0.5	0.1	1.4	0.2
Na_2O	6.8	4.4	8.3	5.3	6.5		18.8	18.2	17.9	17.8
K_2O	4.7	4.1	3.5	3.9	4.7		8.5	8.6	5.3	5.9
SO_3	0.0	0.0	0.0	0.0	0.0		0.0	0.0	0.0	0.0

Note: locations are as follows; A within aggregate, B aggregate surface, C in crack in paste, D in void, E paste analyses, F surface exudations.

CONCLUSIONS

Concrete containing extra alkalies and a reactive aggregate, treated from 4 to 16 hours in an autoclave at 120°C and 0.1 MPa, shows significant gel formation, cracking and expansion. The expansion created is equivalent to two or more decades of reaction in the field. The addition of dispersed silica gel and soluble sodium and potassium silicates did not suppress the reaction, but if anything the rate of reaction slightly increased. This is interpreted as being due to the release of extra alkalies by reaction with the portland cement paste. The extent to which the autoclave generated expansion, cracking and gel formation in the concrete suggests that this may be a very simple way of detecting whether field concrete is likely to show further expansion or whether a designed concrete may show a potential for reaction. It was observed that some of the quartz sand and some parts of the granodiorite also showed traces of reaction after being held in the autoclave for 16 hours. These aggregates are from field experience known to be thoroughly stable. It seems likely therefore that a reliable test could be established at lower alkali levels than were employed here by means of autoclaving concrete.

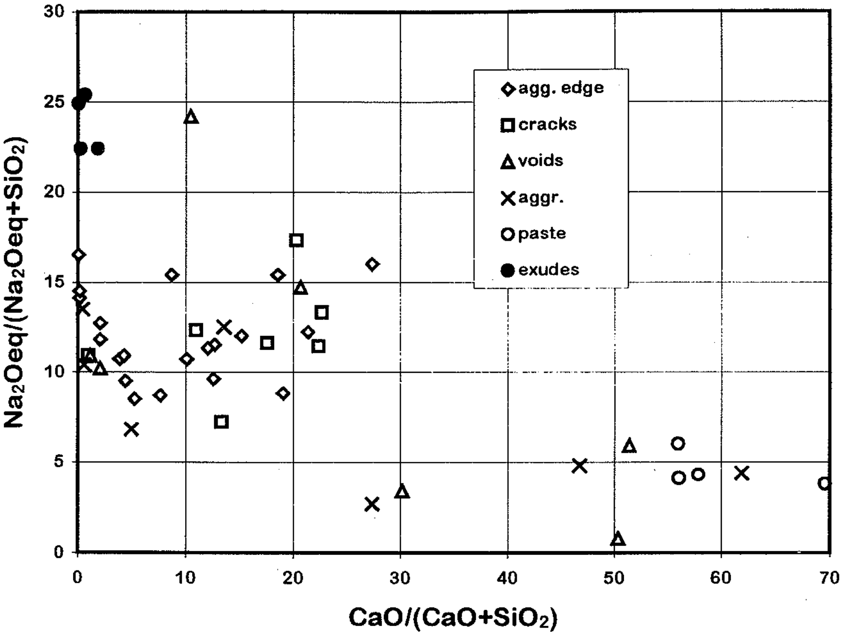

Figure 1. Variation of gel composition with location in the concrete. Note that the values labelled aggregate but with high Ca are altered silica gel phases.

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ALKALI-SILICA REACTIVE AGGREGATES IN BEIJING AREA

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ABSTRACT

The main results of a series of field surveys of concrete aggregates used in Beijing construction industry are reported in this paper. It was found that the main silica reactive aggregate is mainly located in Yongding as well as Wenyu river. Effective measures should be taken when the reactive aggregate is presented. Proposals, which have been adopted by the local government, were raised by the authors to limit the risk of alkali-silica reaction.

Keywords: Alkali silica reactivity, Field survey, Control measures .

INTRODUCTION

In recent years, there is a large construction market in Beijing area. The annual consumption of cement and aggregate are about 5 and 30 million tons. The aggregates used in Beijing are mainly supplied by gravel deposited in river beds in nearby suburbs. Since the several cases of alkali-silica reaction damage in Beijing was reported in the early 1990' (1,2), government officers and construction engineers have become widely concern on this concrete durability problem. Researches and discussions on alkali-aggregate reaction also arouse interests among geologists because of the large verity and complicity of aggregates. Limestone, clay, siliceous shall, sand stone, salt, gypsum mirabilite and so on are very significant in cement and concrete industry as well. A research program on AAR, which was supported by Beijing Construction Committee and the Bureau of Construction and Architecture Industry, was conducted in early 1991. This paper reported some surveys of the alkali reactivity of aggregates used in Beijing concrete industry. Discussions are also made in terms of geology.

During 1990 to 1993, geological inspections were made thirteen times in Beijing suburb areas where concrete aggregates were produced. The total amount of the sand and coarse aggregate collected was about 2.8 tons. Detailed petrographic examination and X-ray diffraction analysis were performed for the determination of rock types and minerals involved. The reactivity of the aggregates was determined by the autoclave mortar bar method, ASTM C227, NBRI as well as ASTM C289 method. 7479 mortar bars were prepared and tested.

RESULTS AND DISCUSSIONS

Based on the Tang's results (2) and the findings obtained by the present investigation, it was found that most sands collected in Beijing area are non-reactive and only small amount of samples showed very slight reactivity. Thus it is believe that alkali-silica reaction in Beijing was mainly due to certain coarse aggregates.

Coarse aggregates used in Beijing can be grouped into 8 rock types, among which five types showed obvious reactivity as listed in Table 1. Only four groups of samples among sixteen random-selected samples gathered from eight typical quarries presented considerable expansion according to C227 and autoclave method test. (Table 2)

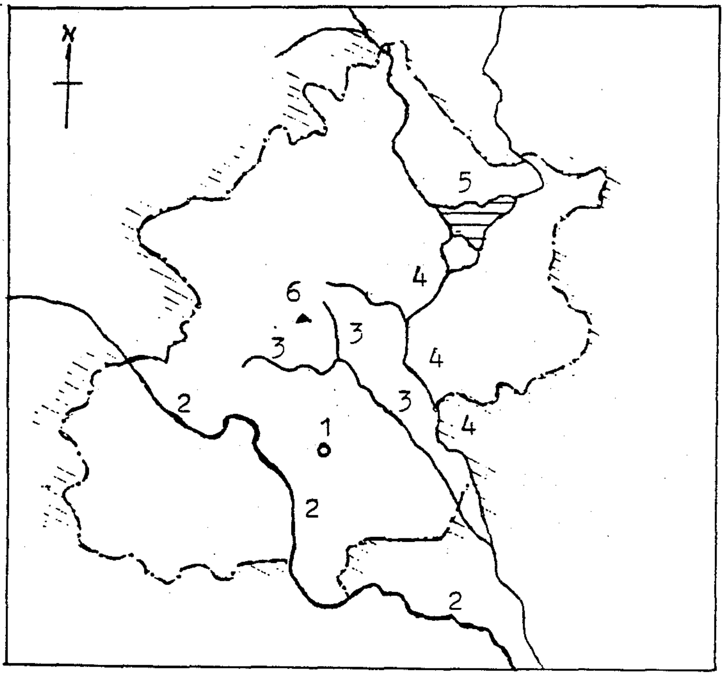
Table 1 Classification of coarse aggregates in Beijing area and their alkali reactivity

