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BASIC STUDY FOR DIAGNOSIS OF CONCRETE STRUCTURE AFFECTED BY ALKALI-SILICA REACTION USING DRILLED CONCRETE CORE

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1. INTRODUCTION

For diagnosis of reinforced concrete structures, viz. the degree of damage and estimation of development of expansion, physical properties and expansion of the drilled core taken from the existing structure are usually measured. In this case, it is well known that the static modulus of elasticity shows most notable change after damage. It is suggested that the expansion of cores under condition of 20°C, 100%R.H. shows the degree of existing damage and expansion at 40°C, 100%R.H. indicates the development of damage[1].

In this study, the influence of reinforcement, diameter of core and degree of expansion of concrete on the physical properties and the expansion of cores were examined.

2. EXPERIMENTAL PROCEDURES

2.1 Materials and Mix proportions

The material used were ordinary portland cement(equivalent Na_2O=0.63%), non alkali reactive sand(s.g.=2.59, f.m.=2.92)and gravel(s.g.=2.70, f.m.=6.65), and alkali reactive gravel judged deleterious by chemical methods (Sc=807 m mol/l, Rc=119 m mol/l, s.g.=2.55, f.m.=6.75). The mix proportion of concrete is shown in Table-1. The alkali content of the concrete was adjusted to 8 kg/m³ (equivalent Na_2O) by addition of NaCl. The reactive aggregate content were 0%, 50% and 100% by absolute volume.

Table-1 Mix proportion of concrete

Mix No.	Reactive Gravel Content (%)	W/C (%)	S/a (%)	Unit weight (kg/m³) W	C	S	G Gr	G Gn	Slump (cm)	Air (%)
1	0	50	44	176	352	783	0	1031	8.0	3.5
2	50	50	44	176	352	783	487	516	8.0	3.5
3	100	50	44	176	352	783	974	0	8.0	3.5

2.2 Experimental Method

Two types of specimens, the restrained specimens (main reinforcement ratio=1.72%, stirrup ratio=0.38%) and the unrestrained specimens (30x30x50cm), were prepared as structural model specimens as shown in Figure-1. These specimens were cured outdoors up to the age of 2 weeks and then cured 40°C, 100%R.H. Expansion and ultrasonic pulse velocity were measured for these specimens. When the curing period reached the predetermined ages (6 weeks, 39 weeks), three diameters of cores were drilled (5cm, 7.5cm and 10cm). Compressive strength (Fc), static modulus of elasticity (Es), dynamic modulus

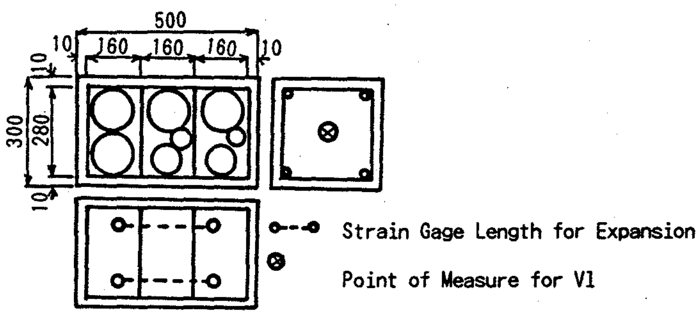


Figure-1 Size, Point of Mesure and Position of cores taken of Model Specimens

of elasticity (Ed), ultrasonic pulse velocity (Vl) and expansion were measured for these cores. After the core was drilled, expansion was measured under conditions of 20°C, 100%R.H. for 4 weeks and then at 40°C, 100%R.H. until expansion terminated.

3. RESULTS AND DISCUSSION

3.1 Expansion and Ultrasonic Pulse Velocity of Model Specimens

The relationship between expansion of model specimens and the curing period at 40°C, 100%R.H., is shown in Figure-2. It shows that the expansion of unrestrained specimens is $1700 \sim 3000 \times 10^{-6}$ larger than that of restrained specimens after the curing period of 39 weeks.

The relationship between relative ultrasonic pulse velocity of the model specimens and the curing period at 40°C, 100%R.H., is shown in Figure-3. The relative velocity of restrained model specimens is about 5% smaller than that of unrestrained model specimens after the curing period of 39 weeks.

3.2 Physical Properties of Cores

Measures of expansion and physical properties of drilled cores are shown in Figure-4. The condition of restraint of model specimens and the diameter of cores had little influence on the physical properties of cores. The relationship between the expansion of specimens and the physical properties of cores is shown ih Figure-5. Compressive strength increased until the

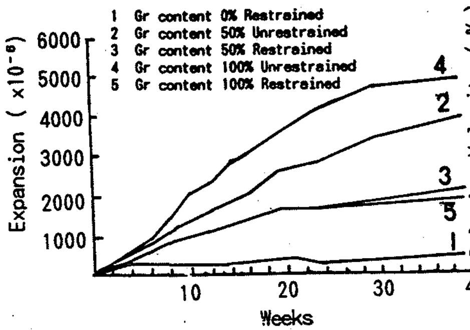


Figure-2 Expansion of Model Specimens and Curing Period at 40°C, 100%R.H.

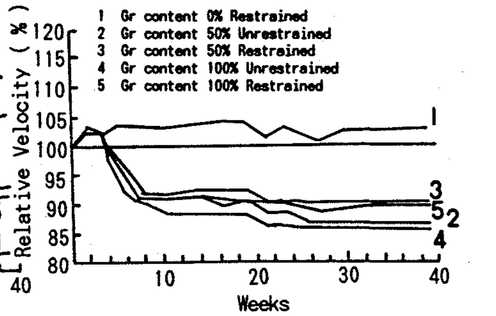
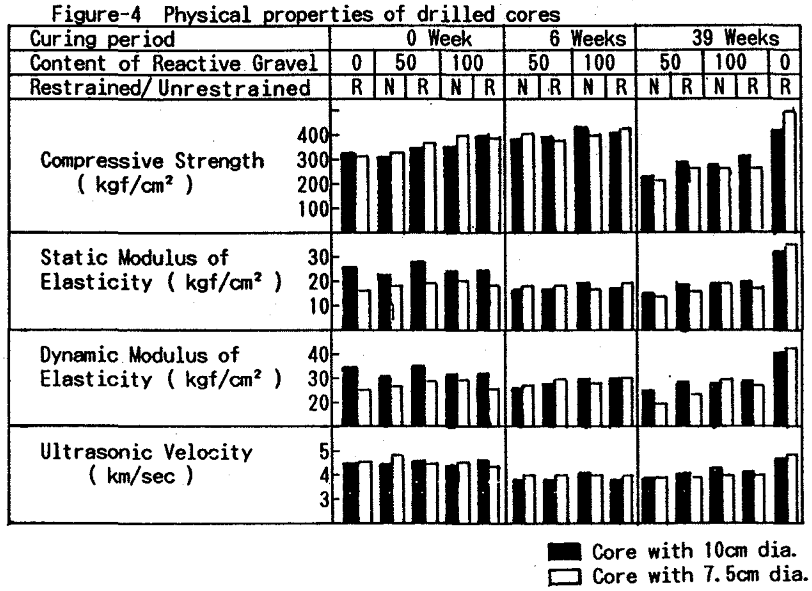


Figure-3 Relative Ultrasonic Pulse Velocity of Model Specimen and Curing Period at 40°C, 100%R.H.

Figure-4 Physical properties of drilled cores



expansion reached to about $1000x10^{-6}$, and then decreased remarkably.
But when the expansion exceeded about $2000x10^{-6}$, it gradually decreased. On the other hand, other physical properties decreased dependent on expansion, especially the static modulus of elasticity.

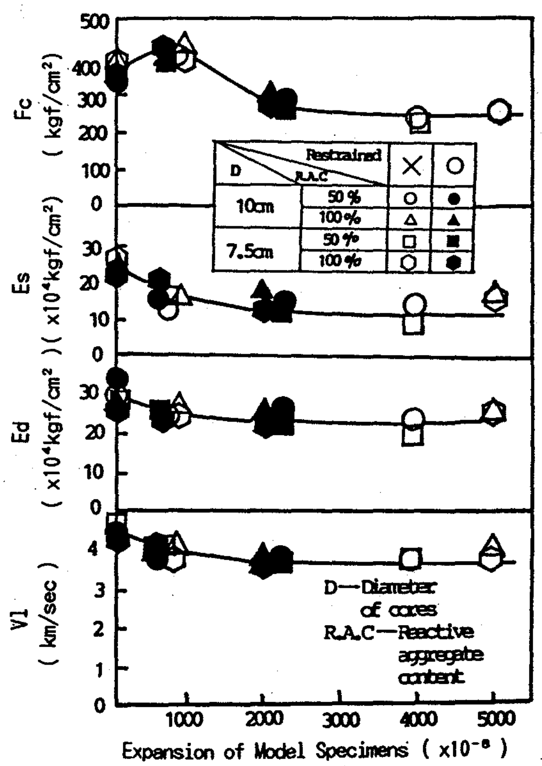


Figure-5 Relationship between Expansion of Model Specimen and Physical Properties of Cores

3.3 Expansion Test

3.3.1 Expansion of cores at 20°C, 100%R.H. The expansion of cores at 20°C, 100%R.H. is shown in Figure-6. These results indicated that the expansion of cores from restrained model specimens was larger than that from unrestrained specimen, but the expansion of cores held at 20°C,100%R.H. did not increase in spite of expansion of model specimens.

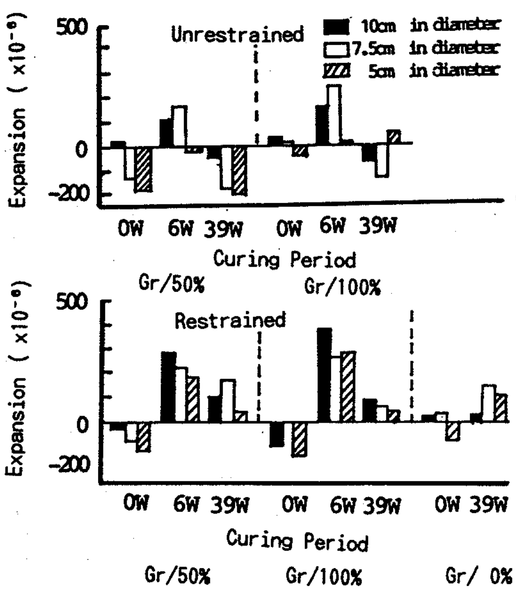


Figure-6 Expansion of Cores at 20°C, 100%R. H.

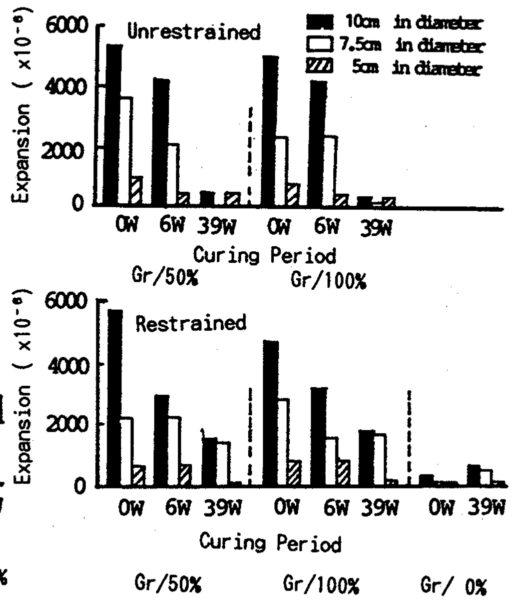


Figure-7 Expansion of Cores at 40°C, 100%R. H.

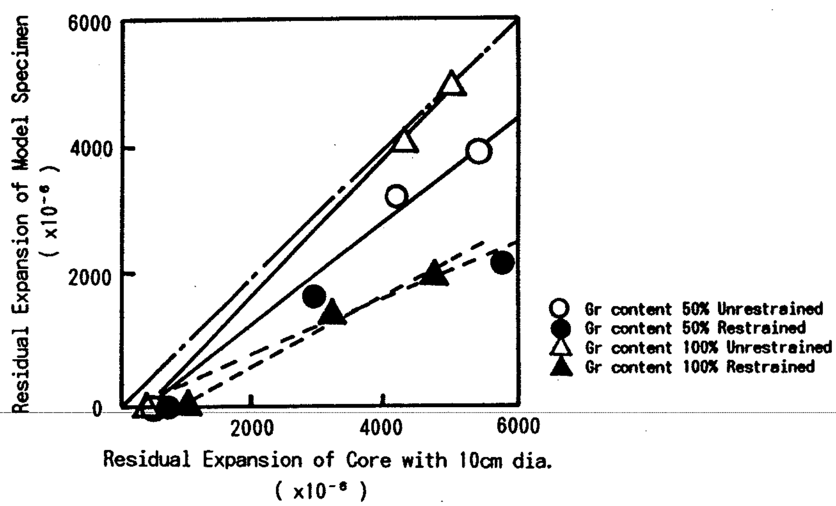


Figure-8 Relation between Residual Expansion of Cores and Model Specimens

3.3.2 Expansion of cores at 40°C, 100%R.H. The expansion of cores at 40°C, 100%R.H. (residual expansion) is shown in Figure-7. It increased with increase of diameter of cores and decreased with increase of age drilled.

The relationships between residual expansion of cores with 10cm diameter, and residual expansion of model specimens after cores were drilled, is shown in Figure-8. Residual expansion of cores from unrestrained model specimens were approximately equal to residual expansion of the model, but in the case of restrainted models, residual expansion of cores showed three times as much residual expansion as the restrained model.
These results suggest that, in the future, the degree of damage of an existing structure could be estimated by measuring the expansion of cores taken from the strucure after consideration of the reinforcement ratio.

4. CONCLUSIONS

1. Alkali-silica expansion of restrained concrete with reinforcement was aproximately half that of unrestrained concrete.

2. Physical properties, i.e, compressive strength, static and dynamic modulus of elasticity and ultrasonic pulse velocity, of cores decreased with increases of expansion of the model specimens, changes in the static modulus of elasticity were especially remarkable.

3 It is suggested that the future damage of existing concrete structures affected by ASR could be estimated from drilled cores.

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AN ENGINEERS PERSPECTIVE ON U.K. EXPERIENCE WITH ALKALI–AGGREGATE REACTION

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1. ABSTRACT

This paper reviews the developments of studies on AAR in the UK where the reaction has been found since the 1970's to be developing in a substantial number of buildings, bridges and water retaining structures. UK developments in specification to minimise the risk of ASR are presented in the context of their impact on construction practice. The evolution of research on materials, for diagnosis for structural assessment and to quantify the physical severity of the reaction, is discussed. Gaps in our knowledge which necessitate research are identified.

2. INTRODUCTION

The reaction of aggregate with alkali from cement in concrete was first identified in 1940 in a bridge structure in the USA. Alerted by this, UK studies started at BRS in the 1950's [1] to consider and evaluate the risks of a wide range of land aggregate sources with the then available cements. However, the cautions about checking new materials in these research reports were not carried through into BRE Digest 126 [2] which stated that AAR wasn't a UK problem. In 1971 Val de la Mare Dam in Jersey was found to have AAR [3], but that island was nearer France than the UK and the aggregate was unusual, so the mainland UK remained off guard.

It was in the South West of England that the first cases on the UK mainland of Alkali–Silica Reaction (ASR), were diagnosed in 1975. These were CEGB Transformer Station foundations and Charles Cross Car Park [4]. Since then a wide range of UK structures have been found to be suffering from Alkali Aggregate Reaction. There is a cluster of structures in the South West, others in the Trent Valley and isolated cases throughout the UK including one in Northern Ireland, an area which was thought to be immune until the Autumn of 1988!

In the South West (S.W.) Charles Cross Car Park and many of the other major structures with the reaction were built in the late 60's or early 70's using Plymstock Cement. At that time it had a high alkali content of over 1.2% sodium oxide equivalent (Na_2Oeq), giving alkali levels of over 6kg/m^3 in some cases where high cement contents (400–500 kg/m^3) were used. High alkali mixes have not caused problems where the aggregates are free of reactive proportions of silica. The main form of reactive aggregate in the S.W. has been found to be chert contained in a sea dredged material, used as fine aggregate. When mixed with either limestone or granite coarse aggregate, which are free of reactive silica, this gives a 'pessimum' proportion of reactive silica, with about 5% chert. This combination of aggregates with the old high alkali Plymstock cement has caused some of the most severe damage from AAR. Core expansions can exceed 2.5mm/m and cracking is often over 1mm wide. A number of the structures with this mix, which are being monitored, show the movement of cracks from the ASR continuing after nearly 20 years. Annual rates of crack width growth of over 0.1mm/year are not uncommon.

Studies on other structures in the S.W. have shown that a range of other aggregate types have produced AAR to damage the concrete. These include both aggregates containing cherts and quartzite giving Alkali Silica Reaction (ASR) and aggregates containing greywackes and argillites which produce an Alkali Silicate Reaction. Alkali Carbonate Reaction, the other variety of AAR, is still not thought to be a problem in the UK.

Similar problems are arising in the Midlands, Trent Valley, aggregates but with somewhat lower cement alkali levels. Structural damage and expansions are generally less severe and develop slowly. Trent Valley Aggregates are an all in one alluvial material with cherts and slower reacting quartzites.

The development of damage from AAR raised a range of questions for the owners of structures. They have put these to the engineering profession and, to the construction industry including cement and aggregate suppliers as follows:–

1. How are you going to prevent the reaction in new construction?
2. How do I know if my structure is free from the reaction?
3. If the structure has the reaction, does it reduce its safety or serviceability?
4. How much extra maintenance expenditure will be required?

The publication of the reports in 1987 and 1988, by the Concrete Society Hawkins Committee [5], Building Research Establishment [6] the Cement & Concrete Association/British Cement Association Palmer Working Party [7] and the Institution of Structural Engineers Doran Committee [8] provide balanced professional views on the current state of the art on these questions. However, the preparation of the documents and their application in industry have highlighted gaps in our knowledge and the need for further research.