No.	Rock Types	Petrographic characteristics	Alkali Reactivity evaluated by			
			C289 method	C227 method	Autoclave mortar bar method	Reactivity
1	Sedimentary of feldspar and quartz	Microlite, chalcedony and microcrystallized quartz	Interrupted by carbonate	0.218 %	0.189 %	Reactive
2	Pyrolith of feldspar and quartz	Absence of reactive silica	non-reactive	0.042 %	0.085 %	non-reactive
3	Striped silicolite	Microlite and chalcedony	Potentially reactive	0.254 %	0.200 %	reactive
4	Massive silicolite	Microlite and chalcedony	Potentially reactive	0.236 %	0.269 %	reactive
5	Morderate acidic pyrolith	Absence of reactive silica	non-reactive	0.078 %	0.083 %	non-reactive
6	moderate basic pyrolith	Absence of reactive silica	non-reactive	0.071 %	0.126 %	reactive
7	Carbonatite	Chalcedony, fine-grained dolomite and aplite	Interrupted by carbonate	0.152 %	0.079 %	non-reactive
8	Pay siliceous carbonatite	Microlite and chalcedony	Interrupted by carbonate	0.207 %	0.186 %	reactive

Table 2 Alkali silica reactivity of typical coarse aggregate collected in Beijing area

Location	Percentage of the reactive rocks containing in the sample						Expansion of mortar bars (%)		
	1	3	4	7	8	Total	ASTM C227	NBRI	Autoclave method
Yongding river	12	5	4	11	21	53	0.082	0.131	0.090
	12	5	4	11	21	53	0.074	0.104	0.097
	11	6	3	11	22	53	0.096	0.122	0.092
	12	5	4	13	21	55	0.107	0.136	0.103
	11	5	5	10	23	54	0.114	0.099	0.106
	13	5	3	15	17	53	0.119	0.127	0.101
	14	4	3	16	14	51	0.090	0.105	0.084
Wenyu river	14	2	4	5	5	30	0.043	0.113	0.085
	16	2	3	5	5	31	0.044	0.098	0.094
Chao bai river	17	3	2	4	6	32	0.040	0.090	0.103
	19	3	2	6	4	34	0.051	0.073	0.095
	15	2	1	5	5	28	0.050	0.086	0.084
	15	3	2	6	6	30	0.050	0.066	0.096
	14	3	2	5	6	30	0.047	0.087	0.081
	16	3	2	5	6	32	0.052	0.070	0.092
Nankou	0	0	0	5	95	100	0.147	0.132	0.114

There are three rivers, Yongdin, Wenyu and Chaobai river, passing through Beijing area as shown in Figure 1. Rock types at the upper reaches are different. After many years weathering and scouring, the rocks were smashed and carried down to lower reaches. Thus the rocks at the upper reaches determine the gravel types in lower reaches. In upper reaches of Chaobai river there are mainly quartzite, granite and gneiss. The main part belongs to metamorphic rock group originated in Archaeozoic era. The rocks are very old and usually hard and condensed. The amount of the alkali-silica reactive rocks are very low. Sand stone, grammite, limestone, dolomite, marlite and volcanic emanation formatted rocks such as liparite, andesite, basalt and hyaline are distributed in Yongdin and Wenyu river. They formed in proterozoic, palaeozoic and mesozoic era younger than that of Chaobai river. The rocks belonged to sedimentary and volcanic rock group containing some reactive rocks. Therefore, care should be taken when gravel is used in cement concrete.



1. Beijing City 2. Yongding river 3. Wenyu river 4. Chaobai river 5. Reservoir 6. Nankou

Figure 1 River system in Beijing area

CONCLUSION

Considering the complexity and unevenly distributed reactive rocks in Beijing area, the authors proposed some suggestions, which have been adopted by Beijing local government, to minimize alkali-silica reaction deterioration in Beijing area. It includes routinely reactivity evaluation of concrete aggregates used in important concrete structures, limitation the total alkali content in concrete no more than $3Kg/m^3$ and use of mineral admixtures to control the ASR expansion. High alkaline admixtures are strictly restricted in Beijing.

In the following years, output of aggregate should be controlled Yongdin and Wenyu area. Instead, local government will encourage the quarries located in Chaobai area to increase their production. On the other hand, high quality limestones have been found in the north of Beijing. Cement manufactures are trying to use them to produce more low alkali cement in recent years.

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Effects of Surface Coatings and Cathodic Protection on Alkali-Aggregate Reaction

TREATMENT OF STRUCTURES BY WATERPROOF COATING

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ABSTRACT

In the field of the treatment of the structures affected by A.A.R. (Alkali-aggregate reaction), the drying out of concrete represents, in theory, the most judicious way to reduce or to stop the development of this phenomenon. However, its application in ideal conditions is not easy. It requires especially, the application of a sufficiently efficient waterproof treatment on all surfaces exposed to humidity.

In this context, laboratory testing, based on expansion tests of reactive concrete specimens which were, dried, treated and then stored in an atmosphere at 38°C and 100 % relative humidity, was first, carried out to evaluate the effectiveness of various waterproofing systems. Although, these tests showed great variability of the performances of products, they did not find one perfect product.

Nevertheless, the deck of a bridge affected by an evolutive A.A.R. was completely treated with one of the products which gave the best results on test samples, this being a thin coating of acrylic resin-cement. Accurate monitoring of the dimensional variations of the structure, its mechanical behaviour and the condition of its facings show, up to now, no adverse effects. It is still too soon to come to a conclusion on the longterm outcome of this method of treatment on structures affected by alkali-aggregate reaction, and the measurements are being continued.

Keywords : *Alkali-aggregate reaction, expansion test, longterm outcome behaviour, monitoring dimensional variation, waterproofing coating.*

INTRODUCTION

Structures affected by A.A.R. cause management problems, insofar as their treatment has not provided up to the present time a reliable and definitive solution, and disorders caused A.A.R. promote the appearance of other pathological phenomena such as corrosion or frost-thaw damages. Amongst all the parameters having an effect on the development of an A.A.R. in a structure, water is *a priori* the parameter which one can control more easily. If the drying out of concrete represents, in principle, a more judicious process in order to reduce or stop the phenomenon, in practice the thickness of the structures make it difficult to completely dry out concrete.

The first attempts at treating structures affected by A.A.R. were by injecting the cracks, usually with epoxy resins. Whether such attempts took place in France [1]

[B.Godart 1993] or abroad, either they nearly all resulted in failure; some cracks reopened or new cracks appeared beside old ones. It is understandable that if the resin prevents water penetrating the cracks, it provides no protection from the penetration of water or humidity to the major part of the facings.

If methods of treatment using coatings would seem *a priori* better than filling in the cracks, it must be acknowledged that the few attempts made using paint are far from being conclusive, even when paint is applied at the time of construction. On the otherhand, technical literature shows that certain treatments using waterproof coatings would tend to appear effective [3] [B.Godart 1992].

CRITERIA IN THE CHOICE OF COATINGS

In order that coatings are effective against A.A.R., they must comply with a minimum of five criteria as follows :

- Impermeability to water and if necessary to water vapour ;
- Resistance to the progressive cracking of the support (the coating must be able to withstand, thanks to its elasticity, against future cracking when the A.A.R. continues to develop during a certain period after the structure has been treated);
- Durability of the coating, this characteristic covers not only the usual criteria to withstand peeling or blistering, but also the maintenance of its mechanical properties such as elasticity, withstand cracks, etc.
- Resistance to the sun ultra-violet radiation ;
- Sufficient bonding to its support

Even though thick coatings might have to be used in order to comply with the criteria, we thought it would be also useful to test, for comparative purposes, the effectiveness of more standard coatings such as paint, or products whose efficiency has already been praised in technical reports, such as silane-based impregnation products. Finally, we tested a coating using aluminium foil which appeared to us as a product which can be mentioned for reference purposes as a coating which is more watertight than others.

The list of different products used for tests is as follows :

- Water-repellent surface coating (silicones and silanes)
- Stain products and paint used in the building trade (acrylic, polyurethane, polyester, epoxy)
- Thin cement-polymer coatings

EFFICIENCY TESTS

The principle of the test is, after applying the products, to test concrete samples which are potentially expansive due to A.A.R., to measure the elongation of the samples which are kept in an atmosphere favouring the development of A.A.R.

The samples were prisms measuring 7x7x28 cm made of concrete with a mix using 600kg/m^3 Portland cement containing a 1% equivalent of Na2O and highly reactive quartzite aggregates. The samples attained maturity by retaining them at least 2 months in water at 20°C, then they were stored at 20°C in a 50% relative humidity environment for approximately 3 months. The different coatings were then applied on the samples according to the manufacturers' instructions.

After being left to dry for a week, all the samples were placed in metallic containers included a water supply to humidify the air. The containers were placed in a reactor having a stabilized temperature of 38°C and saturated with humidity.

Measurements, which include weighing and extensometry are taken before the treatment which is applied at 38°C, then the samples were measured once a month during 6 months after a 24 hour cooling period at 20°C. In addition to the coatings tested, for reference purposes tests were carried out on a bare concrete sample and two others covered with two thin layers of aluminium foil which were stuck to the samples using epoxidic resin. The results of these tests are shown in Table 1 below and figure 1 next page.

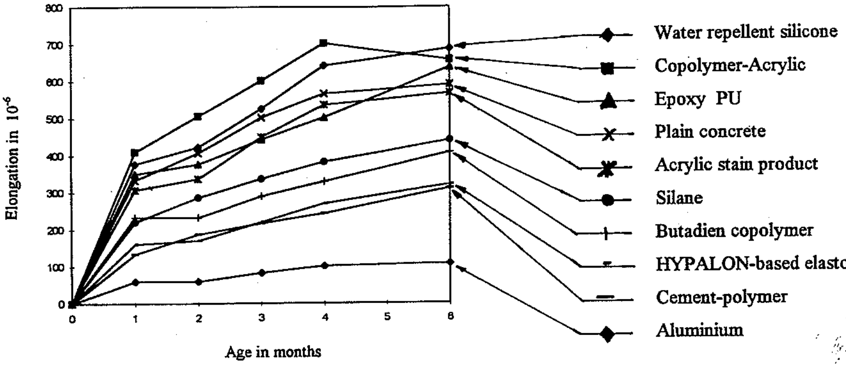
Table 1 Results obtained from measurements taken after 6 months treatment at 38°C, 100% relative humidity, filed in increasing order of elongation

N° Order	Coatings	Elongation (1) 10^{-6}	Efficiency (%)	Weight increase in comparison with original weight
1	Aluminium sheets glued together with epoxidic resin	107	82	1
2	Thin cement-polymer coating	310	47	10
3	Hypalon-based elastomer	320	46	10
4	Butadien copolymer in aqueous dispersion	407	31	20
5	Silane	440	25	20
6	Solvated acrylic stain	567	4	56
7	Bare concrete (comparison) sample	590	0	107
8	Epoxy and polyurethane paint	637	-8	30
9	Copolymer and acrylic paint	657	-11	64
10	Solvated water repellent silicone	687	-16	55

(1) Elongation reduction when compared with the reference sample N° 7

This experiment showed that a reduction in the expansion of concrete or at least its speed of evolution is possible. Apart from the experimental aluminium coating which was far more effective than the other systems and can be used as an optimal reference, relatively good results were achieved using a thin cement-polymer coating and a Hypalon-based elastomer. The effectiveness of other systems was low or even negative. A good correlation was also found between the elongation and increase of weight due to water absorption at the least for products presenting a certain efficiency. Nevertheless, it remains difficult to explain the lack of efficiency of products like paints or water repellent silicone which however reduce the humidity penetration. This experiment still confirms that it is of interest to limit the penetration of humidity in concrete and therefore to warrant the waterproofing of potentially reactive concrete structure facings.