3. MINIMISING THE RISK FOR NEW CONSTRUCTION

The Concrete Society Committee, chaired by Michael Hawkins of Devon County Council, on 'Alkali Silica Reaction – Minimising the Risk of Damage to Concrete' has made a major contribution to developing the understanding of ASR in the UK profession through conferences and publishing guidance. The original 1983 guidance [9], which was developed from BRE advice in 1982 [10], was based on consideration of the three essential components needed for alkali silica reaction.

i) The amount of alkali available
ii) The amount of reactive silica in the aggregates
iii) The availability of moisture

The view was taken that the majority of concrete structures have a proportion of their elements sufficiently damp for there to be a risk of the reaction, therefore control must normally be based on either limiting alkali levels or the selection of the aggregates. The majority of UK aggregate sources contain some minerals of potentially reactive types. The difficulties of evaluating UK aggregates to ensure freedom from reactive proportions of silica were such that the main emphasis was put on limiting the alkalis. Two approaches were adopted 0.6% and $3kg/m^3$. The first was based on the old American procedure of specifying cements with an alkali content guaranteed below 0.6%; however this cannot be achieved in the UK with OPC, although SRPC cement is supplied with guaranteed low alkalis. The other was the $3kg/m^3$ rule limiting the amount of alkali in the mix. The use of pfa and slag was put forward as a method of diluting alkali levels.

In 1985 a draft revision [11] to the original document was produced bringing in some new research results and including draft specification clauses. The 1983 Hawkins report had mainly considered the alkalis from the cement and discounted alkali contribution from pfa, slag, admixtures, or aggregates when used in the mix. In 1985 the scientific view had moved on and the water soluble alkalis from pfa and slag and alkali from sodium in NaCl in aggregates as well as alkalis from admixtures and water had to be considered. Following a period of discussion and public comment this 1985 draft was developed into what has been referred to as "Hawkins 3" which was published by the Concrete Society in November 1987 [5]. However, publication did not stop the controversy. The opinion of the cement makers [12] is that a higher proportion of the alkalis in pfa and slag might become available in the mix, but research data is uncertain. The Hawkins Committee recommendations have not been revised, but fuller research co-ordinated by Dr Nixon at BRE has been initiated to provide data. Those making concrete may well be inclined to maintain their legal position by following the cement manufacturers recommendations when using OPC with slag or pfa.

The BRE updated their recommendations on AAR as BRE Digest 330 in March 1988 [6]. This provides an excellent and balanced explanation of the reaction and its potential consequences. It acknowledges the cement makers view and, in the absence of clear scientific evidence, suggests that their pfa ($^1/6$ total alkali) and slag ($^1/2$ acid soluble alkali) recommendations should be followed. The other difference between the 1985 Hawkins document and the 1987 version is that the definition of the $3kg/m^3$ has been revised and slightly relaxed. However this tends to be balanced by the tighter quality control now applied to the variability of alkalis in UK cements.

The Department of Transport (DTp) [13], which has pioneered UK specification against ASR, adopts slightly more rigorous rules. Hawkins 1987 bases alkali limits of cements on the monthly average alkali content, while the DTp uses the average plus 2 standard deviations. Hawkins puts a qualified 5% limit on cherts, but DTp uses a 2% limit. Many cases of ASR have chert proportions in the mix of the order of 5%. However, in considering the difference between the DTp approach and Hawkins it must be remembered that bridge structures are particularly liable to AAR because of the damp conditions and risk of salt ingress. They fall into the category classified as 'more vulnerable construction' in clause 4.3 of Hawkins, which may require extra precautions.

The title of Hawkins clearly states "Alkali–silica Reaction – Minimising the Risk of Damage to Concrete". The structures where serious damage has occurred generally have substantially higher levels of alkali than the $3kg/m^3$ limit so it will substantially reduce and minimise occurrences of AAR damage. One cannot be sure that it will eliminate AAR. It does not consider alkali–silicate or alkali carbonate reaction nor are rules available for aggregates containing opal for which lower alkali limits are necessary; fortunately opal is rare in UK aggregate sources. The alkali levels of UK produced cements are now substantially lower than they were in the early 70's. For example Plymstock which was over 1.2% is now typically of 0.8% alkali content. Thus it is not difficult to achieve the limits of alkalis set out in the Hawkins document, provided that water/cement ratios are kept down to achieve the strength required. The somewhat lower cement contents appropriate to minimising the risk of AAR have beneficial effects in reducing shrinkage and excess thermal effects. Similarly the introduction of pfa or slag into mixes to assist in the control of AAR can (provided they are well cured for a sufficient period) give benefits from the less permeable microstructure of the concrete.

British Standards are being developed for concrete prism expansion tests and to cover guidance on petrographical examination of aggregates. This will in time enable more emphasis to be put on selecting non–reactive aggregates, as a means of controlling AAR, as is the practice in USA, Canada and Japan. The high proportion of aggregate sources containing flints, chert and quartzites makes the non–reactive aggregate option difficult in the UK, but it is used on some major structures where the scale and importance of works justify the detailed petrographic inspection and testing.

While AAR control is now recognised as an important part of concrete specification the three main priorities in ensuring durable concrete are the avoidance of:

i) Inadequate cover.
ii) Low grade concrete, poorly compacted and cured leading to premature spalling from carbonation triggered corrosion.
iii) Chloride induced corrosion.

4. DIAGNOSIS OF AAR

The well publicised cases of AAR have to a degree produced a hypochondria and a demand for diagnostic checks for AAR. A wide range of methodologies have been suggested. The Palmer Committee (the C & CA/BCA Working Party, chaired by Dennis Palmer) has recently produced "The Diagnosis of Alkali Silica Reaction" [7]. This sets out the range of tests available for diagnosis from which a cost effective and appropriate set can be selected by the Engineer for a particular structure. Normally AAR checks are only desirable if:–

a) the structure is showing unusual cracking. The checking of the possible causes of cracking should follow the approach in the Institution of Structural Engineers 'Appraisal of Existing Structures' [14] and Concrete Society, TR22 'Non-structural Cracks in Concrete' [15]. ASR should be considered as only one of many possible contributory causes of cracking.

b) the structure is known to have mix characteristics which are similar to those in other cases of AAR.

The Palmer Committee document enables the Chartered Engineer checking the structure to commission appropriate tests from experienced materials laboratories. If AAR is present in the structure on the basis of these diagnostic tests, its structural significance should be considered on the basis of the Institution of Structural Engineers Doran report on Structural Effects of ASR [8].

5. THE STRUCTURAL EFFECTS OF ASR

In December 1988 the Institution of Structural Engineers published 'Structural Effects of Alkali-silica Reaction, Interim Technical Guidance on Appraisal of Existing Structures' [8] and it is summarised by David Doran and John Moore [16] at this Conference. A study tour [17] of research centres and structures in Japan greatly assisted in the evolution of the report.

In our experience the majority of cases where petrography indicates AAR can be categorised as having little structural significance, either because the magnitude of expansion from the reaction is small or the structure is robustly detailed with good reserves of strength to make it insensitive to the effects of the reaction. Remedial works are required only in those parts of the structure with:

i) the most severely expansive material and/or
ii) highly stressed elements and/or
iii) in damp conditions and/or
iv) lacking well anchored 3-D reinforcement.

However, any above ground structure with the potential for expansive AAR can benefit from a degree of cladding and improved drainage to help dry it.

The Institution of Structural Engineers Report [8] is an interim document and provides a reasonable basis for assessing those structures which are well detailed and in which the degree of cracking from expansion to date and potential for further expansion is small. For the structures where there is a more substantial problem, specialist investigation is recommended. A number of new techniques are being evolved both in the investigation of particular structures and in research on laboratory concretes now underway at BRE, The University of Birmingham, Plymouth Polytechnic, BCA etc..

Mott, Hay & Anderson, Special Services Division working with the local offices of the Mott MacDonald Group, have carried out detailed studies on over 100 structures with AAR in the UK and overseas. Cases include bridges, dams, water retaining structures, a multi-storey car park and multi-storey commercial and public buildings. The specialist techniques developed have been set out in a series of papers which are referenced in [18]. Expansion tests are discussed in the Palmer Report and form the basis of the Institution of Structural Engineers grading. Our tests for expansion cover temperatures from 38°C to 5°C with and without restraint, freeze/thaw and site exposure. The interpretation includes consideration of the variability of expansion in concrete and its relationship to weight change from moisture uptake. Coring and sampling techniques are particularly important, if sufficient representative samples are to be obtained without damage to the structure. Surface grinding of local areas of concrete, so that aggregate types can be determined at low cost with little damage to the structure, has greatly simplified the checking of the materials in structures.

We have been developing a stiffness damage test to provide a quantitive measure of the degree of microcracking damage to the structure of the concrete. This is linked to strength testing for the compressive and tensile strengths of concrete which is combined with expansion data in finite element analysis of effects to compare with large scale tests. We have also evolved techniques of measurement of insitu relative humidity in concrete to evaluate its effect on the rate at which cracking from the reaction develops. Tests are in progress to evaluate the effects of coatings and exposure conditions on the rate of expansion and changes in moisture uptake of concrete samples.

Using these techniques the majority of the structures we have assessed have been classified in the mild range for which management can be based on:–

i) Improvements to waterproofing.
ii) More frequent monitoring of the structure.
iii) Additional protection to prevent secondary deterioration due to corrosion and frost action.

Other structures have needed a degree of strengthening of weakened sensitive details. In some instances this has been made necessary by low reserves of strength inherent in the original design or standard of construction. Structures with AAR often have a number of other peculiarities (eg. increased loading, thermal cracking, badly placed steel etc) which need to be dealt with in the overall structural management. This is one reason why the work should be handled on an engineering basis with material science support rather than purely as a materials and testing study.

6. RESEARCH

From the perspective of Practising Engineers the following are of the highest priorities for the UK research on Alkali–Silica Reaction. We would hope that co–operation and exchange of information with our colleagues around the world will help these endeavours:

a) Specification.

i) The better fundamental understanding of the sources of alkali in concrete over the 100 year timescale at normal temperatures and environments from cement pozzolans, aggregates and sodium chloride, including the effects of migration of alkali within the concrete.
ii) Improved assessment and classification of source mineral deposits for aggregates using petrography, concrete prism expansion tests and chemical testing to categorise clearly non–reactive materials and the long term safe limits of alkali appropriate to the UK types of silica, silicate and carbonate reactive minerals.

b) Management of existing structures.

For the existing stock of structures with alkali–silica reaction the long term priorities are as follows:

i) A better knowledge of the rate of development and timescale of the reaction in large members subject to low fluctuating temperatures and limited water supply. These conditions apply to actual structures, and are distinct from the artificial conditions in the majority of research.
ii) Improving techniques of monitoring the behaviour of structures in the field.
iii) Prediction of the rate of deterioration from primary alkali–aggregate reaction with secondary deterioration from frost action and corrosion initiation down cracks, especially in structures subjected to de–icing salts and in marine conditions.
iv) Determining changes to the strength and ductility of structural elements with the more sensitive types of detail identified in the DABI paper [19] and commonly found in 1960's and 1970's UK concrete structures with the reaction. This includes establishing the sensitivity of structural behaviour to the load and restraint applied while the reaction is developing.
v) Further developing the quantification of physical changes in AAR affected concrete in a way which can be input into finite element analysis for comparison with structural testing.

7. CONCLUSION

We now know how to minimise, but not eliminate, the risk of alkali silica reaction in new structures on the basis set out by the Hawkins Committee and the DTp. Techniques of diagnosis are available and can be used in a balanced and cost effective way to identify structures which may be at risk of damage from AAR. In the small proportion of structures which have significant AAR, detailed overall assessment of the behaviour of the structure in terms of material and structural behaviour can be carried out. This provides a basis for maintaining the function of a wide range of structures at a reasonable cost consistent with minimising disruption. However there is still a need for further research to provide the most cost effective approaches to specification and management of structures.

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CONCRETE STRUCTURES DAMAGED BY ALKALI-SILICA REACTION

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1. INTRODUCTION

Alkali-aggregate reaction (AAR) had been believed not to occur in Japan. Since 1982 when AAR was recognized in a pier of a bridge, vigorous efforts have been made to detect damaged structures by AAR. As the results, remarkable number of damaged structures by AAR have been detected mainly in the area of southwest of Japan. This paper introduces the examples of the damaged structures and the results of investigations made for these structures.

2. METHOD OF INVESTIGATION

Damaged concrete structures were detected by inspection. Investigations were carried out mainly on core drilled from these structures. These investigations included measurement of core expansion at 40℃ and 100% RH, rock identification by mineralogical inspection, measurement of alkali and chlorine content, measurement of crack width and depth, checking of corrosion of reinforcing bars, compressive strength and Young's modulus of concrete.

3. RESULTS OF INVESTIGATION

3. 1 Damaged concrete structures

Figure 1 shows location of damaged structures investigated.
The investigations were carried out at 47 locations mainly in the southwest of Japan. Table 1 shows types of structures at these locations and Table 2 shows the construction time of these structures.

Figure 2 shows the production and usage of aggregate in Japan.
The appearance of AAR in Japan at late 1960 seems to be in accordance with rapidly increased usage of crushed stone due to the lack of river gravel and due to increased demand of aggregate for concrete, in addition to the increase of alkali content in concrete due to the increase of cement content by adoption of pump placing, due to the use of sea dredged sand and due to the increase of alkali content in cement itself. Most of the alkali aggregate reaction occurring in Japan was found to be alkali-silica reaction (ASR).

Figure 3 to Figure 11 show the examples of damaged structures. Figure 3 is a typical example of ASR cracks in a reinforced concrete beam. Main cracks are developing in the horizontal direction. This beam was repaired 4 years ago by an ordinary coating. These cracks include cracking of the coating itself. Figure 4 shows cracking in a bridge column where vertical cracks are developing. Figure 5 shows cracking in an abutment where map like cracks are developing. Figure 6 shows cracking in a retaining wall where horizontal cracking is dominant. Figure 7 is an example of irregular cracking in a sea defence without reinforcement. Figure 8 is an example of cracking in a building. Figure 9 shows cracking in steps where the axial cracking is dominant. Figure 10 shows an bridge pier. More cracks are developing at the unsheltered part since this part is exposed to direct sunshine and rain drop. Figure 11 is deformed parapet and cracks are also developing.

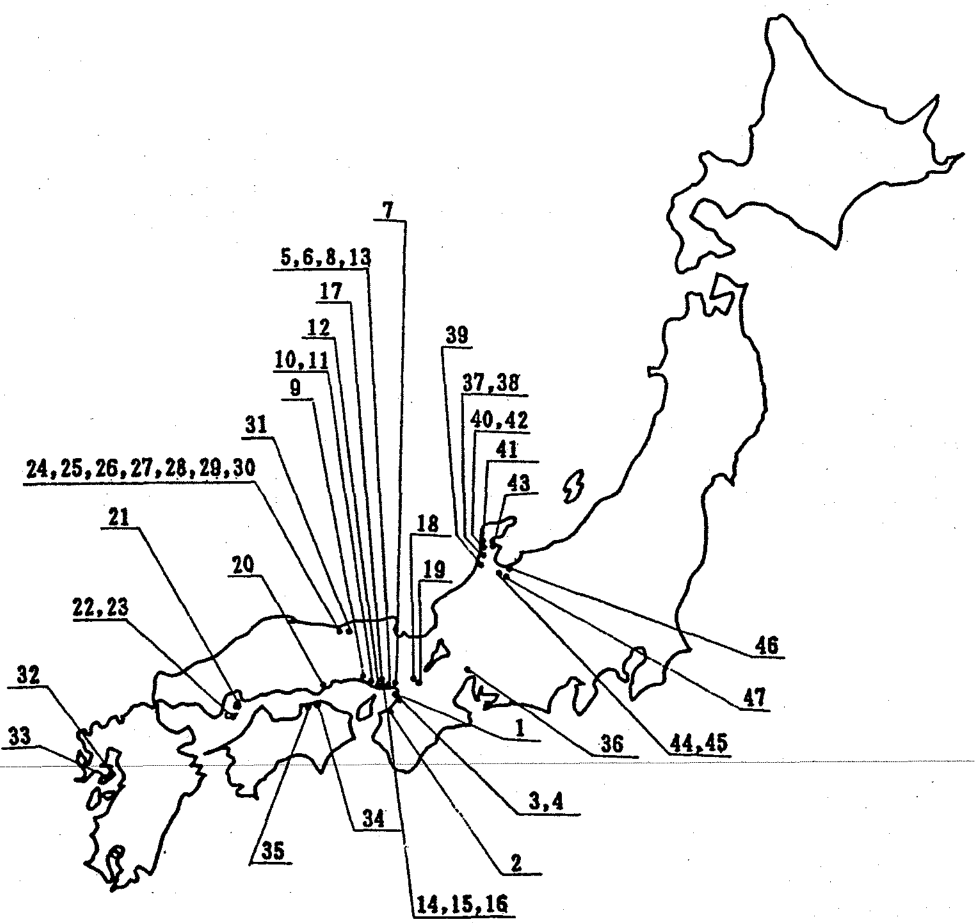

Figure 1 Locations of damaged structures

Table 1. Type of the damaged structures

Type of structure	Number of locations
Abutment Pier	28
Concrete defence Retaining wall	4
Tunnel Box culvert	5
Building	4
Others	6

Table 2. Construction time of the damaged structure

Construction	Number of locations
1966～1968	2
1969～1973	25
1974～1978	11
1979～1983	4
unknown	5

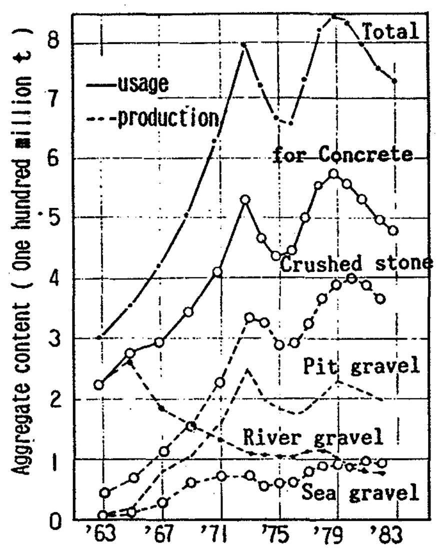

Figure 2 Production and usage of aggregate in Japan [1]