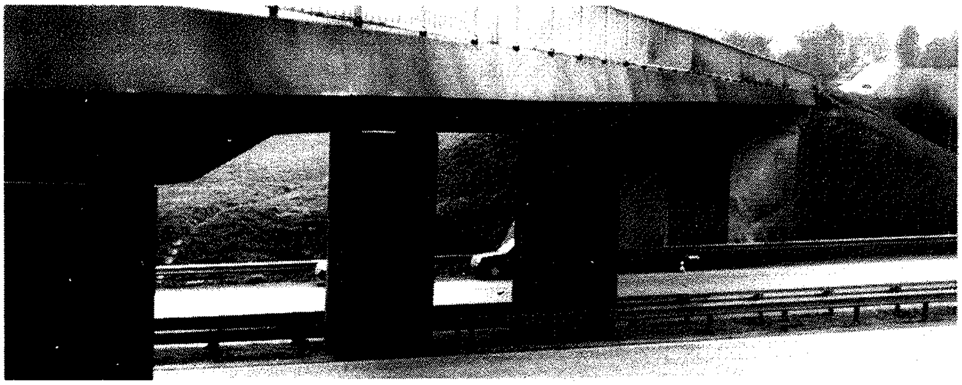
Fig. 1 : Efficiency test on the waterproof coating
(elongation of samples 7x7x28 cm at 38°C and 100% relative humidity)



EXPERIMENTAL TREATMENT OF A BRIDGE

Treatment was attempted on a bridge on the A4 motorway in the Parisian region. This bridge is part of a number of structures affected by A.A.R., two of which have already been destroyed and replaced due to the extent of the disorders. The structure, treated on an experimental basis, is a 4 span continuous reinforced concrete slab, the overall length is 69.50 meters, 10.50 meters wide and 75 centimeters thick, as shown in Fig. 2 below.

Fig. 2 General view of the structure

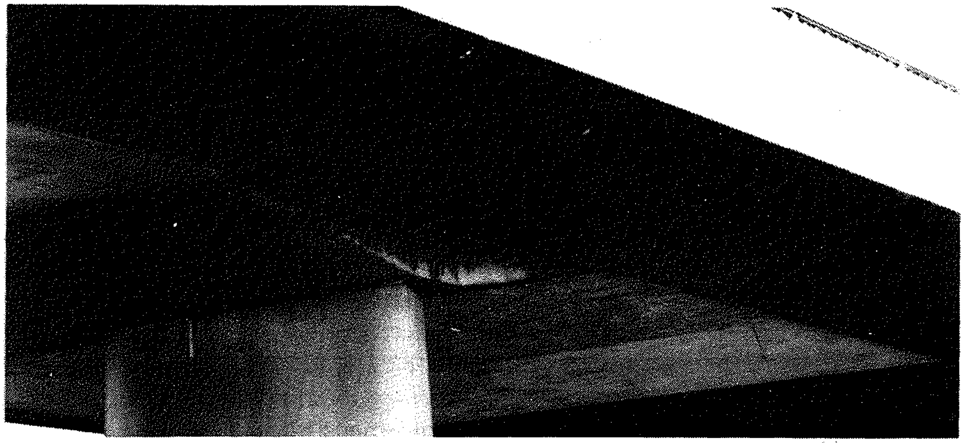


It was built in 1976 and the disorders were noted for the first time in 1986. The disorders mainly concern a network of cracks which affect both the intrados and extrados of the deck and the cantilevers in particular were cracked.
The treatment applied to this structure in 1990 par the Orion (Dune) company, was divided into three major operations :

- The first operation was to strip the deck and to do so it meant removing the carriageway, the pavement joint, the cornices and adjacent cornices, the pavement and the existing epoxy pitch seal.

- The second operation was to protect the complete upper surface of the deck : after bridging over the numerous existing cracks and trimming up the areas requiring reprofiling, using an epoxy resin mortar. A watertight coating was applied successively made up of a cold impregnation coating, a prefabricated polymer bituminous membrane, a protective mechanical layer of gritted asphalt, a coated wearing course was then applied.
- The third operation concerned the protection of the slab intrados, the piles and abutments, as well as the cornices (See Fig. 3 below), using at least two layers of a coating made from mortar which included an hydraulic binder modified by polymers, this PH12 mortar, which had to be on average 3 mm thick. is the same as the N° 2 coating of table 1.

Fig. 3 Details on the intermediate bearing - Cracking resurgence of the cantilevers, as well as efflorescence, (already noted in 1975).



The mortar included :

- a liquid component with an acrylic and copolymer base (30%)
- a pulverulent component with a cement and sand base (70%)

This product is mainly used to waterproof walls or water tanks and its main characteristics are summarized in Table 2 below.

Table 2 Characteristics of the coating (according to VERITAS Report N° DLC/79 416/2)

Type of tests	**Results**
Withstands cracking (tensile strength on the cement mortar sample was notched	2.10 mm at 23°C 0.65 mm at 0°C 0.55 mm at -10°C
Adhesion to the support (pull out test)	2.4 MPa on mortar 1.7 MPa on concrete
Permeability to water vapour resulting from the test for possible blistering by measuring the transmission of water vapour (according the AFNOR standard T30-704)	140 gr/m^2 24 hours at 38°C (on the reference sample) 40 gr/m^2 24 hours at 38°C (on the coated sample)

Other characteristics : Elongation at rupture exceeds 30% at 20°C. It withstands temperature variations ranging from -40°C to +50°C and does not produce any major variation by ageing. Finally, its alkali content is lower than 0.4% (according to standard ASTM C 150-83).

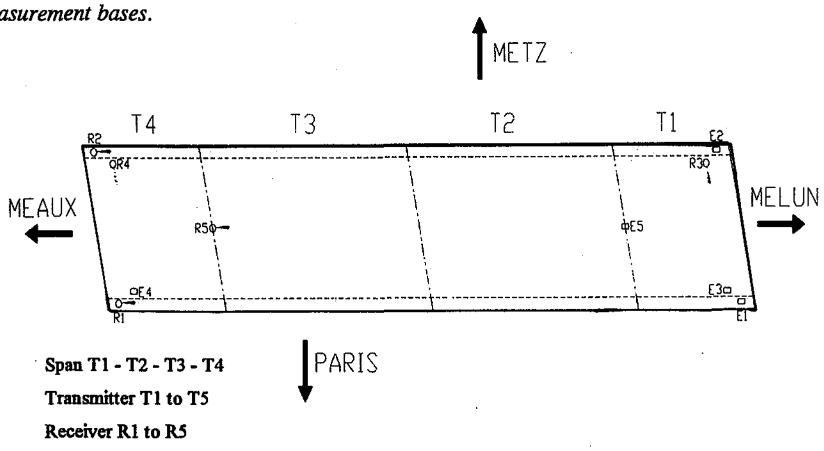
Inspection work carried out by Metram on behalf of Scétauroute concerned the quality of preparatory work on supports, the coating's adhesion properties, its thickness and conformity to the producer's specifications. The coating's adhesion values on its support are in the region of from 1.0 and 2.0 MPa on the basis of a direct tensile test.

EXPERIMENTAL MONITORING OF THE STRUCTURE'S BEHAVIOUR

Only the behaviour of the structure's deck was monitored. The principle was to measure the dimensional variation of large bases (several tens of metres) orientated according to the structure's principle axes.

Distance measurements were taken using infrared equipment, including a swivelling transmitter (E) positioned on a lockplate and one or several optical prisms receiver (R) installed on reference studs shown in Fig. 4 and 5 below. These lockplates and studs were glued and mechanically connected to the structure.

Fig. 4 : Orientation of the PS 25 bridge on the A4 motorway at Coutevroult and layout of the distance measurement bases.

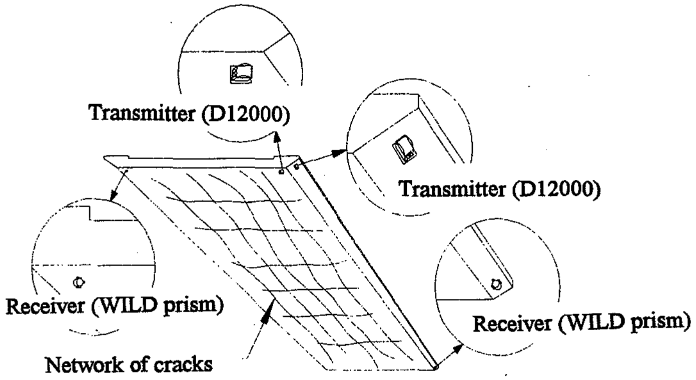


« Second generation » WILD measuring products were used. The regional Bridges and Highways Laboratory in Lille adapted the equipment for specific use on the sites. Its accuracy characteristics are : Distance type ≤ 1mm + 1 part per million for a distance of ≤ 100 m.

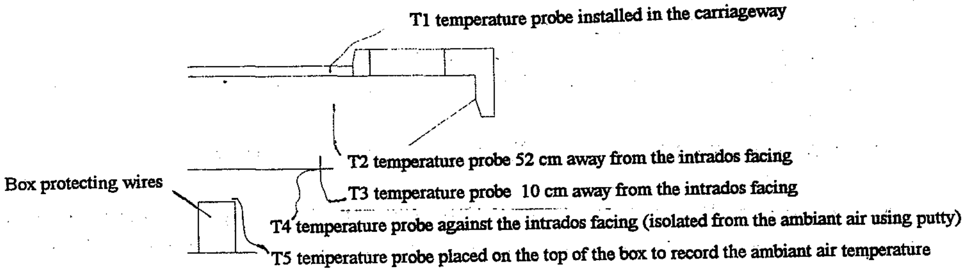
Simultaneously temperature, hygrometrical and external barometric parameters were measured as well as the temperature distribution gradient within the structure, using temperature probes embedded at different levels in the thickness of the slab. The elements enable one to calculate the means of correcting elongation or shortening values, to be applied to calculations for the analysis programme. A microcomputer and processing software for this information is also used in the measurement acquisition procedure [2] [B.Godart-P.Fasseu-M.Michel 1992].

Fig.5 : Layout of the distancemeter WILD prisms and temperature probes

Layout of the distancemeter and WILD prisms

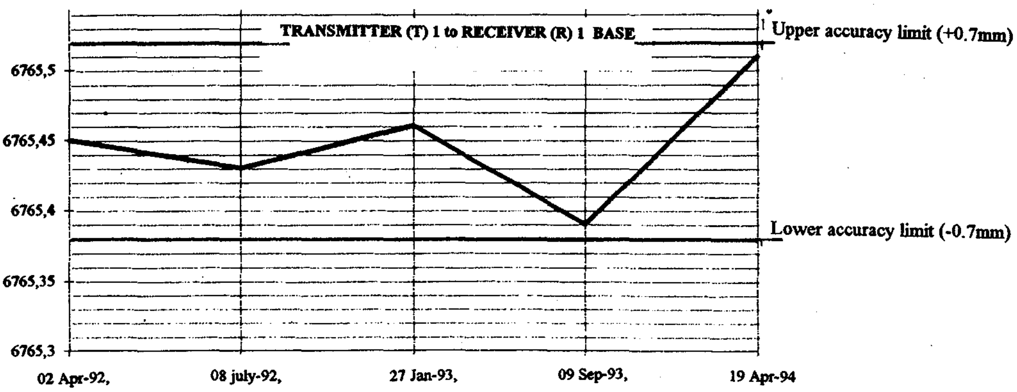


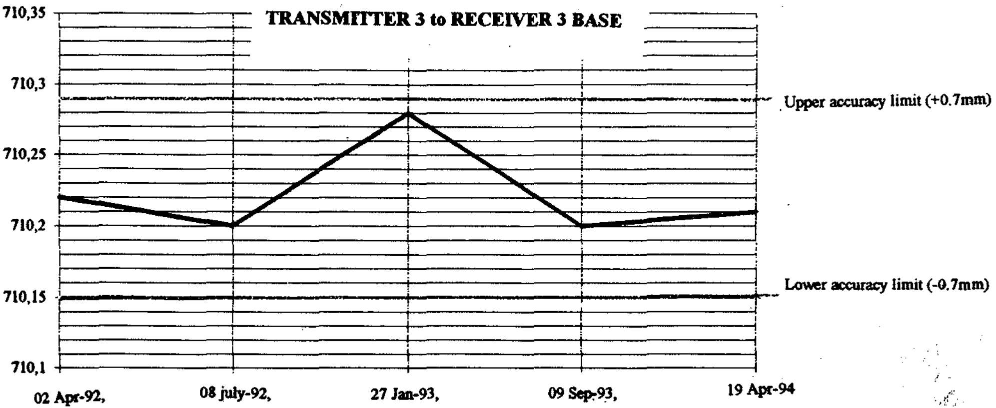
Layout of temperature probes between the slab's intrados and extrados



From April 1992 (namely approximately two years after applying the protective coating to April 1994, 5 series of measurements were taken following the principle mentioned above. Whether taken on a longitudinal or transveral axis the dimensional variations, measured in relation to zero at the start, remain mainly between a ± 0,7 mm gap, corresponding to the accuracy of the method used on site.

Fig.6 : Results of measurements of dimensional variations





COMMENTS AND CONCLUSIONS

Four years after the treatment, we are able to establish the following assessment : the coating is globally intact if one excludes some local zones covering the extrados of the pavements and the intrados zones of the cantilevers on either side of the intermediate bearings where we found :

- Reparation of the coating on the extrados zone
- Resurgence of transversal cracks observed during inspection carried out before the tests
- There was a whitish bloom (efflorescences) with the cracks (see Fig. 3).

However, due to its elasticity, it should be noted that this type of coating can hide the development of cracking for a certain length of time. The dimensional variation measurements of the limits confirm however there is no global expansion of the structure. Even if these results are encouraging, nevertheless it is still too soon to give a definite opinion on the efficiency of this treatment and even less on its durability ; the monitoring programme needs to be pursued.