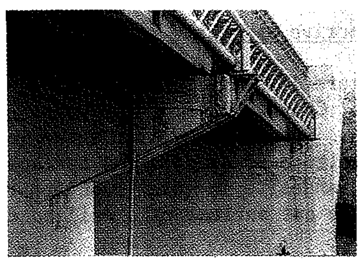
Figure 3 Pier

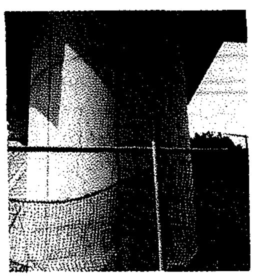
Figure 4 Column


Figure 5 Abutment

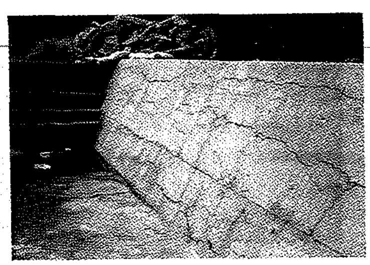
Figure 6 Retaining wall

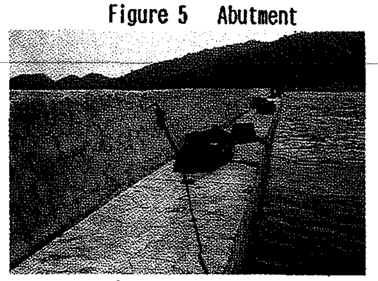
Figure 7 Sea defence

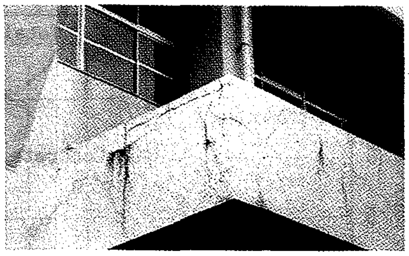

Figure 8 Building

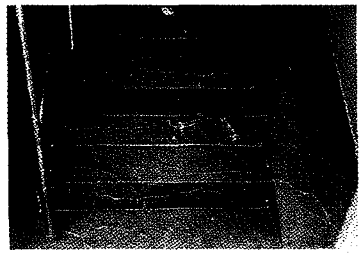

Figure 9 Steps

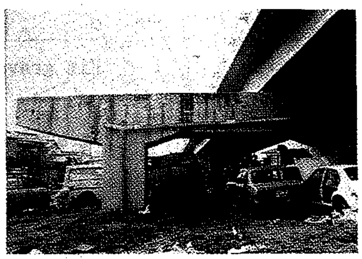

Figure 10 Pier

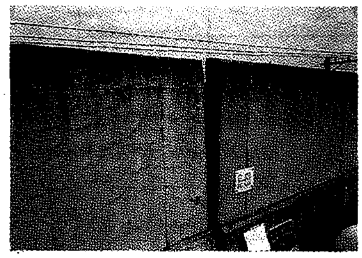

Figure 11 Parapet

3. 2 Reactive aggregate

Figure 12 shows reactive aggregate found in the investigations. Bronzite andesite, Chert, Slate, Tuff, Opal, etc. have been found in the damaged structures in Japan. Bronzite andesite seems to be the most popular reactive aggregate in Japan. Figure 13 shows a cut surface of core drilled from a damaged structure by Bronzite andesite in which reaction rims can be seem. Figure 14 shows the exuded gel at the cut surface of a drilled core.

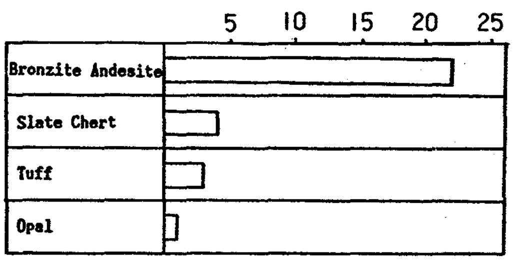

Figure 12 Reactive aggregate

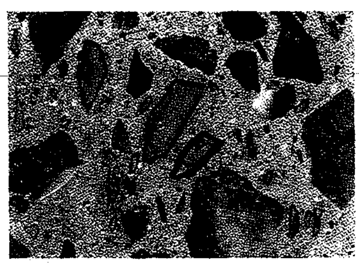

Figure 13 Reaction rim

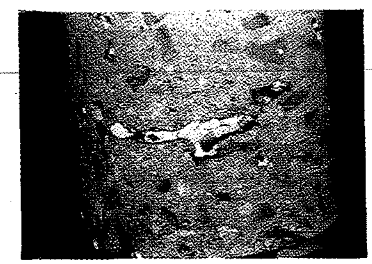

Figure 14 Exuded gel

3. 3 Expansion of core

Figure 15 shows the total and residual expansion of cores obtained at 21 locations among 47. The total expansion ranged from 160 to 2000×10^{-6} and the residual expansion ranges from 0 to 1400×10^{-6}. The average total expansion was 770×10^{-6} and the average residual expansion was 450×10^{-6}. It should be noted that some of the structures still have high potential of expansion even at the age of 15 years.

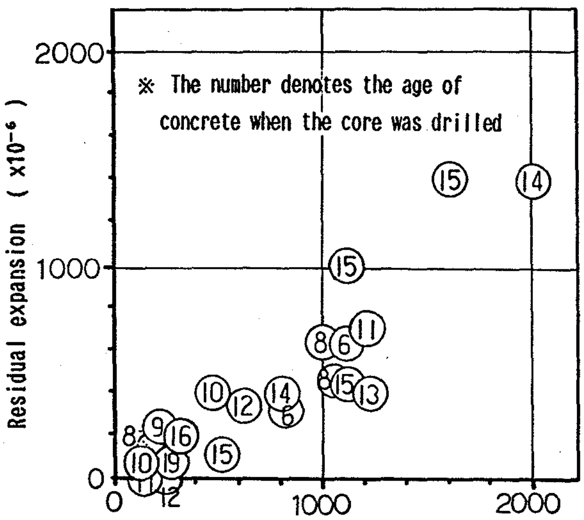

Figure 15 Expansion of cores

3. 4 Alkali and chlorine content in the concrete

Figure 16 shows the amount of water soluble Na_2Oeq obtained from cores. The alkali content ranged 3.5 to 8.6kg/m³ and the average value was 5.7kg/m³. According to the measurement of Cl^- in the concrete, about 1.2kg/m³ of alkali was estimated to be supplied by the sea dredged sand.

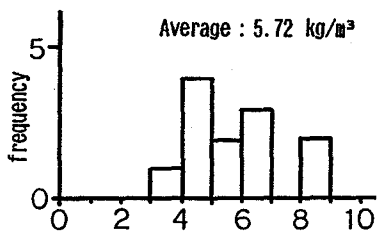

Figure 16 The amount of water soluble Na_2O eq

3. 5 State of the damaged structures

Figure 17 shows the relation between the width and depth of crack due to ASR. According to these results, there is a tendency that wider cracks are deeper. Although some cracks reached beyond the reinforcing bar, the crack depth in most of the reinforced concrete structures remained within a range of the concrete cover.

Figure 18 shows the compressive strength and Young's modulus of cores. These results indicate that loss of the compressive strength occurs in ASR concrete. These results also indicate that Young's modulus of reacted concrete is very low. However, Young's modulus back-analized from the deflection of the beam was not so low [2]. The low Young's modulus measured by the core may be due to release of restriction existed in the structure. However, the gap has not been clearly understood.

According to the investigation of reinforcing bars by the half cell potential method and direct observation, reinforcing bars in damaged concrete structures in Japan due to ASR are still in good condition.

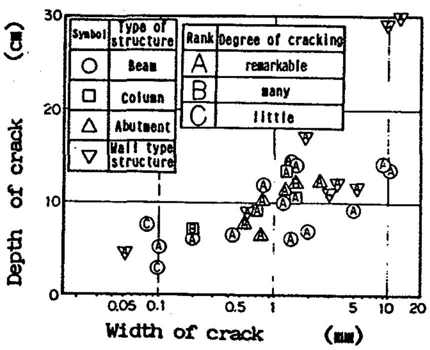

Figure 17 Relation between crack width and depth

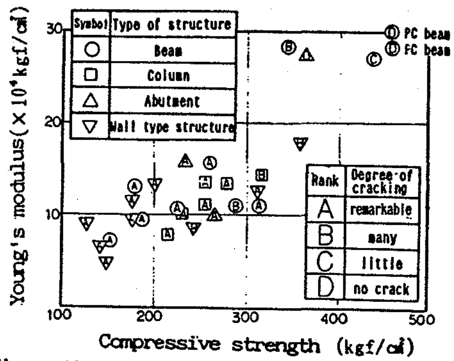

Figure 18 Compressive strength and Young's modulus of drilled cores

4. CONCLUDING REMARKS

From the investigation, following points have been found;

(1) Most of alkali-aggregate reaction occurring in Japan is alkali silica reaction.
(2) ASR in Japan is generally caused by aggregate rather than sand.
(3) The main reactive aggregate found in the investigation is Bronzite andesite, Chert and Slate.
(4) Reactive aggregate found in damaged structures was generelly crushed stone rather than river gravel.
(5) Most of the damaged structures by ASR were constructed after the late 1960.
(6) There was a tendency that wider crack was deeper. However, most of the cracks in reinforced concrete structures remained within a range of the concrete cover.
(7) Corrosion of steel in ASR concrete have not advanced yet.
(8) Compressive strength measured by drilled core indicated that it could be lowered by ASR.
(9) Young's modulus measured by drilled core was very low. However the back analized value obtained by direct loading test to the structure was not so low. The low rated Young's modulus obtained from the core is probably due to release of restriction existed in the structure. The gap of Young's modulus depending on the evaluation method should be taken into account when assessment of a structure damaged by ASR is done by the loss of rigidity.

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on Alkali–Aggregate Reaction

INVESTIGATIONS ON THE MOXOTŌ POWERHOUSE CONCRETE AFFECTED BY ALKALI–SILICA REACTION

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1. INTRODUCTION

The Moxoto powerhouse consists of four 30.5 m wide concrete bays, each one housing a 110 MW turbine-generator group. It was constructed in the period from 1972 to 1977. Both the coarse and fine aggregates contain variable amounts of strained quartz, which caused the alkali-silica reaction development. [1]

A set of experiments was programmed to study the aggregates used in the construction as well as concrete cores taken from the structure, in order to assess the expansion potential of the reaction and the influence of moisture and temperature. The use of carbon dioxide injection was also tested, to mitigate the concrete expansion.

2. INVESTIGATION PROGRAM

2.1 Mortar Bar Tests

The ASTM C227 mortar bar test was performed with aggregates used in the construction, with the temperature of 37.8^{o}C and a modified temperature of 60^{o}C. The cement used had an alkali content of 1.04% equivalent Na_2O.

Table 1 represents the mortar bars expansion values at 1 and 3 years, for the eight different petrografic types of aggregates, employed at the Moxotō concrete structures.

2.2 Influence of Temperature

The temperature influence on the expansion rate is being studied by measuring the expansion of concrete Ø 120 mm x 400 mm cores, stored at 100% RH and 30^{o}C and 60^{o}C temperature. Six cores were taken from the upper part of the structures, where the reaction was at an incipient stage, and five others from the lower part of the structures, where the reaction attained an advanced phase, as can be seen in Fig. 1 and 2. The reaction stage was established by petrographic examination.

2.3 Influence of Superficial Sealing

In order to investigate the effectiveness of sealing the concrete surface on the mitigation of the expansion rate, eight Ø 120 mm x 400 mm concrete cores were drilled from both the upper and lower part of the structure. The cores were saturated in boiling water and sealed with a PVC film. Four cores were left with their bases unsealed and all cores were stored at 100% RH and 30°C temperature, and their expansion are being measured fortnightly. Fig. 3 presents the measured core expansion.

Table 1

MORTAR BAR EXPANSION OF MOXOTÓ AGGREGATES (Na_2O + 0,658 k_2O of cement = 1.04%)						
Petrographic type	QUARTZ		EXPANSION (%)			
	%	Undulatory extinction angle	37.8°C ±1.2°C		60.0°C ±2.0°C	
			1 year	3year	1 year	3year
Granite	30	20°	0.033	0.034	0.053	0.057
Granite	25-30	18°	0.034	0.034	0.050	0.055
Cataclastic Granite	20-25	24°	0.035	0.034	0.044	0.048
Biotite Granite	20-25	18°	0.034	0.034	0.045	0.051
Diorite gneiss	-	13°	0.035	0.037	0.045	0.052
Biotite granodiorite	20-25	17°	0.033	0.034	0.045	0.051
Biotite granodiorite	15	10°	0.029	0.035	0.055	0.061
Anorthosite microcline	-	-	0.028	0.030	0.038	0.040

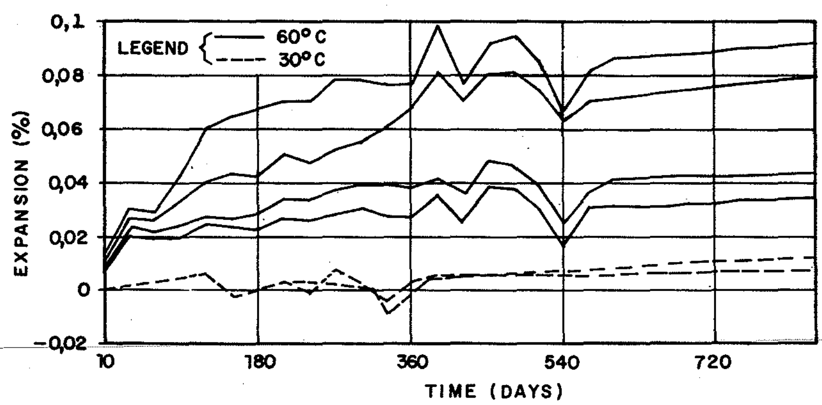

Fig. 1- Concrete Expansion with Cores Taken from the Upper Part of the Structures

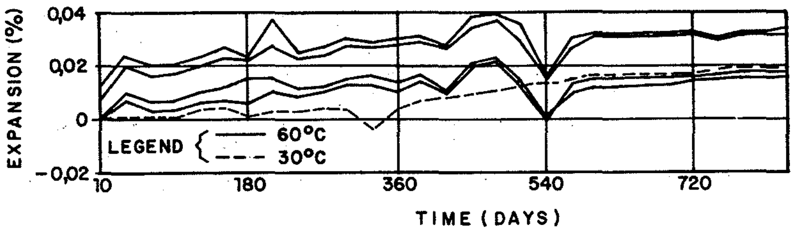

Fig.2- Concrete Expansion with Cores Taken from the Lower Part of the Structures

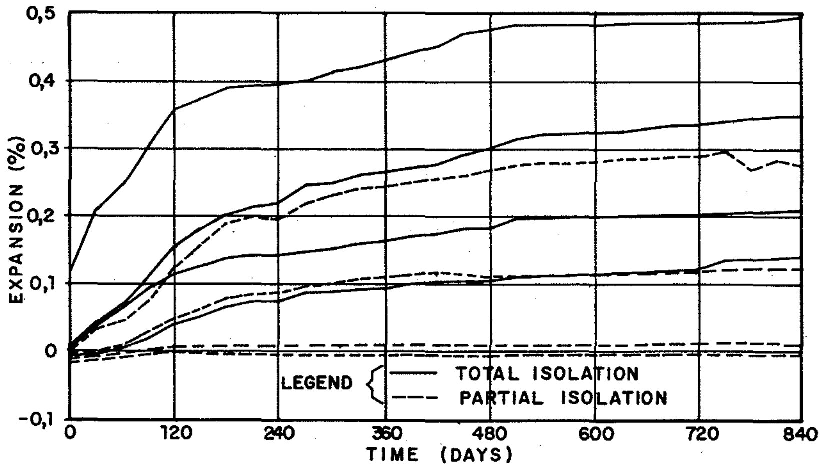

Fig.3- Concrete Expansion with Superficial Sealing

2.4 Influence of the Injection

The effectiveness of carbon dioxide in controlling the ASR expansion was investigated by measuring the expansion of a 1.0 m³ concrete block, molded with the same materials and mixture of the concrete used in the Moxotó powerhouse, but with 20% (in weight) replacement of the aggregate by pyrex glass and kept at 100% RH. The expansion was measured with 10 joint meter bases installed on the block faces. The measures started after a 30 day periode of cure. When the expansion of the concrete block was noticed, at the 207th day carbon dioxide was injected through a Ø 50 mm central hole, during 102 days, under 2.0 daN/cm² of pressure. A sample was taken by vertical drilling from the upper block surface and the carbonation extent was chemically determined. Fig. 4 presents the mean expansion measured on the block faces, while Fig. 5 presents the CO_2 and the corresponding $CaCO_3$ contents, related to the mortar and cement content, respectively.

3. MONITORING INSTRUMENTATION

A permanent monitoring instrumentation was installed in order to detect any movement between the concrete structures and their foundation, through the installation of deep multiple extensometers. It was also decided to control the differential displacements between bays, by installing triorthogonal joint meters in the contraction joints.

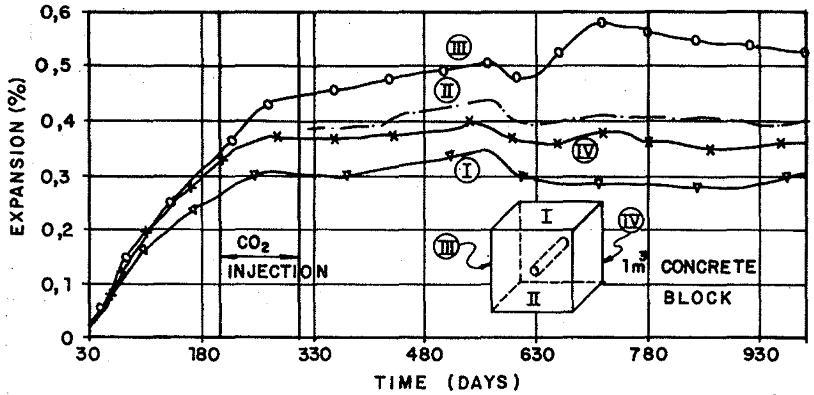

Fig.4- Effect of Carbonation on the Concrete Block Expansion

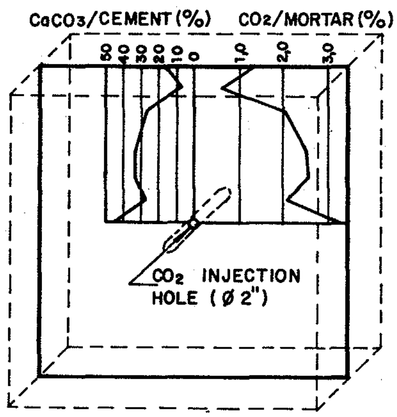

Fig.5- Results of the Carbonation Investigation Tests

Multiple extensometers are showing no movements at the foundation rock mass, while in the concrete a continuous and almost constant expansion is being measured since july 84, when they were installed. Concrete expansion measured by the multiple extensometers presents a mean strain value of 8.0 x 10^{-5} per year. The displacements measured by extensometer EM-4, installed in block nº 3, are presented in Fig. 6.