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EFFECT OF VARIOUS TYPES OF SILANES ON EXPANSION DUE TO ALKALI-SILICA REACTION

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ABSTRACT

Recently, a number of examples of premature deterioration caused by alkali-silica reaction has been reported. Various types of surface treatments are applied for repair work. This paper deals with various types of silanes, which are typical hydrophobic impregnation agents, as repair materials. In the deterioration mechanism, water control is one of the most important factors. Thus, many kinds of surface treatments are applied to control the amount of water present in concrete. Surface treatments can be classified into two types from the viewpoint of how to control the water in concrete. One type allows no movement of water into or out of the concrete. The other type is a hydrophobic treatment which restricts water penetration into concrete, but allows water vapour to move out. The former system may promote concrete deterioration. Silane has been used as typical hydrophobic for concrete structures deteriorated by alkali-silica reaction. In this study, the effect of the molecular size (molecular weight: 120 - 416), the type (alkyl, alkoxyl: methoxy, ethoxy) and the number of alkoxyl groups of silane on the hydrophobicity of concrete were investigated.

Keywords: hydrophobic impregnant, molecular structure, repair, water control, weight change

INTRODUCTION

A number of cases of premature deterioration of concrete structures caused by alkali- silica reaction and/or chloride-induced corrosion of reinforcing steel have been reported. Water plays one of the most important roles in these deterioration mechanisms. Therefore, in order to avoid the deterioration, many kinds of surface treatments which can control water content in concrete are applied to concrete structures.

These surface treatments can be classified into two categories from the viewpoint of how they control the water in concrete. One type permits no water ingress into concrete, and no water to get out. Another type, the hydrophobic treatments, restricts water penetration into concrete, but allows vapour phase water to get out. Since the former may cause deterioration by the water left in concrete, the latter is a superior methods (Miyagawa et al. 1991).

In the latter system, silanes are commonly used as typical hydrophobic impregnants. Silanes are silicone-based products of low molecular weight and are used as the alkylalkoxylsilanes. This paper deals with the hydrophobic surface treatment of concrete using some types of silanes used in repair of concrete structures.

OUTLINE OF EXPERIMENT

Molecular structure of silanes

By changing the kind and number of the alkyl and alkoxyl groups of silanes, nine kinds of silanes shown in *Table 1* were prepared. These were used as 1 mol solutions in isopropyl alcohol.

Series 1: Preliminary test

Preliminary experiment were conducted on the nine kinds of silanes to select silanes used in Series 2. The specimens were small concrete prisms (W/C=0.60, 40x40x160 mm) with non-reactive aggregate. After being cured in water for three months, they were dried in air for a week and then impregnated with the silanes. The amount of impregnating silane solution was 400 cm^3/m^2 in all test series. Two days after impregnation, the specimens were placed under four different conditions-indoors, under water, in the dry and wet chamber (20C, 60%RH, 12Hr - 40C, 100%RH, 12Hr), or outdoors. The weight change of specimens impregnated

Table 1 Silanes

Name	Molecular formula **(alkyl)** *(alkoxyl)*	Molecular weight	Note
dimethyldimethoxy silane	$(CH_3)_2Si(OCH_3)_2$	120	two alkoxyls
methyltrimethoxy silane	$CH_3Si(OCH_3)_3$	136	alkoxyl is methoxy.
ethyltrimethoxy silane	$C_2H_5Si(OCH_3)_3$	150	
iso-butyltrimethoxy silane	$C_4H_9Si(OCH_3)_3$	178	
n-octyltrimethoxy silane	$C_8H_{17}Si(OCH_3)_3$	234	
n-deciletrimethoxy silane	$C_{10}H_{21}Si(OCH_3)_3$	262	
n-octadeciletrimethoxy silane	$C_{18}H_{37}Si(OCH_3)_3$	374	
methyltriethoxy silane	$CH_3Si(C_2H_5)_3$	178	alkoxyl is ethoxy.
n-octadeciletriethoxy silane	$C_{18}H_{37}Si(C_2H_5)_3$	416	

Table 2 Factors for Series 2

concrete	non-reactive without chloride, non-reactive with chloride reactive without chloride, reactive with chloride
silane	no treatment, 234, 262, 374, 416 (molecular weight)
condition	outdoors, dry and wet chamber, partially immersing in chloride solution (NaCl: 3.13wt%)

with various silanes were measured to evaluate the hydrophobic performance and to select silanes for Series 2.

Series 2: Alkali-silica expansion

The specimens were concrete prisms (W/C=0.70, 100x100x400 mm). *Table 2* shows the types of concrete, silanes, and environmental conditions. The Sc/Rc of reactive aggregate was 4.63. The total amount of equivalent alkali of the concrete was 8 kg/m^3. After being cured at 20C, 80%RH for two weeks, each specimen was impregnated with the silane selected from the previous test and exposed to the different conditions. Weight change and strain were measured. The long term performance of prisms impregnated with typical silanes were examined.

Series 3: Quasi-actual scale test

Finally, by using the quasi-actual size concrete specimens (non-reactive, W/C=0.51, 1000x1000x150 mm, *Fig.1*) placed outdoors, the effects of two types of hydrophobic treatment systems were evaluated in the state of quasi-actual condition. The applied hydrophobic systems were the silane which was selected based on the results of Series 2, and the same silane with a water vapour permeable, flexible mortar top lining with a thickness of 1.2 mm. In Japan, a mortar lining is usually adopted as top coat from an aesthetic viewpoint and to control of carbonation. The water vapour permeability of the mortar lining was 12 g/m^2.day and the elongation was 75%. PVC tubes of various lengths were embedded in the concrete to measure the relative humidity in the concrete.

RESULTS AND DISCUSSIONS

Series 1: Preliminary test

Surface treatments are expected to have the ability not only to permit little liquid phase water to penetrate into concrete but also to permit a lot of water vapor in concrete to get out. As the weight changes of indoor specimens and those of underwater specimens can be regarded as indexes of "water vapour permeability" and "water liquid permeability" respectively, a larger ratio of "water vapour permeability/water liquid permeability" corresponds to better hydrophobic performance.