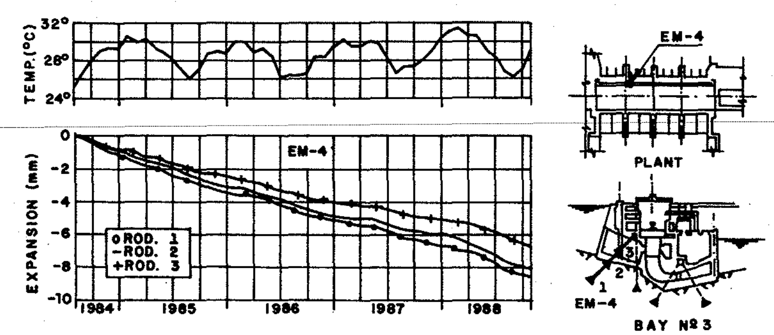

Fig.6- Concrete Expansion Measured by Extensometer EM-4

4. DISCUSSION

4.1 Mortar Bar Tests

The results of the mortar bar tests performed in three years confirm the inadequacy of the ASTM C227 in evaluating the potencial reactivity of the strained quartz, even when tested at a 60°C temperature.

The Corps of Engineering Manual (1983), which is thought to be more appropriate for strained quartz aggregates, establishes that the reactivity occurs when: the strained quartz content is more than 20% and undulatory extinction is more than 15%; or, the mortar bar tests show expansions greater then 0.025% in 6 months or 0.040% in 12 months.

Based on this criterion all aggregate samples from Moxotó dam showed potential reactivity, as can be seen in Fig. 7, where an attempt to sumarize the results of the main criteria concerning the reactivity of the strained quartz was tried.

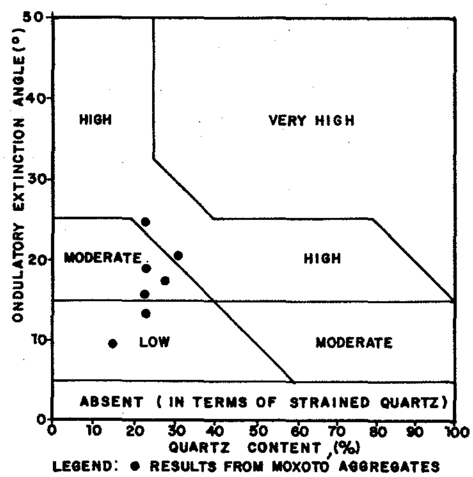

Fig.7- Reactivity of Aggregates with Strained Quartz Based on Usual Criteria

4.2 Influence of Temperature

Considering the concrete cores taken from the same region of the structures the expansion at 60°C was five times greater than that at 30°C.

4.3 Influence of Superficial Isolation

Unfortunately all tests with total isolation were made with cores taken from the upper part of the structures, which explain the higher expansion when compared with those with partial isolation, taken from the lower and wetter part of the structures (that means with concrete at an advanced reaction stage). However these tests were important to show that even after 2.5 years, the concrete expansion goes on.

It is important to emphasize that the high expansion rates showed by these tests were a consequence of the saturation process, carried out with boiled water at 100°C.

4.4 Influence of Carbon Dioxide Injection

During the CO_2 injection operation, which took about 4 month, the concrete expansion measured at the 1.0 m³ block faces showed a significant decrease, and later, except one face, the concrete expansion practically ceased. The authors intend to perform some more tests to confirm and complement this, but it seems that the CO_2 injection can be, in some circunstances, a good way to decrease or even cease the concrete expansion caused by alkali-aggregate reaction, even in the presence of the strained quartz.

5. ACKNOWLEDGEMENTS

The authors wish to express their thanks to CHESF - Companhia Hidro Elétrica do São Francisco, for allowing the publication of the data and information on the Apolônio Sales Powerplant (Moxotó) and to PROMON ENGENHARIA S.A., for the support and cooperation offered during the preparation of this paper.

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THE BACKGROUND OF AAR PREVENTIVE MEASURES ADOPTED BY THE JAPANESE MINISTRY OF CONSTRUCTION

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1. INTRODUCTION

Since around 1982 in Japan, many concrete structures in which alkali aggregate reaction (referred to as AAR) has been taking place have been found, and the Ministry of Construction issued a notification for AAR preventive measures in 1986[1]. It requests that the durable concrete structure has to be taken one of the following four methods.

1) To use only inocheous aggregate.

2) To use low-alkali cement.

3) To use pozzolan blended cement.

4) To limit the total amount of alkali (Na_2Oeq) in concrete to 3 kg/m³ or less.

This paper describes briefly the experimental background for the proposed methods of 3) and 4).

2. PREVENTION BY GROUND GRANULATED BLASTFURNACE SLAG [2]

It is well known that ground granulated blastfurnace slag (referred to as ggbs) has a preventive effect against AAR reaction. However, no established view has been proposed about whether all kinds of ggbs in Japan have this preventive ability and at what mixture ratio ggbs is suitable for preventing AAR.

Therefore, we conducted a study to examine the AAR preventive ability of all kinds of Japanese ggbs (12 brands) by ASTM C441[3]. The results of the tests are shown in Figure 1. It indicates that the AAR preventive ability is uncertain for almost all of the tested ggbs, unless the slag content is 65% or more. However, we could not understand clearly the reason why the experimental results obtained with very sensitive and reactive pyrex glass have to be adopted.

Then we performed mortar bar tests using deleterious andesite aggregate, which is produced in Japan in large quantities, using different slag substitution ratios, slag brands, and amounts of alkali in base portland cement. The following results were obtained:

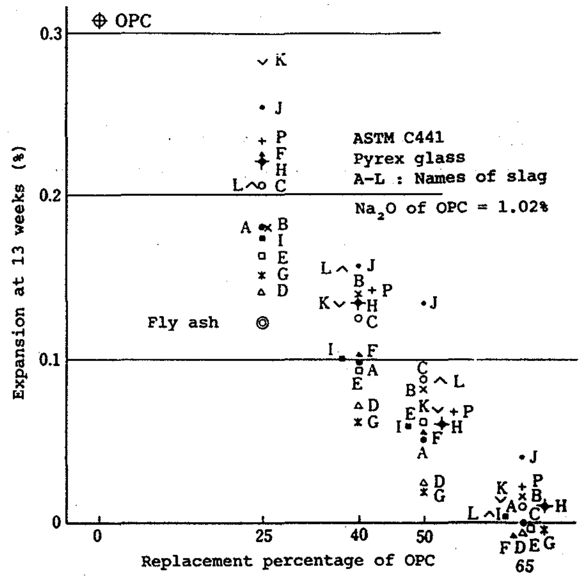

Figure 1 ASR Preventing Effects of ggbs [2]

1) When the base portland cement is replaced with 50% of ggbs, the expansion of the mortar bar is below 0.1%, even when the alkali content of the cement is approximately 2% and J-brand ggbs, which has the least preventive ability, is used.

2) When the alkali content in the cement is 1.2%, the expansion of the mortar bar is below 0.1%, even when the replacement ratio is 30% and the J-brand is used. Figure 2 shows a part of the experimental result.

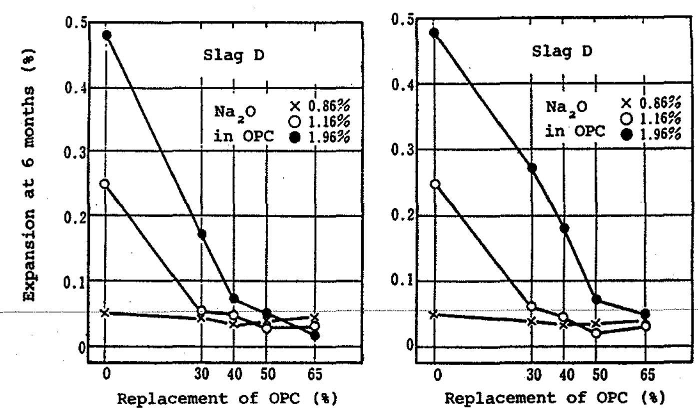

Figure 2 Examples of Percentage Replacement and Expansion of Mortar Bars with the Andesite Aggregate [2]

Based on these results, we concluded that AAR can be prevented sufficiently if blastfurnace slag cement containing more than 50% ggbs is used and if the alkali content in base portland cement is less than 0.8%, the required minimum ggbs content is decreased to 40%.

3. PREVENTION BY FLY-ASH [4],[5]

Some published papers say that pulverized fuel ash (referred to as pfa) has the ability to prevent AAR, while other papers doubt pfa's preventive ability. This is because some brands of pfa have a very high alkali content and some reports indicate that AAR took place even though pfa was used.

We conducted tests for mortar bars made from deleterious andesite aggregate produced in Japan and all types of pfa produced in the country. Figure 3 shows amounts of expansion after 6 months when the replacement ratio of pfa was 10, 20, or 30% with an alkali content of 1.2% in the cement.

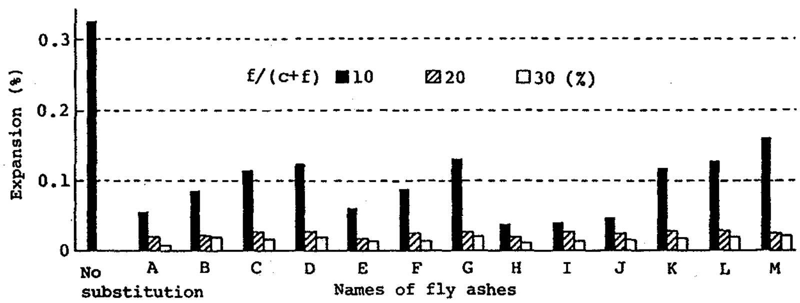

Fig. 2 f/(c+f) and Expansion of Fly Ashes (R_2O = 1.2%)

Figure 3 f/(c+f) and Expansion of Fly Ashes (R_2O = 1.2%) [4]

The Figure indicates that when mixed at 10% some brands of pfa have a higher expansion preventive ability. All types of pfa, when mixed at more than 20%, showed satisfactory preventive ability. We conducted a factor analysis for pfa components and the amount of expansion and found that Na^+ in pfa is a primary factor in mortar bar expansion, while SiO_2 is a factor in preventing expansion.

Based experimental results, we concluded that fly-ash cement can prevent the expansion caused by AAR, if the equation below is met:

$$\Sigma CA + 0.83\Sigma FA - 0.046\Sigma F \leqq 4.2 \ (kg/m^3) \qquad (1)$$

where ΣCA, ΣFA: total alkali content in cement, in pfa, by Na_2Oeq
ΣF: total amount of pfa

Taking account of the fact that the maximum alkali content in pfa and portland cement are 4% and 1.2% at the most, we concluded that if fly-ash cement with a pfa content of more than 20% is used AAR can be prevented, and if the alkali content in the cement is less than 0.8%, the requested minimum pfa content is decreased to 15%.

4. PREVENTION BY LIMITING TOTAL ALKALI AMOUNT IN CONCRETE [6]

AAR prevention by controlling the total amount of alkali in concrete is a method which was proposed and has been used in the United Kingdom.

We conducted mortar bar and concrete prism tests in order to know whether there is any amount of alkali in which expansion does not take place even when a reactive aggregate is used.

Figure 4 shows the results of mortar bar tests in which the total amount of alkali varies.

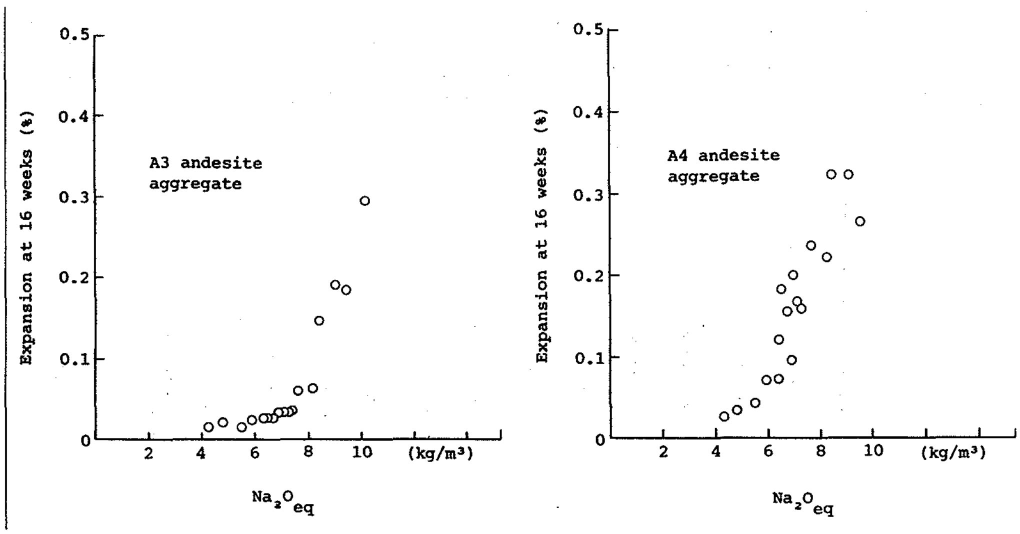

Figure 4 Expansion of Mortar Bar and Amount of Na_2Oeq [6]

Figure 5 shows the results of concrete prism tests. The specimens were made at different ratios of inactive aggregate to harmful aggregate in consideration of the affection of the pessimum, using only coarse aggregate as a reactive aggregate, and then cured at 40°C in moist conditions for 6 months.

In all the tests, expansion did not take place when the total amount of alkali in concrete or mortar was less than 3 kg/m³. Therefore, the authors concluded that AAR can be prevented when the total amount of alkali (Na_2Oeq) in concrete (including soluble Na^+ in the admixture and aggregate) is below 3 kg/m³. According to this theory, AAR takes place if the total amount of Na_2Oeq has increased even when low alkali cement is adopted. This was also proved in other experiments.

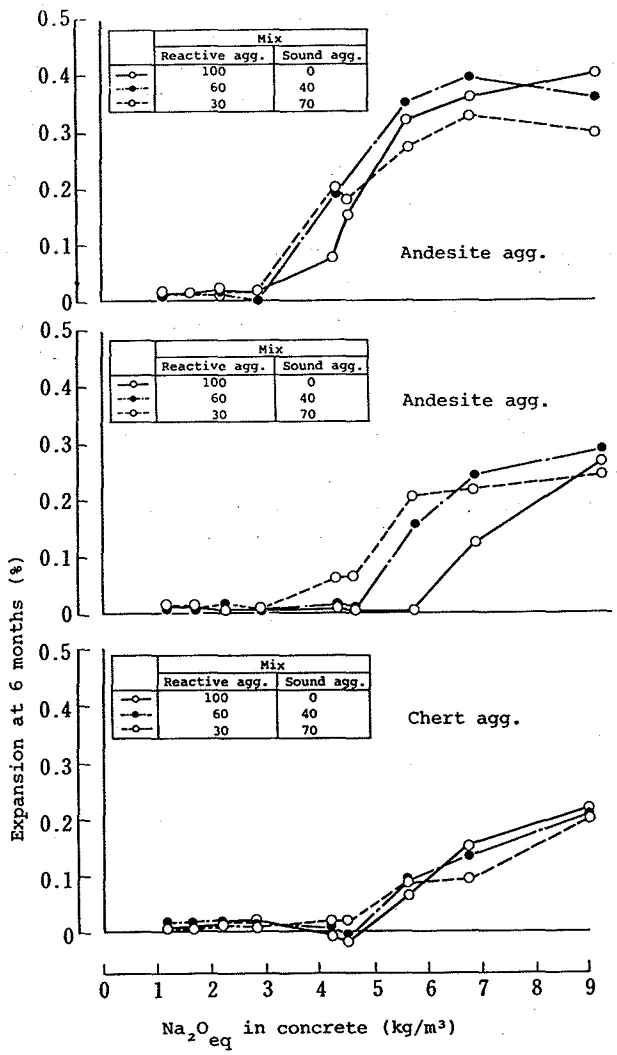

Figure 5 Expansion of Concrete Prism and Amount of Na_2Oeq in Concrete [6]

5. ACKNOWLEDGEMENT

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8th Internationl Conference on Alkali-Aggregate Reaction

ALKALI-AGGREGATE REACTIVITY — IS IT ALWAYS HARMFUL?

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Abstract

In an extensive survey of the present state of aging of concrete in 8 dams built between 1910 and 1960 with a wide range of aggregates, evidence of alkali-aggregate reactions were observed in almost all the core specimens tested. Alkali-aggregate reactions in these dams manifect themselves in a number of ways: reaction rim around aggregates, discoloration of aggregates in the peripheral region, polygonization of grains, loss of cohesion between paste and aggregate, inter and intragranular fissures, and delamination of stratified rocks. Compressive and splitting strength, and permeability measurements show that in most cases these alkali-aggregate reactions did not cause any significant loss in the engineering properties of concrete. It may be stated that unless the aggregate deterioration is well advanced, mere evidence of alkali-aggregate reactivity itself need not cause alarm. It is possible that due to lack of favorable environment and limited amount of reactive materials available, the reactivity tends to stabilize at an innocuous level.

1. INTRODUCTION

Hydro Québec owns several hydroelectric installations comprising over one hundred concrete structures of different ages made with different concrete mixes. A large variety of aggregates have been used for these concretes. Thirty six of these aged concrete structures were examined in an effort to learn more about the aging characteristics of concrete in hydraulic structures built in a very harsh environment, and in different administrative regions. Eight of these concrete structures (listed in Table 1) were investigated in much greater details.

The results discussed in this paper were obtained from representative core samples, extracted from deep within the structure, and from the surface of the structure (less than 3 m deep).

Practical observations and monitoring during the last decade have shown that in most cases the development of alkali aggregate reaction within the concrete of a hydraulic structure may not necessarily require any immediate corrective measures. Repairs as and when required, are generally carried out as a part of punctual action for the purpose of restoration of some specific operating part (deblocking of a gate, friction within a turbine, etc.), rarely to restore the short stability of the structure. Therefore, it is time to question whether alkali aggregate reaction is really always so harmful?

2. DESCRIPTION OF THE STUDIED STRUCTURES

The principal characteristics of these structures are presented in Table 1, where the type of coarse and fine aggregate used are described in terms of mineralogy, petrography and the maximum diameter. This table also indicates the external appearance of the concrete.

From Table 1 it is evident the aggregates used in the concrete of these structures represent a wide variety, which originate from igneous rocks like diorite, granite, monzonite, sedimentary and metamorphic rocks like marble metagreywacke, dolomitic limestone, sandstone, gneiss, etc. On some occasions these were found to be of detritic origin, and composed of wide range of minerals, mainly quartz, feldspar, biotite, muscovite ferromagnesians, (amphibole, pyroxene), while in other cases these were the processed aggregates from a rock quarry, and therefore, were more homogeneous from a petrographical point of view. The maximum size of the coarse aggregate varied between 25 and 100 mm.

The fine aggregates mostly were of detritic origin, except in two cases; in Beauharnois concrete it was a manufactured sand composed of the finer part of the crushed sandstone, and in the case of Grand-Mère a small portion of it was crushed diorite.

All the cores extracted from these hydraulic structures were tested for compressive and splitting strength, as well as for permeability measurement. In some cases in-situ permeability measurements were made in an attempt to correlate the laboratory measurements with some field values. These results are presented in Table 2. It is clear that the concrete compressive strength is spread over a wide range, not only from one dam to the other, the lowest value measured being 22 MPa for Shawinigan dam while the highest is 59 MPa for Mercier dam, but also within the same structure, eg., the compressive strength varied from 23 MPa to 51 MPa in La Gabelle dam. The same spread for the splitting strengths was also noted. It can be stated that the most affected concrete (by an AAR), the Beauharnois concrete, does not show drastic strength loss, but a low elastic modulus was measured in this case (18 GPa).

Water permeability values measured radially in 150 × 300 mm hollow cores (37.5 mm axial hole) shows that the permeability of the concrete also covers a wide range, from 2×10^{-6} m/s to 5×10^{-13} m/s.

The most extraordinary feature detected when examining the microstructure of these concretes under a transmitted light optical microscope was the presence of some form of AAR. in almost all the cases. Table 3 presents a synthesis of these observations that are discussed in more details in the following section.

3. MICROSTRUCTURAL OBSERVATIONS

Optical microscopy still forms one of the principal tools for studying degradation, such as alkali-aggregate reactions, in concrete. Bulk chemical analysis of the affected region no doubt provides an insight into the possible chemical alterations which may have taken place in the course of these reactions, but the morphology, the present state and the chemical composition of resultant products, eg., the reaction rim itself or the deposits within the expansion cracks can only be determined by SEM/EDX analysis. Hence, these two techniques were principally used to characterize the effects of alkali-aggregate reactions in these concretes.

TABLE 1. Summary of construction data of dams studied

	NAME	AGGREGATE TYPE	PRESENT CONDITION OF THE DAM: VISUAL OBSERVATIONS
LAURENTIDE	FARMERS	COARSE: diorite, granite, monzonite gneiss FINE: detritic sand - 65% quartz, 10% feldspar, ferromagnesians 5%	The exposed surface concrete is in bad condition, but in excellent condition inside. AAR is noticeable. Ø max 2 in.
	MERCIER	COARSE: marble 45%, metagreywacke 50%, gravel 5% FINE: quartz 70%, K - feldspar 25%, ferromagnesians 3 to 4%	Excellent. Non air entrained concrete (1.5%). Only localized signs of AAR. Ø max 4 in.
	RAWDON	COARSE: granitic pebbles with some quartzite, anorthosite, clayey dolomitic limestone. FINE: detritic sand - quartz, feldspar	In some areas the concrete is of excellent quality, in others quite poor. Localized signs of AAR. Ø max 1 in.
MAISONNEUVE	BEAUHARNOIS	COARSE: Postdam sandstone. FINE: Postdam sandstone.	The whole structure subjected to severe AAR. The exposed concrete appears to be more damaged. Ø max 2 in.
	LES CÈDRES	COARSE: 80% dolomitic limestone, 16% quartzite FINE: detritic sand - quartz, feldspar.	Concrete in good condition, but signs of AAR are visible around the quartzite aggregates. Ø max 2 in.
MAURICIE	GRAND'MÈRE	COARSE: quartzitic diorite 75%, granite 10%, quartzite 15% FINE: quartzitic diorite and detritic sand	In fairly good condition. No significant exterior signs of AAR. Ø max 4 in.
	LA GABELLE	COARSE: sandstone containing some mafic inclusins. FINE: siliceous sand.	Most of the exposed concrete is in bad condition. The concrete inside the dam is much better. Ø max 1 in.
	SHAWINIGAN	COARSE: gneiss FINE: granitic sand	Parts of the exposed concrete is in good condition, whereas others have deteriorated, depending on the location. Local exudation of silica gel. Ø max 2 in.

TABLE 2. Summary of engineering properties of concrete of each dam

	Compressive strength (MPa)	Splitting strength (MPa)	Permeability	
			Laboratory (m/s)	In-situ L/min/m
FARMERS	Not recovered	Too weak	N.M.	7 to 23
MERCIER	42.4 to 59.2	1.5 to 4.0	4.5×10^{-8}	N.M.
RAWDON	49.3 to 54.2 24.5 to 34.7	3.1 0.4 to 1	1×10^{-8}	0.3 to 0.8
BEAUHARNOIS	40 to 45	1.6 to 3.4	15×10^{-12}	N.M.
LES CÈDRES	36 to 67	N.M.	2×10^{-6} 0.2×10^{-8}	N.M.
GRAND'MÈRE	24 to 43	1.6 to 3.7	1×10^{-9} to 5×10^{-10}	N.M.
LA GABELLE	23 to 28 47 to 51	N.M.	8×10^{-8} 5×10^{-13}	0.2 to 24
SHAWINIGAN	22 to 45	4 to 5	10^{-10}	N.M.

TABLE 3. Summary of the observations of alkali aggregate reaction in each dam

	Manifestation of alkali aggregate reactions
FARMERS	Exudation of some silica gel. Severe internal cracking in the laminated rocks (Fig. 1). The other aggregates are intact.
MERCIER	Localized reaction rims around some coarse granitic aggregates (Fig. 2). Some quartz grains are heavily microcracked (Fig. 3). No detrimental effect on the concrete.
RAWDON	Some reaction rims around some coarse granitic and quartzite aggregates. Some quartz grains show polygonization (Fig. 4).
BEAUHARNOIS	Reaction rims around coarse aggregates and severe microcracking inside. Boundary discoloration visible (Fig. 5).
LES CÈDRES	Severe microcracking around quartzite aggregates (Fig. 6). Secondary cracks are filled with some silica gel. Dolomitic aggregates are unaffected by AAR.
GRAND'MÈRE	Only very localized AAR observable with an optical microscope. No detrimental effect on the concrete.
LA GABELLE	Quartz with undulatory extinction. Reaction rims can be observed at microscopic level. No detrimental effect on the concrete.
SHAWINIGAN	Two types of localized reactions. Alkali silica and alkali silicate reaction rims around some aggregates with no harmful effect on the concrete.

It is well known that a large number of these rocks used in the construction of these dams are reactive [1-4]. Hence it is needless to state the concretes examined do exhibit alkali-aggregate reactions. They manifest themselves in a number of ways, which are illustrated in a series of optical micrographs (Figs. 1-6).

Under the SEM, ettringite crystals in the reaction zones were found to be very common, and at times they appear to have carbonated. Both alkaline and non-alkaline silicate and aluminate gels were also observed. Rosette structure corresponding to calcium alumino-ferrite in composition have formed in places. The reaction rim and decohesion between aggregate and paste is also evident. Similar products in concrete structures in Quebec have been observed by other workers [5,6].

4. CONCLUSIONS

When this investigation started it was already known that two of the eight studied dams were severely affected by AAR, and in one case minor repairs had already been carried out. In the six remaining dams the concrete was still in good condition although through a detailed microscopic examination evidence of localized AAR was detected. The type and extent of reaction dictates whether immediate corrective measures are necessary, or the nature and frequency of follow up that has to be undertaken. Characterization of engineering properties of these concrete show that even if localized AAR is prevelant in hydroelectric concrete structures, it is not necessarily always harmful.

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Fig. 1: Delamination in a stratified rock

Fig. 2: A thick reaction rim around the aggregate

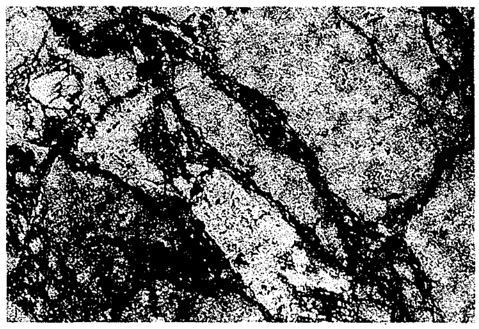

Fig. 3: Inter and intragranular fissures in quartz grains

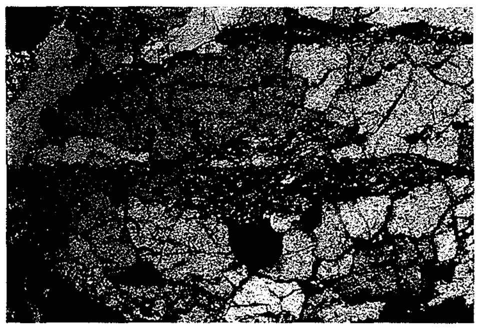

Fig. 4: Polygonization of a quartz aggregate

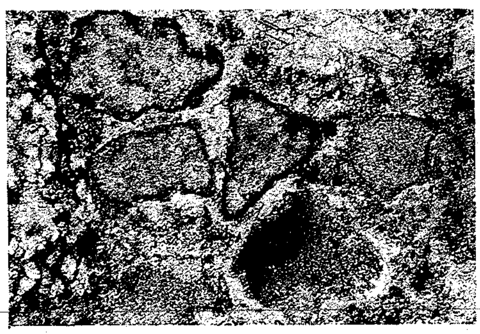

Fig. 5: Discoloration along the periphery of aggregates

Fig. 6: Boundary cracks in a quartz aggregate

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STUDY ON THE EFFECT OF CONCRETE SURFACE COATING FOR PREVENTION OF ALKALI SILICA REACTION

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1. ABSTRACT

Cracking and deterioration of concrete structures caused by the alkali silica reaction are now becoming a matter of social concern, with studies undergoing on the reaction mechanism for this deterioration and repairing measures of deteriorated buildings as well as prevention of such deterioration.

However, no effective acceleration method which can cause cracking by this reaction in reasonably shorter period of time and is not excessively severe to the coatings and yet which can simulate the actual ambient conditions as close as possible has been established.

This report is intended to describe a testing equipment which can simulate actual ambient conditions and effective for acceleration of concrete cracking and the results of studies carried out at the Public Works Research Institute, Ministry of Construction, as the guest researchers on effectiveness of concrete surface coating with this equipment.

2. OUTLINE OF EXPERIMENT

2.1 Concrete Test Pieces

In total, 7 types of concrete samples were prepared using two types of crushed stones which show alkali silica reaction (T.F.) and two types of crushed stones which do not show this reaction (K.C.). Table 1 shows analysis of crushed stones while Table 2 and Table 3 respectively show standard mixing of concrete and types of mixing. The dimension of concrete test pieces was 10 x 10 cm.

Table 1 Results of Chemical Analysis

	Rc (mmol/ℓ)	Sc (mmol/ℓ)
Crushed stone from T	183.0	639.0
Crushed stone from K	34.3	39.6
Crushed stone from F	110.0	236.6
Crushed stone from C	21.1	17.9

Rc: Reduction in alkali concentration
Sc: Silica elusion amount

Table 2 Concrete Standard Mixing

Maximum size of coarse aggregate (mm)	Slump (cm)	Air (%)	A/C (%)	s/a (%)	Unit quantity (kg/cm³)			
					Water (W)	Cement (C)	Coarse aggregate (G)	Fine aggregate (S)
20	8	5	50	40	195	390	905	924

Normally, Portland cement is used.

Table 3 Mixing of Concrete

Concrete No.	Coarse aggregate	Sand	Alkali
A	F/C=80/20	Harmless standard river sand	NaOH
B	F/C=80/20	ditto.	—
C	K	ditto.	—
D	T	ditto.	NaCl
E	T	ditto.	—
F	T/C=50/50	ditto.	NaOH
G	T/C=50/50	ditto.	—

Cement alkali, (NaOH, NaCl), were adjusted to $8kg/cm^3$ on R_2O basis.

2.2 Surface Coating Materials

As the surface coating materials, the following were selected.

(1) Epoxy system materials of high elasticity and high cut-off,

(2) Rubber system materials of high elasticity

These materials which have high cut-off capacity for moisture and harmful ions, with coefficient of extension higher than 200%, can withstand dynamic changes due to generation and growth of fine cracks of concrete, etc.

(3) Silane system penetrants

These penetrants are reasonably priced penetrants and do not form films on concrete surface.

They have features in that they penetrate deep into concrete structures and form water repelling layers by combining with concrete.

(4) Acrylic emersion system materials

Being acqueous emersions, these are low pollutive materials. They are also less expensive and have high elasticity.

Table 4 shows coating specifications of each sample concrete piece.

Coating was made on test pieces with water content of approximately 5%.

Coating had been made after surfaces had been conditioned (CC 50).

After coating all surfaces in the room kept at 20°C, they were dried for ten days for test use.

Table 4 Coating Specification

No.	Type of coating material system	Processing	Film thickness (μ)
①	Silane system	Silane system penerants Silane system penerants	— —
②	High elasticity epoxy system	Epoxy primer Epoxy putty High elasticity epoxy type intermediate coat Elastic polyurethane	— — 170 30
③	High elasticity rubber system	Epoxy primer Epoxy putty High elasticity epoxy type intermediate coat Elastic polyurethane	— — 200 30
④	Acrylic emulsion system	Polymer cement Acrylic emulsion top coat Acrylic emulsion top coat	(2mm) 30 30

2.3 Development of Accelerated Test Equipment

An equipment to satisfy the following requirements was developed:

(1) to be capable of supplying water (or water containing harmful ion)

(2) capable of heating concrete test pieces by radiant heat containing far infra-red ray,

(3) capable to dry with hot air of low humidity, and

(4) capable of repeating drying and wetting by (1) and (2) + (3) above.

These conditions were determined assuming that acceleration of reaction by radiant heat after supplying sufficient water (or water containing harmful ion) into concrete and subsequent rapid evaporation of moisture by hot air of low moisture content from concrete surface should cause moisture migration and concentration of alkali within concrete test pieces and that repeating of these procedures would cause repeated supplies of moisture and alkali around the reactive crushed stones thereby accelerating generation of gels and expansion.

Also, these conditions simulate actual ambient conditions in which actual concrete structures deteriorate by this reaction.

2.4 Test Method

All types of concrete test pieces without surface coating were tested. For surface coating effect test, test pieces of mixing Nos. A, B and D were used. Test was repeated twice or more.

Tests were performed using the following three methods.

2.4.1 Test method using accelerated test equipment In this method, the test equipment as described in 2.4 above was employed. Conditions for repeated tests were water spraying (by tap water) 0.5 hour, heating+drying=2.5 hours, total one cycle 3 hours. Room temperature and water temperature at drying were set at 40°C and 35°C respectively.

2.4.2 Outdoor exposure test by water spraying A test one cycle of which consisted of 0.5 hour of water spraying and 2.5 hours of drying during day time (7:00 to 19:00) was repeated, in other words, 4 cycles a day, which simulates the actual ambient conditions.

2.4.3 High temperature high humidity aging test Test pieces were left in a room kept at 40°C and RH of 100% as the standard test pieces for crackings generated on concrete.

3. RESULTS AND OBSERVATION

3.1 Results on Test Pieces Without Surface Coating

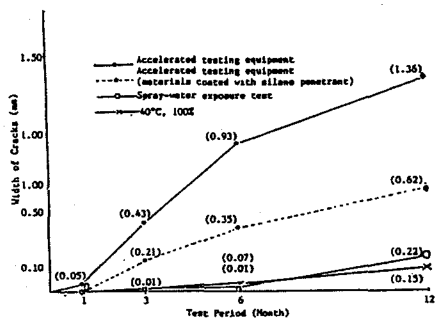
Table 5 shows results after 12-month test period by each of three methods as described in 2.4.1 through 2.4.3 above. As is clear from this Table, on all concretes with increased alkali contents and using harmful crushed stones (mixing Nos. A, D and F), cracking was observed on all three testing methods.