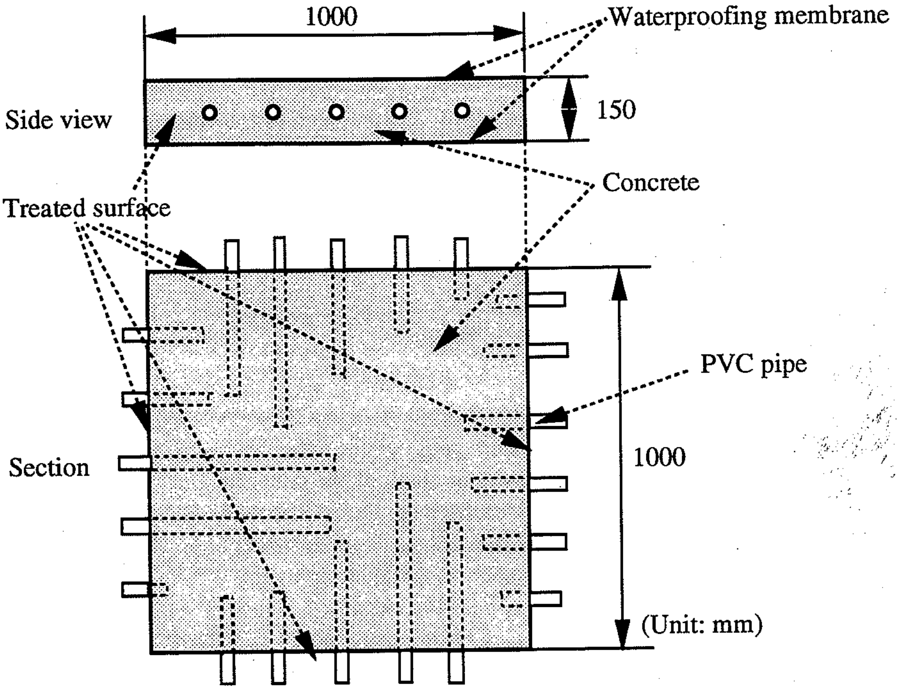

Fig.1 Qasi-actual specimen

Water vapour permeability

Fig.2 shows the relationship between molecular weight and water vapour permeability. After 9 days of exposure, the silanes of smaller molecular weight in the methoxy series had the larger water vapour permeability. The same tendency was observed in the ethoxy series. The tendency was still observed after 30 days of exposure, although the influence of molecular weight on the water vapour permeability was reduced to some extent. These results indicate that the water vapour permeability of the silanes of smaller molecular weight was generally larger than that of larger ones during the above period.

Water liquid permeability

Fig.3 shows the relationship between molecular weight and liquid water permeability. In the methoxy series, the silanes of larger molecular weight showed the lower water liquid permeability. Since the size of the hydrophilic alkoxyls were the same, this might be due to longer hydrophobic alkyls corresponding to larger molecular weight. The same tendency was observed in ethoxy series as well. The silanes of larger molecular weight had better resistance against penetration of water.

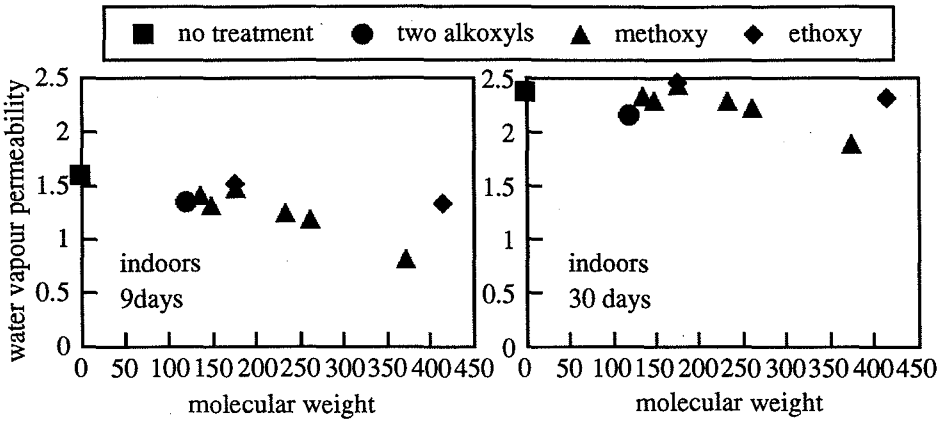

Fig.2 The relationship between molecular weight and water vapour permeability

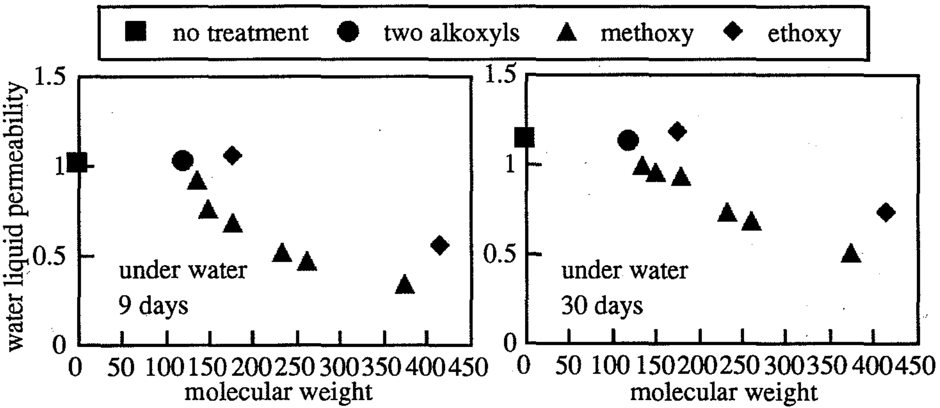

Fig.3 The relationship between molecular weight and water liquid permeability

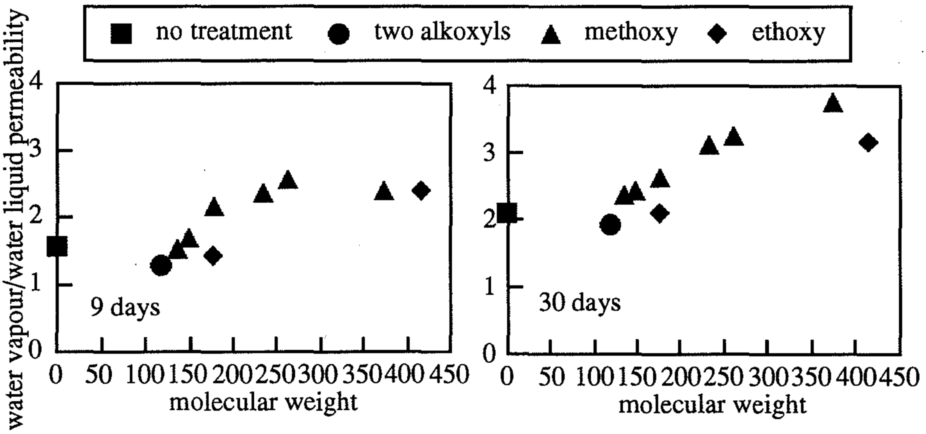

Fig.4 The effect of molecular weight on the ratio of water vapour permeability/ water liquid permeability

"Water vapour permeability/Water liquid permeability"

Fig.4 shows the effect of molecular weight on the ratio of "water vapour permeability / water liquid permeability". After 9 days of exposure, the silanes with molecular weight of 136, 150 and 178(MTES) showed smaller ratios as compared with those of the non-treated specimen, 262, 234, 374 and 416 of larger molecular weight. Similar results were obtained after 30 days of exposure, that is, the larger molecular weight resulted in the larger ratio. The silanes which showed a large ratio also indicated good hydrophobicity under dry and wet chamber and outdoor conditions. Considering that the actual concrete structures are exposed to dry and wet condition and the relatively small size of the specimens used in Series 1, the silanes used in Series 2 were selected mainly on the basis of the results of hydrophobicity in 9 days of exposure. For this reason, the four types of silanes of molecular weight 234, 262, 374 and 416 were selected. Among them, 262 was regarded to have the best hydrophobic property in this series.