Table 5 Cracking of Uncoated Test Pieces by Different Testing Methods

Type of concrete	Test period / Test items / Type of test	1 month		3 months		6 months		12 months	
		No. of cracks	Max. width of cracks (Av. mm)	No. of cracks	Max. width of cracks (Av. mm)	No. of cracks	Max. width of cracks (Av. mm)	No. of cracks	Max. width of cracks (Av. mm)
A	①	3/3	0.05	3/3	0.43	3/3	0.93	3/3	1.36
	②	0/30	—	7/30	0.01>	24/30	0.01	30/30	0.22
	③	0/3	—	2/3	0.01	3/3	0.07	3/3	0.15
B	①	1/3	0.01>	3/3	0.01	3/3	0.01	3/3	0.02
	②	0/6	—	0/6	—	0/6	—	0/6	—
	③	0/3	—	0/3	—	0/3	—	0/3	—
C	①	0/3	—	0/3	—	0/3	—	0/3	—
	②	0/3	—	0/3	—	0/3	—	0/3	—
	③	0/2	—	0/2	—	0/2	—	0/2	—
D	①	0/3	0.05	3/3	0.30	3/3	0.65	3/3	1.08
	②	0/3	—	2/3	0.01>	2/3	0.01	3/3	0.32
	③	0/3	—	0/3	—	2/3	0.01	2/3	0.08
E	①	3/3	0.01>	3/3	0.01	3/3	0.01	3/3	0.02
	②	0/3	—	0/3	—	0/3	—	0/3	—
	③	0/3	—	0/3	—	0/3	—	0/3	—
F	①	3/3	0.10	3/3	0.37	3/3	0.63	3/3	1.13
	②	0/6	—	6/6	0.07	6/6	0.10	6/6	0.41
	③	0/3	—	3/3	0.10	3/3	0.15	3/3	0.20
G	①	2/3	0.01>	2/3	0.01	2/3	0.01	3/3	0.01
	②	0/5	—	0/5	—	0/5	—	0/5	—
	③	0/3	—	0/3	—	0/3	—	0/3	—

Testing Method: 1 Accelerated testing equipment
2 Spray-water exposure test
3 Curing at 40°C, 100% (RH)

As for test pieces without increased alkali but in which harmful crushed stones were used, crackings were generated only on samples tested on our accelerated testing equipment. In Fig. 1, 2 and 3, growth of cracks by each test method is shown. These figures indicate that growth of crackings on samples tested on the accelerated testing equipment is very rapid. No generation of cracks was observed on all test pieces in which harmless crushed stones were used. From these observations, it is considered that cracks generate due to alkali silica reaction.



Accelerated testing equipment
Accelerated testing equipment (materials coated with silane penetrant)
Spray-water exposure test
40°C, 100%

Fig. 1 Growth of Concrete Cracks (No. A)

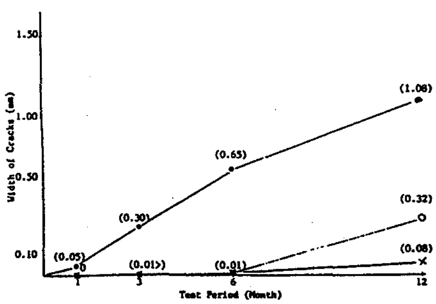


Fig. 2 Growth of Concrete Cracks (No. D)

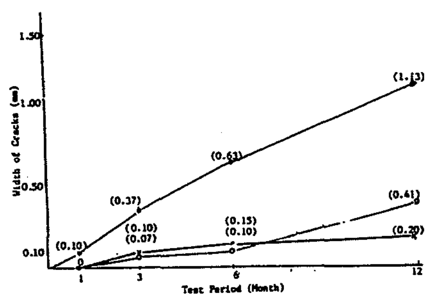


Fig. 3 Growth of Concrete Cracks (No. F)

Studies on effect of increased alkali contents failed to reveal clear indication on difference of influence by NaCl and NaOH as types of crushed stones used were not same.

However, it can be said that NaCl has more significant influence.

Thus, it would be necessary for actual structures to pay careful attention as in case of damage by salt.

3.2 Tests on Effect of Surface Coating

3.2.1 Epoxy system of high elasticity and high cut-off quality On concrete test pieces coated with these coating materials, no cracking was observed after 12-month test (The coating films also remaining in good conditions).

3.2.2 High elasticity rubber system materials Test pieces tested on the accelerated test equipment began to cause cracks under films, after 10-month period. The maximum width of craks was 0.2 mm. Though cracks could be observed through coating films, no defect was observed on film itself. On all other test methods, no cracking nor other change was observed after 12-month test.

3.2.3 Silane system penetrants On test pieces tested on the accelerated test equipment, after three-month tests, cracking was observed on Mixing No. A only. Fig. 2 shows growth of cracks and Photo. 2 indicates status of the sample piece/cracks 6-month after start of the test. On all other test methods, no cracking nor other change was observed after 12-month test.

3.2.4 Acrylic emersion system coating materials On test concrete pieces coated with these coating materials and tested on the accelerated test equipment, blisterings were observed on films with cracks generating as well. Studies of test pieces by removing films revealed cracks on concrete test pieces. The maximum width of cracks was 0.1 mm.

Again, on all other test methods, no cracking nor other change was observed after 12-month test.

4. CONCLUSION

The above clearly proved that the accelerated test equipment developed by us is an effective instrument to accelerate concrete cracking due to alkali silica reaction. It was further confirmed that application of proper coatings on concrete surfaces, cutting off infiltration of external moisture, etc., can prevent or delay cracking of concrete due to this reaction.

It is our film belief that this equipment can be applied to verification test of cracking of concrete which uses crushed stones determined harmful by chemical analysis, verification test using cores taken from actual concrete structures as well as for salt-resistance tests using brine.

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EVALUATION OF SURFACE TREATMENT EFFECT FOR PREVENTING EXCESSIVE EXPANSION DUE TO ALKALI-SILICA REACTION

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1. INTRODUCTION

Durability of concrete is effected by alkali-silica reaction(ASR) which causes excessive expansion,cracking and occasionaly significant deterioration of concrete. Many cases of concrete damage due to ASR have been obserbed in real structures subjected to constant wetting such as dams,bridges,retaining wall,and foundations(1). Since 1982, a number of instances of severe damage of concrete structures resulting from ASR have come to light in Japan,and ASR has become a subject of greate concern(2).

Concrete surface treatment systems to control the moisture in concrete can be a useful way of reducing excessive expansion due to ASR(3) while the effectiveness of the systems seem to vary widely depending upon coating materials,exposure conditions, applying time of coating, and moisture content in concrete.

The object of this paper is to evaluate the effectiveness of concrete surface treatment with coating materials under various conditions for controling moisture which may contribute to ASR in concrete.

2. OUTLINES OF TESTS

This investigation is divided into three test series as follows:

1st test series: Influence of the coating materials shown in Table 1 on the reduction of expansion due to ASR under an accelerating condition (40℃,>95%R.H.).

2nd test series: Influence of applying time of coating for deteriorated concrete specimen and the concrete type(plain concrete,reinforced concrete) on the reduction of expansion due to ASR under the outdoor condition.

3rd test series: Influence of the moisture content of concrete specimens on surface treatment effects in reducing expansion due to ASR under an accelerating condition (40℃,>95%R.H.).

Table 1 Coating Materials

No.	1 st	2 nd	3rd	Thickness
1	Modified Silane *	——	——	——
2	Alkyl Alkoxyl Silane*	Alkyl Alkoxyl Silane*	——	——
3	Acryl P-C #	——	——	800~1000
4	Acryl Primer	Acryl P-C #	Acryl P-C #	700~ 800
5	Acryl Compound*	Acryl Compound*	——	——
6	Silicon Compound*	——	——	——
7	Silicon Primer	Silicon Coating	Silicon Coating	350
8	Epoxy Primer	Epoxy Coating	Epoxy Coating	700
9	Polyester Primer	Polyester Coating	Polyester Coating	70
10	Urethane Primer	Urethane Coating	Urethane Coating	350

cf 1) * Surface Modifier　　　　　　　　　　　　unit : μm
　　# Polymer-Cement Composite
　2) Epoxy Coating contains glass flake filler.

2.1 Applying Time of Concrete Surface Treatment

In the 1st test series, surface treatments was carried out on the specimens which had been stored in the room of 20℃ and 60%R.H.for 4 weeks.In the 2nd test series, the treatment was carried out on the specimens of plain concrete at the time when the expansion due to ASR reached 0%,0.03%, 0.10%,respectively while the specimens of reinforced concrete were surface-treated at the time when the expansion due to ASR reached 0%,0.05%,0.08%,respectively. In the 3rd test series, the treatment carried out on the specimens when their moisture content decreased down to 4.5%,1.5%,0%,respectively,in a drying chamber after storing in a room of 20℃ and 60%R.H. for 4 weeks. For comparision, specimens of which moisture content were 6.0% as of no accelerating drying were also prepared.

2.2 Materials and Concrete Specimens

Ordinary portland cement, non-reactive crushed sand and crushed coase aggregate mixed with reactive and non-reactive aggregate were used. Concrete specimens were adjusted to have an equivalent alkali content of 7.0Kg/m^3 with an addition of NaOH and KOH. The potential reactivity of used aggregates is shown in Table 2.The mix proportion is shown in Table 3.

Plain concrete specimens(10x10x40cm prism) were prepared in all test series. In addition, reinforced concrete specimens(10x10x40cm prism,ratio of reinforcement=1.57%) were prepared in the 2nd test series.

Table 2 Potential reactivity of aggregates

Aggregate	Type of rock	Chemical method			Specific gravity	Absorption (%)
		Sc (mmol/l)	Rc (mmol/l)	Division		
Crush stone A	Andesite	613	262	Potentially deleterious	2.54	2.40
Crush stone B	Sand stone	56	76	Innocuous	2.66	0.80
Crush sand	Sand stone	52	92	Innocuous	2.62	1.91

Table 3 Mix proportion of concrete

W/C (%)	S/a (%)	Unit Weight (kg/m^3)					Admixture (Cx%)		Na_2O (Cx%)	K_2O (Cx%)
		W	C	S	G					
					Reactive	Non-reactive	PZ #70	#303A		
55	47	193	350	807	540	360	0.18	0.0027	0.91	0.98

3. RESULTS AND DISCUSSION

3.1 Influence of Coating Materials under an Accelerating Condition

The ratio of the expansion of coated specimens to that of non-coated specimens in percent with time passing is shown in Fig.1. Under an accelerating condition,the expansions of coated specimens decreases to 65% to 85% of that of the non-coated specimens at the expsure of 12 months regardless of the type of coating materials except No.8 shown in Table 1.

Based on Fig.1 and Fig.2 which shows the relationship between the expansion and the weight change of concrete due to water absorption,it is recoginized that the expansion increases with weight. However,the degree of the expansion with weight varies according to the type of coating materials. It seems unsatisfactory effect of surface treatment in reducing expansion was due to too much moisture content of the concrete specimens before coating and high moisture exposure condition.

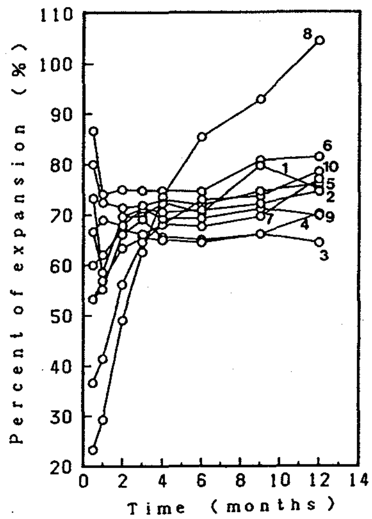

Fig.1 Expansion ratio of coating specimens to non-coating specimen

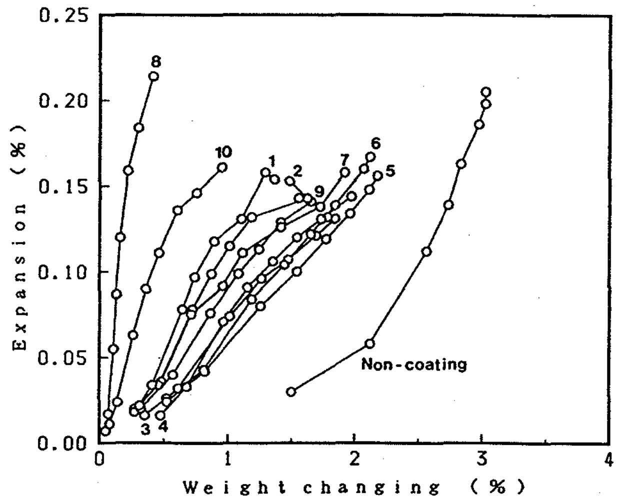

Fig.2 Relationship between expansion and weight changing of concrete

3.2 Expansion Reducing Effect of Surface Treatment in Outdoor Condition

The expansion with time is shown in Figs. 3 and 4. Compared to non-coated specimens,the expansion of coated specimens were remarkably small,regardless of coating applying time and there were no defference either among specimens with and without reinforcement.

The coatings of No.1 and No.4 specimens seems to prevent the passage of moisture in the liquid but not in the vapor. Therefore, in outdoor condition, a significant reduction in expansion can be produced by surface treatments as compared to those effect in high moisture condition tested in the 1st series.

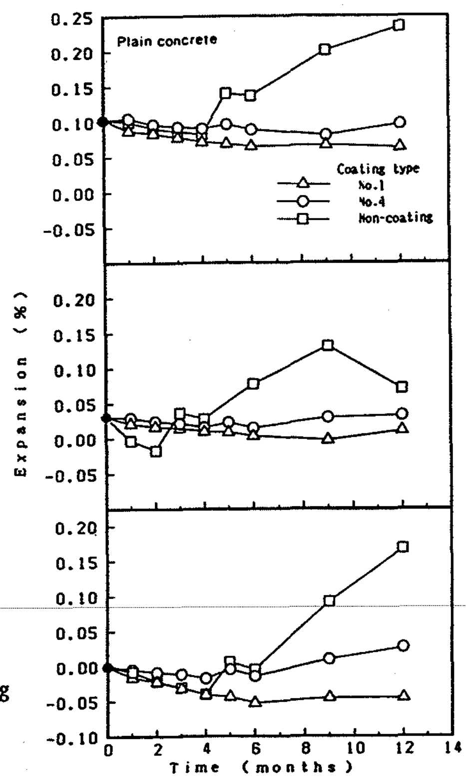

Fig.3 Expansion curves of concretes for several coating applying times

3.3 Influence of Moisture Content

The expansion curves of coated specimens which were stored in accelerating condition(40℃,>95%R.H.) is shown in Fig.5. For coating materials, epoxy which showed a superior ability to prevent water penetration through the surface from outside was used in this experiment.

Concrete specimens which had the moisture content of 6.0% showed larger expansion while those which had the moisture content of 4.5% or less showed less expansion. However, in the case of the moisture content of 4.5%, the expansion is expected to go on even after 12 months, because compared to the moisture content of 0% and 1.5%, the specimens of which the moisture content is 4.5% indicate a large degree of expansion with weight due to water penetration from outside as shown in Fig.6. Therefore, it is considered that surface treatment using highly impermeable coating materials would give a remarkable effect in reducing expansion under high moisture condition provided that the moisture content does not exceed approximately 4.5%.

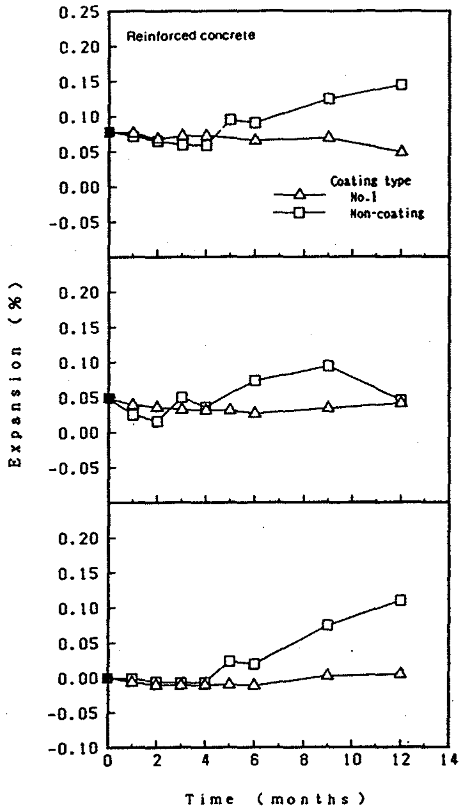

Fig.4 Expansion curves of concretes for several coating applying times

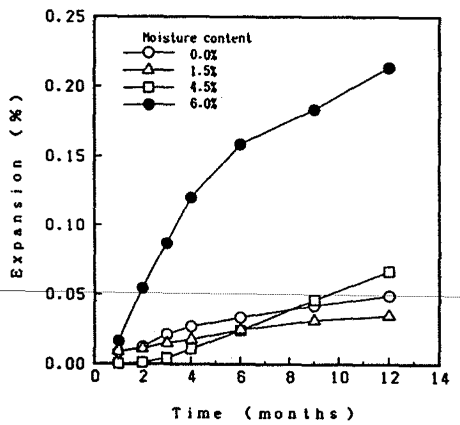

Fig.5 Expansion curves of coating concretes containing various moisture contents

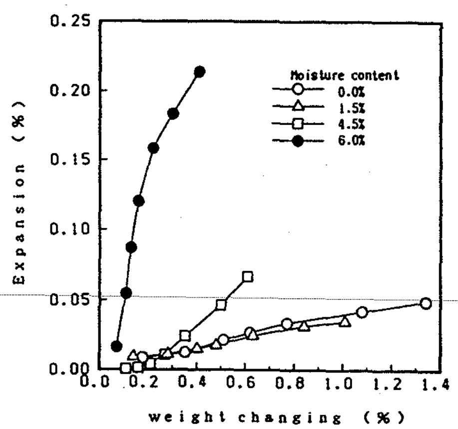

Fig.6 Relationship between expansion and weight changing of concrete

4. CONCLUSION

The major results obtained from the experiments are summarized as follows:

(1) Even under high moisture condition such as the accelerating test method of ASTM C 227, concrete surface treatment with highly impermiable coating materials gives a remarkable effect in reducing expansion due to ASR provided that the moisture content of concrete before being coated is approximately 4.5% or less.

(2) For concrete surface treatment materials, modified silane and polymer-cement composit give a remarkable effect in reducing expansion due to ASR under the outdoor condition in which moisture is expected to evaporate from the inside of concrete. The same effect can be obtained by these treatments in the case of both plain and reinforced concrete,and every applying times of coating for deteriorated concrete due to ASR.

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SUPPRESSION OF ALKALI-AGGREGATE REACTION BY CONCRETE SURFACE COATING

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1. INTRODUCTION

One of the causes of concrete deterioration is alkali-aggregate reaction. In this study, concrete specimens containing alkali reactive aggregate were coated with various organic and inorganic coatings to examine suppressing effect of cracking caused by alkali-aggregate reaction (AAR). Some of the specimens were coated prior to AAR acceleration test, and the others were coated after cracking has appeared. In addition, transmission rate of AAR accelerating substances and extension ratio at break for the coating films to discuss relationship between these basic properties of coating films and crack formation.

2. OUTLINE OF THE STUDY

2.1 Objective of the study

The study was conducted to make following points clear. (1) Suppressing effect of coating films on concrete stability, before and after cracking starts. (2) Which properties of coating films affect to suppress cracking? Flow diagram of the study is shown in FIGURE-1. The experiment mainly consists of two parts; the stability test using concrete specimens, and the measurements of coating film perfomances.

2.2 Procedures of the experiment

2.2.1 Performance tests of coating films   Thirty two coatings, i.e., heavy duty types, such as epoxy resins and a vinyl ester resin(with glass flake), acrylics and inorganic coatings for architecture, a new material, such as fluoropolymer coating were subjected to tests.

(1) Measurement of transmission rate

1) Water vapor transmission rate

a) JIS-Z-0208 (dish method)

b) Lyssy method (electrical method, using humidity sensor)

2) Water transmission rate

a) JIS-A-6900-5-10

b) Gravimetrical method (using coated specimens)

3) Ion transmission rate

Transmission rate of Na^+ and Cl^- was determined after a month's immersion test using double cells (3%NaCl soln./distilled water) which were divided by a free film.

(2) Extension ratio at break of coating films to over coat surface cracking concrete

Extension ratio at break was determined using Instron type tester with crosshead speed 5mm/min. at 20 C.

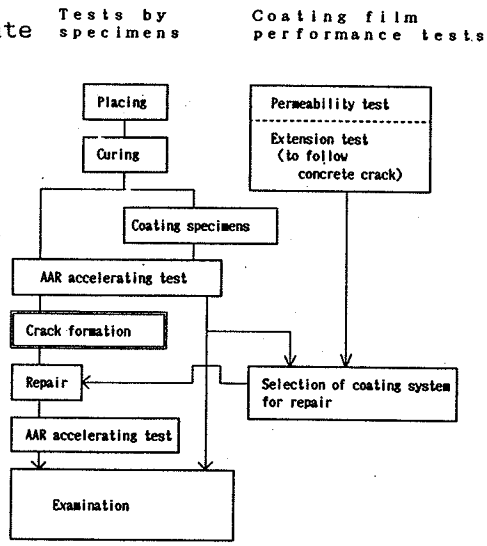

FIGURE-1 The flow diagram of the study

2.2.2 Preperation of concrete specimens and the procedure of AAR acceleration test

(1) Mix proportion of concrete

Mix proportion of concrete is shown in TABLE-1. Specimens were subjected to the test after 28 days' curing in water.

No.1 and No.3 were subjected to the test as uncoated, No.2 as both coated and uncoated. Specimens were cubic, having dimension of 100 mm on each side.

TABLE-1 Mix proportion of concrete

No.	(1)	W kg/m³	C kg/m³	S kg/m³	G kg/m³	(2)	NaCl kg/m³	W/C %	S/A %	Air %	Slump cm	(3)	(4)	(5)
Standard		181	362	686	993	0.025	-	50	40	5.0	8.0	-	-	-
1	T	180	362	687	993	0.025	-	50	40	5.2	11.3	0.1	502	3.27
2	T	176	362	691	993	0.025	8.8(6)	50	40	5.4	13.8	0.7	481	3.09
3	K	180	362	687	993	0.025	-	50	40	5.2	11.3	0.1	459	3.30

(1) Sort of aggregate

T: Judged potentially reactive through ASTM C 289. Rc=183, Sc=639

K: Judged nonreactive through ASTM C 289. Rc=34.3, Sc=39.6

(2) A E Agent (%)

(3) Surface moisture in fine aggregate (%)

(4) Compression strength (kgf/cm²)

(5) Modulus of elasticity (10^5kgf/cm²)

(6) As total $R_2O(Na_2O+0.658K_2O)$=8kg/m³

(2) Coating of specimens

1) Coating before crack appeared

Specimens were treated with disc sander (cc#50 paper) prior to coat after 28 days' curing in water. They were coated all of the surfaces. Targets of dry film thickness were 100 μm as standard, 350 μm as specified. Coatings contained neither primer nor putty inorder to evaluate their properties directly. They were dried for 7 days at 20 °C prior to subject to the test.

2) Repair coating after crack appeared

Target of dry film thickness was specified value of each coating system. Coating films were evaluated as film system (each contains primer, putty, top coat). Pretreatment and coating method were as same as mentioned above.

(3) AAR acceleration test

Specimens were subjected to the wet/dry cycle exposure test. They were placed under the sun periodically being sprayed water.

Spraying period was 30 minutes, four times a day. It was obvious that the wet/dry cycle test accelerated AAR more effectively than immersion or high temperature humidity test.

3. RESULTS AND DISCUSSION

3.1 Performance tests of coatings

Test results were shown in TABLE-2. Vinyl ester, epoxy and polyurethane resins were highly resistant to water or water vapor transmission, on the other hand, inorganic and acrylic coatings were highly transmittable. Inorganic and acrylic coatings were highly transmittable to sodium ions, while vinyl ester, epoxy and polyurethane coatings hardly transmitted sodium ions.

TABLE-2 Test results of coatings

Coating		1) F.T. (μ)	2) M	V V T 3) a	V V T 3) b	4) VT(1)	5) VT(2)	6) Na^+	7) Cl^-	8) E.R.
Epoxy	Epoxy (primer)	-	A				0.017			
		-	B			23.0	0.079			
		-	C			78.0	0.024			
	Epoxy (int.)	100	A	3.4		7.0	0.009	‡	6.96	0.93
		100	B		1.7		0.007			1.2
		100	C	1.4		2.1	0.003	‡	‡	4.1
	Epoxy(top)	100	A	2.7		6.9		‡	0.10	2.2
	Epoxy (flexible)	100	A	5.8		10.0	0.015	‡	1.38	130
		100	C	2.8				‡	‡	25
	Epoxy (high-build)	350	A	1.4		1.7	0.025	‡	‡	1.8
		350	B		2.5			‡		1.5
		350	C	0.2				‡	‡	5.8
Urethane	Ur.(primer)	-	B			12	0.004			
	Ur.(top)	100	B		35			‡		41
		100	A	2.6		21		‡		77
	Ur.(flex.)	100	B		52			‡	1.38	68
In-organic	Water repellent	-	A	81		60	0.10	900	400	
		-	B			20	0.14			
		-	C			69	0.16			
	Inorganic	350	A			4900	1.40	31000	53700	55
		100	C	240					944	55
	P.cement (flexible)	350	B		35	7.0		6.88		94
V.Es.	Glass flake	350	B		0.6	‡		‡		1.5
Others	Acrylic (solvent)	100	A	13.0		10.0	0.048	0.17	2.43	171
		100	B		160			2490		28
		100	C	13.0				20.5	12.6	
	Acrylic(em.)	100	B		1960	1560		21000		8.3
	Acrylic(em.) (flexible)	100	C	200					4330	940
	Polybutadien (flexible)	100	C	29				‡	‡	260
	Vinyl	100	B		34			‡		3.0
	Chld.rubber (flexible)	100	C	2.9				19.3	14.6	250
	Fluoric	100	B		26			‡		44

1) Film thickness
2) Paint maker
3) Water vapor transmission rate ($g/m^2 \cdot 24hrs.$)
a: Measured through JIS-Z-0208 (20 ℃)
b: Measured through Dr.Lyssy method (40℃)
4) Water transmission rate ($ml/m^2 \cdot 24hrs.$)
5) Water transmission rate (ℓ/24hrs.)
6) Na^+ transmission rate ($mg/m^2 \cdot 24hrs.$)
7) Cl^- transmission rate ($mg/m^2 \cdot 24hrs.$)
8) Extension ratio at break point (%)

3.2 AAR acceleration test of uncoated or originally coated specimens

All of the No.1 and No.3 specimens in TABLE-1 were subjected to the test as uncoated. Thirty in 86 of No.2 specimens were subjected to the test after being coated, while 56 as uncoated.

3.2.1 Crack formation in uncoated specimens As shown in FIGURE-2, cracking was observed in 90 per cent of No.2 specimens after 12 months, 100 per cent after 24 months. While, No.1 specimens (potentially reactive, but without NaCl) were observed no cracking after 17 months, and 7 in 54 were observed to have been cracked (fine crack; under 0.05mm) after 24 months. The crack was considered to originate in AAR, because white gel was observed and it was remarkably accelerated to appear by addition of NaCl.

3.2.2 Crack formation in specimens which have been coated before crack appeared Crack was observed in 33 per cent of originally coated specimens after 16 months, and in 48 per cent after 23 months. Crack suppressing effect of coatings was obvious.

Relationship between crack formation and water vapor transmission rate or extension ratio is shown in TABLE-3 and FIGURE-2. As shown in FIGURE-2, cracking formation ratio was low in A type, and it formed in early time, no increace was noticed by passage of time.

C type showed low ratio of crack formation too, but it increaced by

TABLE-3 Number of cracked specimens

	Coating properties		Crack	Number of specimens		
	Transmission	Extension		11months	16months	23months
A	Low	Small	Finded Non (%Finded)	1 4 20	1 4 20	1 4 20
B	Medium	Small	Finded Non (%Finded)	4 7 36	6 5 55	8 3 73
C	Medium	Large	Finded Non (%Finded)	0 7 0	1 6 14	4 3 57
D	High	Large	Finded Non (%Finded)	0 1 0	0 1 0	1 0 100
E	High	Small	Finded Non (%Finded)	2 1 67	2 1 67	2 1 67

Transmission; Low Water vapor transmission rate 0 - 1 g/m² · 24hrs.
rate Medium 1 - 100
High 100 <
Extension ; Small 0 - 10 %
ratio Large 20 % <

passage of time.

The reason was considered as follows. As the specimens were coated under well dried condition, if they were coated with nontransmmittible coatings, expansion from AAR was considered to be suppressed.

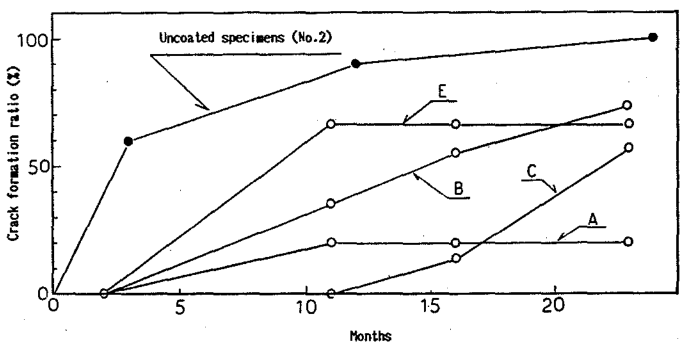

FIGURE-2 Crack formation ratio with time

In case that a specimen is coated with a coating which is comparatively low water vapor transmittible and having high extension ratio, it is considered that the coating may yet suppress cracking of the specimen after it expands to a certain extent, because the coating film will extend following to expantion of the specimen and may keep a certain extent of transmission resistance. On the other hand, in case that a coating film has comparatively high transmission resistance, but poor in extending property, micro-fault might occur in it according to the specimen expands, as a result, transmission resistance of the film might decrease and water transmission into the specimen will occur, resulting in further expantion of the specimen and crack formation on it.

3.2.3 Crack formation in repair coated specimens After 4 months' acceleration test, cracked specimens of No.2 were repaired with 13 types of coating systems shown in FIGURE-3 and subjected to the acceleration test again.

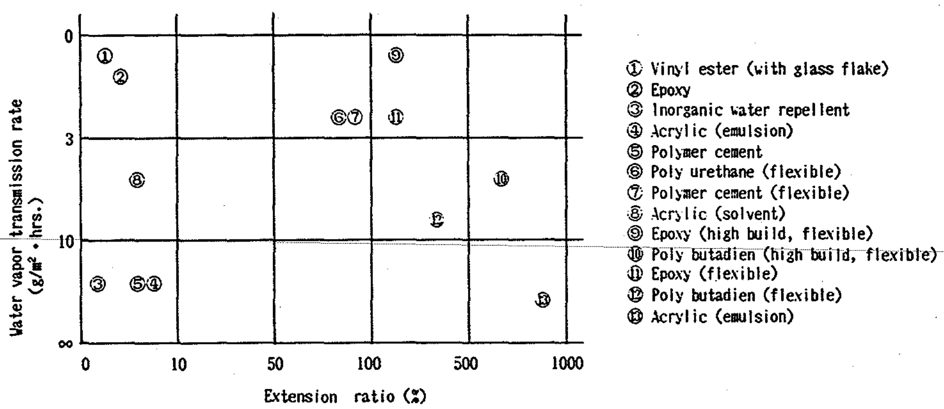

FIGURE-3 Properties of selected repair systems

Each system was coated on two specimens, having small crack and large one. No crack growth was observed on the coating surface after 20 months' acceleration test. Then coating films were removed to examine crack growth under the films, but no growth of crack was observed.

4. CONCLUSION

(1) Cracking due to AAR of concrete which contains alkali reactive aggregate is expected to be suppressed by surface coatings.
(2) As shown in this study, if all of the surface are coated with a coating of low transmission rate towards water, cracking of concrete is considered to be suppressed effectively.
(3) Concrete cracking could not be prevented perfectly by coating concrete surface before crack formation, while crack growth was effectively suppressed by coating after crack formed, because in the former case too large extension ratio of the coating film would be required to cover crack width, on the other hand requirement for extension ratio of the coating film would be much smaller than that of latter case.
(4) In the actual concrete structures which might be inevitable to have uncoated portion from where water can permeate, other coating system might be required. (The authors are conducting an further experiment, in which coated concrete specimens having uncoated portion are exposed under the sun being buried uncoated portion in the earth.)

5. ACKNOWLEDGMENT

This report is the result of the study which was performed by M.Takeyoshi, K.Noda, and T.Taki when engaging as outside researcher in Public Works Research Institute in 1985-1986. The authors are grateful to the participants in the Institute who gave guidance and advice.

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on Alkali-Aggregate Reaction

DEVELOPMENT OF INJECTION MATERIALS AND THE STANDARD FOR REPAIRING DAMAGE TO STRUCTURES CAUSED BY AAR

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1. ABSTRACT

In order to establish a guideline for repairing damage to concrete structures caused by Alkali-Aggregate Reaction (AAR), new repairing materials and a standard were desired.

As for repairing materials, this paper dealt with injection materials and these materials were selected through 20 experiments.

Through a series of studies, a repair standard and a quality specification for injection materials have been developed.

2. INTRODUCTION

Because the behaviour of deterioration caused by AAR in concrete structures is different from that of cracks caused by outside sources, a technique for repairing those cracks up to now had been not useful for cracks cuased by AAR.

The purpose of our study was, firstly, to develop by experiment repair materials, the characteristics of which should be taken into account the

Table 1 Test Materials

Classification of Material	Outstanding Properties	Mixed Ratio (weight)
1 Epoxy Resin Group		
(1)Regular (Hard)	tensile strength ≧ 300kgf/cm²	Masteragent(M)
(2)Flexible - 1	elongation ≧ 50% & Thixotropy	/ Hardner(H) = 2 / 1
(3)Flexible - 2	elongation ≧ 100%	
(4)Flexible - 3	elongation ≧ 200%	
2 (5)Polyuretane	elongation ≧ 300%	M / H = 100 / 2
3 Polymer Cement Group		
(6)Acryl	cement slurry injectable to wet section	Powder / Latex / Water = 9/1/2.5
(7)Epoxy		Slurry / Powder / Cement = 8/5/10
4 (8)Slag	colloidal state injectable to wet section	Powder / Nonshrink / Water = 100 / 2 / 70
5 (9)Sealant:Polysulfide	elastomeric sealing compound	M / H = 10 / 1

behaviour of both damages by AAR and of the structure itself. (This paper covers injection materials. A further paper[1] covers coating materials.) And our ultimate objective is to indicate a practical standard for repairing damage by AAR.

This study was a part of the projects which the Ministry of Construction organized entitled "Development of Techniques for Improving Durability of Concrete Structures".[2]

Table 2 Test Items and Methods

	Test Item	Test Method	Test Condition
Basic Physical Test	coefficient of thermal expansion	T M A	
	visco-elasticity	visco-elastic spectromerter	(measurement of Tg)
	tensile strength	JIS K 7113 ASTM C 190	-20,-10,0,20,40,60 ℃
	elongation percentage	JIS K 7113	-20,-10,0,20,40,60 ℃
	Adhesives in shear by tension loading	JIS K 6850	-20,0,20,40,60 ℃
	Water Vaper Transmission Rate	JIS Z 0208	20,40 ℃ ; RH 90 %
	Hardening shrinkage percentage	JIS A 6024	20 ℃
	Adhesion to Mortar	JIS A 6024	-10,0,20,40℃ ; dry & wet
Workable Test	Injectable depth	Reinforcing Concrete Bars (50×80×120cm; 20 bodies)	(except sealant)
	Injectable Crack Width	Checking by drilled-out cores	5,20,35 ℃
	Viscosity	JIS K 6833	
	Pot Life	JIS K 6833	
	Setting Time	JIS A 6024 Code by J C E A	
Durability Test	Water Resistance	JIS K 7114	20℃,30days; Ion exchange water
	Alkali Resistance	JIS K 7114	20℃,30days; Saturated $Ca(OH)_2$
	Salt-Water Resistance	JIS K 7114	20℃,30days; Solution of 3% salt
	Durability of Adhesion: repetition of drying and moistening	JIS A 6024	68℃30cycles (1cyc = Dry & Moist,24HR)
	Durability of Adhesion: fatigue by bending	JIS A 6024 JIS K 7188	20℃ 10^6 cycles
	Durability of Adhesion: Resistivity to Creep	JIS A 6024 JIS K 7116	bending deflection after 1000Hrs at 20℃
Effect Test	Resistance for progressing of crark	reinforcing concrete bars (25x25x100cm ; 12bodies)	R_2O = 2.3% repair(Injection,Coating) accelerated curing time ; 6M(+repair) + 6M

3. DEVELOPMENT OF INJECTION MATERIALS FOR REPAIR

3.1 Outline of Investigation

Experiments were carried out in order to judge whether injection materials retained the physical properties which were desired for repairing cracks due to AAR in use. Using the nine kinds of injection materials composed from organic and inorganic materials shown in Table 1, 1) basic physical tests of the material, 2) workability tests for repair work and 3) durability tests were carried out. As a result of these tests three of the materials were eliminated. Then, 4) a test for ascertaining the effect of the injection method for repair was carried out. These test items and methods are shown in Table 2. The test were mainly conducted according to Japanese Industrial Standards. Some of the conditions or definitions of physical values were, however, defined by ourselves. Discussion will be required regarding items defined in this way.