Series 2: Alkali-silica expansion and corrosion

Weight changes

In all conditions, the weight change of all specimens impregnated with the silanes were smaller than that of non-treated specimens. From the viewpoint of weight change, this indicates that the silanes could also control the water content of the specimens used in Series 2, even under the condition of partial immersion in chloride solution.

Effect on expansion caused by alkali-silica reaction

Figs.5 and 6 show the expansion of concrete. Under outdoor conditions, the expansion was very small. In the dry and wet condition, non-treated specimens expanded significantly due to alkali-silica reaction. On the other hand, the specimens impregnated with the silanes expanded much less than non-treated specimens. Among them, 262 was regarded to have the best effect. The result indicates that silanes controlled the water content in the specimens and were effective against alkali-silica expansion. However, the strains of treated specimens with reactive aggregate were much larger than those of the specimens without reactive aggregate. It should be noted that unless a concrete mix was designed properly, silanes would fail to restrain concrete from excessive expansion in the long term.

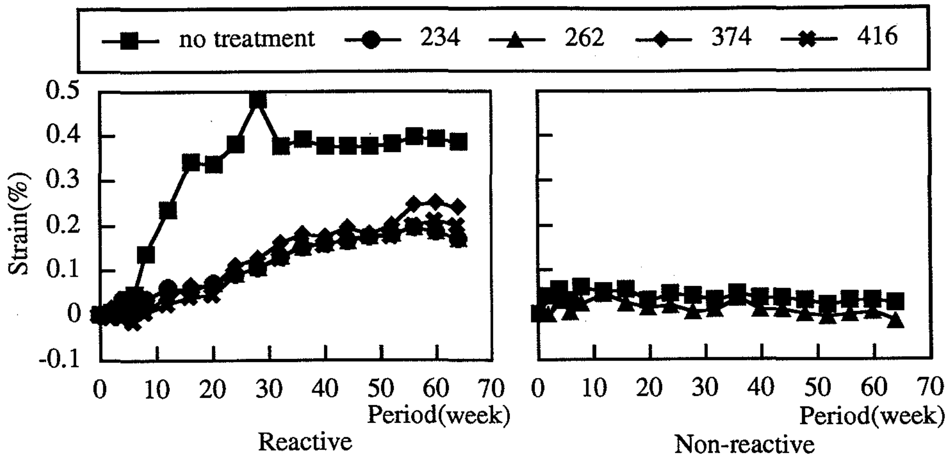

Fig.5 Expansion - Dry and wet

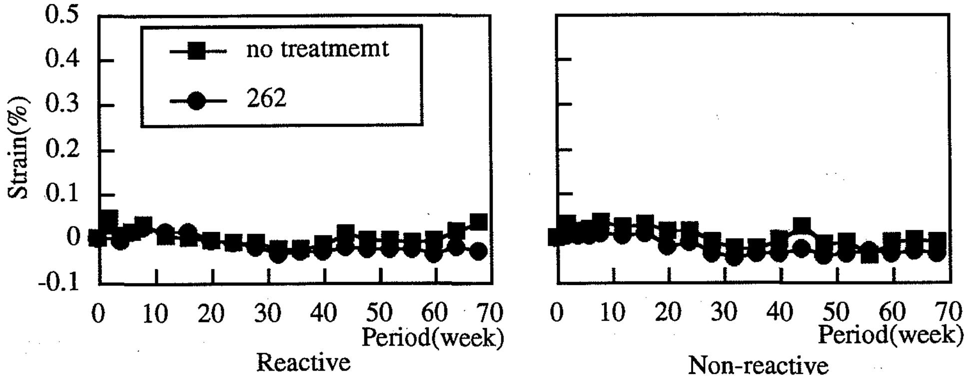

Fig.6 Expansion - Outdoor

Series 3: Quasi-actual scale test

Silane of molecular weight 262, which showed the best effect in Series 1 and the alkali-aggregate reaction series, was used in the quasi-actual scale test. *Fig.7* shows the typical profiles of relative humidity in concrete of quasi-actual scale specimens after about 6 months of exposure. The clear effects of silane systems are recognized in *Fig.6*. In the surface 10 cm layer, the relative humidity of the treated specimens was lower than that of the non-treated one. Particularly, the silane without the mortar system gave a significantly good effect. However, in the zone deeper than 20 cm of the specimen, the effect of the silane systems couldn't be recognized clearly. Therefore, in this experiment, it is thought that the effective depth of silane systems ranged between 10 cm and 20 cm.

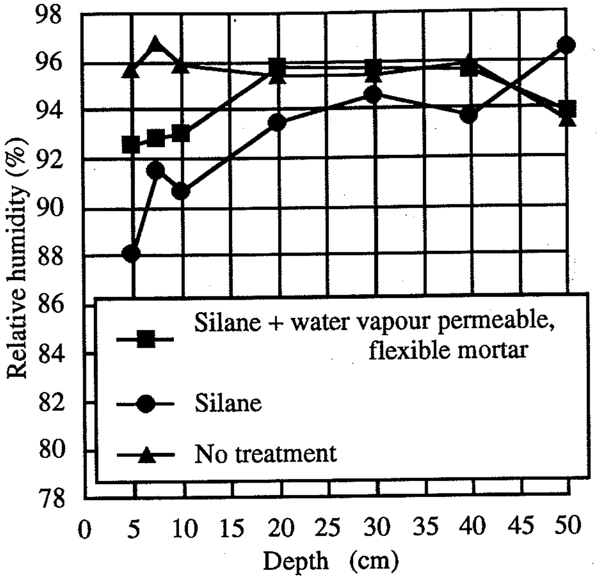

Fig.7 Profile of relative humidity

CONCLUSION

The main results obtained in this study are summarized as follows.

(1) Silanes of small molecular weight allow more water vapour to get out. On the other hand, silanes of large molecular weight which have long alkyl groups had better resistance against ingress of water.
(2) Silanes worked effectively against the expansion of concrete caused by alkali- silica reaction. By impregnating concrete with silane, expansion was reduced or even restrained. However, if concrete is excessively reactive potential, it is difficult to restrain deterioration in the long term.
(3) Under outdoor conditions in Japan, the effective depth to which silane lowers the relative humidity, is between 10 - 20 cm.

ACKNOWLEDGEMENT

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