3.2 Test Results and Selection of Injection Material for Repair

Figure 1 represents elongation percentages at each temperature as one of the results of basic physical tests. Similar elongation factors at temperatures other than 20° C, although desirable, weren't expected because of the materials' thermal sensibility. It should be noticed that polysulfide's elongations at temperatures other than 20°C exceeded the capacity of test machine.

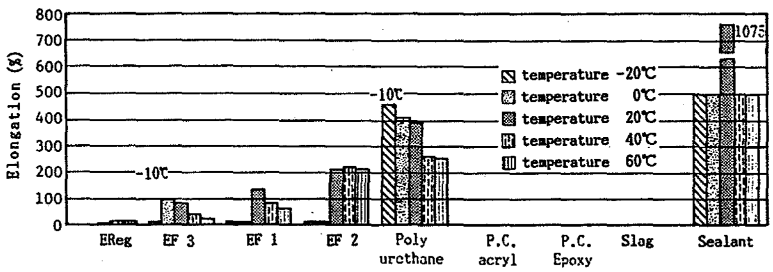

Fig. 1 Result of Elongation

Table 3 Results of Workability Test

Item	Temp (℃)	Epoxy Resin Group (1)Regular	(2) F 1	(3) F 2	(4) F 3	(5) Poly -urethan	Polymer Cement (6)Acryl	(7)Epoxy	(8) Slag	(9) Sealant
	5	158	83	168	175	143		118		
Injected Depth (cm)	20	120	130	158	158	130	95	118	155	
	35	120	108	158	158	130		80		
	5	0.08	0.20	0.04	0.04	0.08		0.15		
Injectable Minimum Width of Crack (mm)	20	0.08	0.10	0.04	0.08	0.08	0.25	0.15	0.04	
	35	0.15	0.10	0.10	0.10	0.04		0.20		
	5	3100	34500	1700	1200	5400	8000	5800	300	1050000
Viscosity (cps)	20	620	25000	550	400	2075	5600	1850	200	890000
	35	185	12000	230	190	800	5000	1050	150	850000
	5	405	120	300	420	120	>300	>300	>300	240
Pot life (min)	20	92	50	56	69	60	>300	>150	>300	240
	35	18	15	15	18	45	180	60	230	120
	5	30	20	45	>48	13	9	12	18	45
Settig Time (HR)	20	10	6	16	23	5	3.5	7.5	6	16
	35	3.5	2.5	6.5	9.5	2	1.5	2.5	1.5	6

Table 3 shows the results of the workability tests. To each concrete bar (50x80x120cm) three kinds of crack width (0.5, 1, 2 mm) were introduced by driving in wedges at each temperature (5, 20, 35°C). After the cracks were repaired by the injection technique using coloured materials, the injected depth and injectable crack width were measured not only visually and ultrasonically from outside the bars, but also by examing drilled-out cores.

Figure 2 and Table 4 represent results of durability tests of adhesion. Photo 1 shows a test for resistivity to creep. On these durability tests of adhesion, held values of adhesive strength and deflection after repetition or long-term loading were evaluated to the strength or deflection at static failure. The injection width at these three tests were 0.5, 2 and 5 mm.

As a result of the durability test, it was found that

1) Epoxy resin flexible type 3 and polyurethane had considerable elongation capacity, but less fatigue strength and less resistivity to creep respectively, and when they failed, the failures were interfacial.
2) Slag in colloidal state had, on the other hand, little elongation and less held ratio of adhesive strength in repeated drying and moistening tests.

These three materials were, therefore, judged to lack durability of adhesion and ruled out as injection materials for reparing cracks due to AAR.

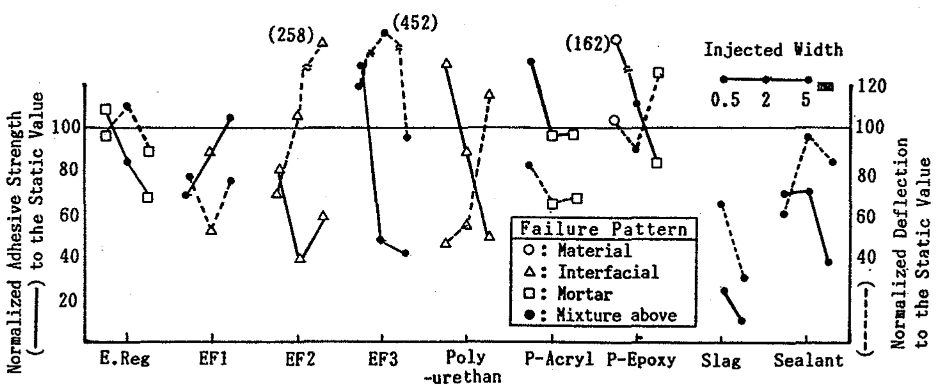

Fig. 2 Results of Repetition Test of Drying and Moisterning

Table 4 Results of Durability Test of Adhesion (at 20℃)

Test Material	Fatigue Strength (kgf / cm²)			Bending Deflection (mm)			
	Static	Fatigue	Ratio(%)	Static	Creep	Ratio(%)	Ultimate time(Hr)
(1) E-Regular	71	44	63	0.42	0.29	69	> 1000
(2) E-F-1	67	23	35	1.17	0.41	35	941
(3) E-F-2	86	35	42	0.58	0.33	57	994
(4) E-F-3	60	15	25	0.78	1.24	35	944
(5) Polyurethane	14	8	56	1.97	0.30	16	560
(6) P-Acryl	52	26	44	0.32	0.13	40	480
(7) P-Epoxy	63	31	47	0.37	0.22	61	616
(8) Slag	25	12	47	0.25	0.08	32	768
(9) Polysulfide	32	23	71	6.5	2.6	40	912

3.3 Repair Effect of the Injection Method

Resistance effect against crack progression of the injection method was investigated by using concrete bars (25x25x100cm) with low reinforcement ratio 0.4% (reinforcing bar's strains were measured). The reactive coarse aggregate was bronzite andesite from Teshima Island which is the same as used on the Hanshin Expressway's structures damaged by AAR[3] and the mixture ratio was 50%. The injection material was epoxy resin flexible type 2. Figure 3 is an example of crack maps and shows cracks before and after repair work. As a result of this test, the repair effect of the injection method was shown with a comparison between repaired and unrepaired parts, that is, 1) no change of injected cracks, 2) enlargement or expansion of unrepaired crack widths and 3) new cracks in the unrepaired surface during accelerated curing. The repair effect of both the injection and the coating methods wasn't, however, shown clearly in this experiment.

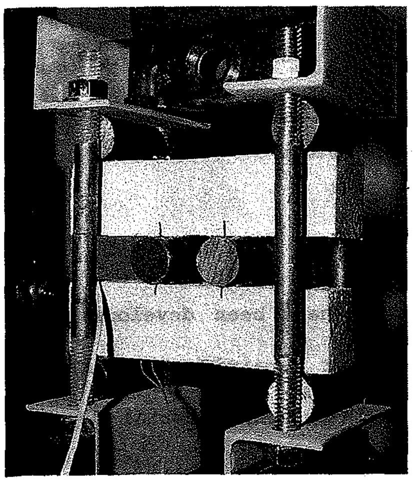

Photo 1 Resistivity to Creep Test

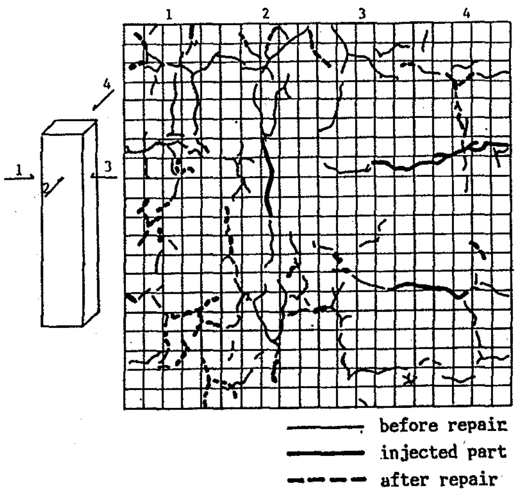

Fig. 3 Crack Map

4. REPAIR DESIGN

The aim of repairing damage to concrete structure caused by AAR, the load carrying capacity of which has not been significantly reduced from the original value,[4] was the rehabilitation of the durability of the concrete structure. Our discussion about how to repair damage due to AAR produced a practical standard for repair in Table 5. By diagnosis damage should be classified into 4 groups, 2 comprising crack behaviour - whether it is progressive or stable, and 2 comprising crack width - 0.2 ~ 5 mm or over 5 mm.

Table 5 Standard for Repair

State of Crack	Crack Width(mm)	Procedure of Crack: Injection	Procedure of Crack: Plug	Coating System
A Progressive	0.2 ~ 5	Epoxy III (EF2)	—	Flexible type
	> 5	—	Sealant	with thick film
B Stable	0.2 ~ 5	Epoxy II (EF1)	—	Flexible type
		Epoxy I (E·Reg)		Hard type
	> 5	—	Sealant	Flexible type
		—	Polymer cement	Hard type

The optimum injection material is determined by a combination of crack width and whether or not it is progressive. Quality specification of injection materials is shown in Table 6. In case of repair work, results of workability test in Table 3 should be especially taken care because of each value in Table 6 is only at 20°C. Tables 5 and 6 should be improved through future repair works and subsequent investigations. Follow-up inspection is, therefore, important.

Table 6 Quality Specification for Injection & Plug Materials (at 20℃)

Item	Injection M.			Plug M.	
	Epoxy I	Epoxy II	Epoxy III	Polymer cement	Sealant
State fo Crack	B		A	B	A,B
Crack Width (mm)	0.2	~	5		> 5
Viscosity (cps)	1000 >	4(1)	1000 >	10000 >	No recognition of film droop
Pot Life (min)		> 30		> 30	> 240
Setting Time (hr)	16 >		24 >	16 >	24 >
Hard Shrinkage (%)		0.1 >			0.1 >
Elongation (%)	—	50 ~ 100	100 ~ 150	—	> 800
Adhesive Strength to Mortar (kgf/cm²)		> 60		> 60	Failure at over 10mm of deflection
Rate of Durability on Adhesive Strength (%) (2)		> 60			> 60

note (1) ; coefficient of thixotropy (value at 2rpm/20rpm)
(2) ; percentage to the setted value

5. CONCLUSION

The conclusions are summarized as follows:

1) Six kinds of desirable injection materials have been developed through experiments with 20 items.
2) A practical standard and a specification for repairing works have been established including test methods for materials.
3) In this repair standard, optimum repair (injection) material depends on crack width or its stability.

6. ACKNOWLEDGEMENT

The authors wish to thank Prof. K. Okada (Chairman of the Committee of Development of Techniques for Improving Durability of Concrete Structures), members of the committee and Mr. Y. Mori (Public Work Research Center) who offered helpful advice and valuable discussion. Thanks are also due to Mr. K. Domon (Sho-Bond Construction) and many colleagues for their assistance.

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8th ICAAR 1989 KYOTO

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THE ESTIMATE FOR DETERIORATION DUE TO ALKALI–SILICA REACTION BY ULTRASONIC SPECTROSCOPY

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ABSTRACT

This paper deals with nondestructive testing of concrete by ultrasonic spectroscopy to assess the deterioration of concrete structures due to alkali-silica reaction (ASR). The response function and its energy of specimens were calculated by applying a linear system theory, and the deterioration of specimens was assessed by the energy of response function.

Tests were carried out for mortar and concrete with reactive bronzite andesite crushed stone. Specimens were cured in the chamber with 40°C and 100% relative humidity for four months after cured in water. The deterioration of specimens was assessed while the reaction was accelerated, and after the reaction was accelerated, cores were drilled from the reinforced concrete specimens and expansion of the core was measured.

By the deterioration due to ASR, the dynamic modulus of elasticity of test specimens was decreased extremely, and the energy of response function and pulse velocity of deteriorated concrete specimens were decreased to about 88% and 95% of non-reactive specimen, respectively. The more concrete was reinforcement, the larger the core was expanded. It is easy to assess the deterioration of concrete structures due to ASR by ultrasonic spectroscopy method proposed in this study.

1. INTRODUCTION

There is a growing need for some form of assessment to be applied to concrete structures. Assessment of the potential durability is also another aim of testing which is increasing in importance. It may be very important to assess the deterioration of concrete due to alkali-silica reaction (ASR) by nondestructive testing evaluation for future maintenance and repair.

In this study, test methods for ultrasonic spectroscopy and pulse velocity were used to assess the internal texture changes of concrete due to ASR. Ultrasonic spectroscopy is the diagnosis in materials evaluation and in characterization of flaws in metal components with frequency spectra of ultrasonic pulse echo[1]. It is necessary to measure the pulse echo and to use the broad band transducers. Although the pulse echo through concrete is very difficult to measure, deterioration due to ASR can be assessed by the response function which was calculated by applying a linear system theory to the ultrasonic spectral analysis.

2. RESPONSE FUNCTION OF CONCRETE SPECIMEN BY LINEAR SYSTEM THEORY

As shown in **Figure 1,** an output $y_o(t)$ from a linear system for an input x(t) is presented as,

$$y_o(t) = x(t)*h_o(t) \quad (1)$$

where a notation * shows a convolution integral and $h_o(t)$ denotes a response function of the linear system. The response function h(t) of a linear system connected in series two linear systems and an output y(t) from this system for the input x(t) is presented, respectively, as

$$h(t)=h_o(t)*h_1(t) \quad (2)$$

$$y(t)=x(t)*h(t)$$
$$=x(t)*h_o(t)*h_1(t) \quad (3)$$

Let $X(f), Y(f), Y_o(f), H(f), H_o(f)$ and $H_1(f)$ be Fourier transforms of $x(t)$, $y(t), y_o(t), h(t), h_o(t)$ and $h_1(t)$, respectively. Then, eq.(1) and (3) are transformed as

$$Y_o(f)=X(f)\ H_o(f) \quad (4)$$

$$Y(f)=X(f)\ H_o(f)\ H_1(f) \quad (5)$$

From eq.(4) and (5), the frequency response $H_1(f)$ of a specimen is given by

$$H_1(f) = Y(f)/Y_o(f) \quad (6)$$

The response function $h_1(t)$ of the specimen is obtained by the inverse Fourier transform of $H_1(f)$.

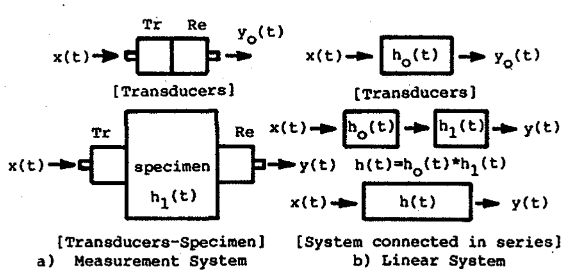


Figure 1 Relation between Measurement and linear System

3. TEST PROGRAMS

3.1 Test Specimens

The test program was divided into two parts. In **Part I,** The deterioration of mortar bars (1x1x11 inches) due to ASR was assessed. Proportion of the dry materials for the test mortar was used 1 part of cement to 2.25 parts of graded aggregate by weight. An amount of mixing water was decided such as to produce a flow of 180 to 200 mm. Mortar was made by ordinary portland cement containing 0.63% equivalent Na_2O (Mix:**N**). In order to accelerate the reaction, the alkali content was adjusted to 1.5% (Mix:**MR1**) and 2.0% (Mix:**MR2**) by the addition of NaCl, and the specimens were cured under the accelerated condition of 40°C and R.H. 100%

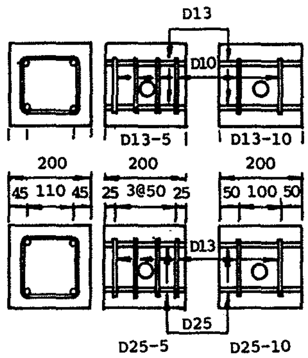
Table 1 Details of Concrete specimens

KIND	axial steel As	axial steel Pl (%)	stirrup As	stirrup s (cm)
D13-5 D13-10	D13	1.28	D10	5 10
D25-5 D25-10	D25	5.07	D13	5 10

after curing in water for three weeks.

In **Part II,** the deterioration of concrete due to ASR was assessed. The cylindrical and the cubic reinforced concrete specimens was prepared. The steel ratio and arrangement of steel bars are shown in **Table 1** and **Figure 2.** Mix proportion of concrete is shown in **Table 2.** The following materials were used:

- Yasu river sand as normal fine aggregate;
- Bronzite andesite crushed stone as reactive coarse aggregate and Takatsuki crushed stone as normal aggregate, maximum size=20 mm;
- Ordinary portland cement containing 0.65% equivalent Na_2O.



Note: ○ is Sampling Position

Figure 2 Specimens and Arrangements of Steel Bars

The total equivalent Na_2O of reactive concrete was 6.88 kg/m^3 which was adjusted by the addition of NaCl, and the specimens were cured under the conditions of 40°C and R.H. 100% after curing for two weeks in water at 20±2°C.

Table 2 Mix proportion of concrete

Mix	required slump (mm)	required air (%)	W/C (%)	S/a (%)	unit weight (kg/m^3) W	C	S	GN	GR	NaCl
N6		6		40			685	1064	0	0
R6	80	6	50	40	172	344	685	532	504	8.86
R2		2		42			763	545	517	8.86

GN is Non-Reactive Aggregate and
GR is Reactive Aggregate

3.2 Test Procedures

In **Part I,** response function, ultrasonic pulse velocity, length change and dynamic modulus of elasticity were measured while the reaction was accelerated. Pulse velocity was measured in accordance with ASTM C 597-83.

In **Part II,** response function, pulse velocity and dynamic modulus of elasticity were measured as same as Part I, and then concrete cores with diameter of 50 mm were taken from the reinforced concrete specimens after four months. Residual expansion and pulse velocity of the cores were measured. The length changes of mortar and core were measured in using the apparatus specified in ASTM C 490 and by Lenzer's method[2] respectively.

The response function of specimens was calculated by the fast Fourier transform method as follows: the rectangular impulse having 1 μs in width and 22 volt in amplitude produced by a function generator was impressed on the transmitter. An output signal from the receiver was measured at a sampling interval of 2.44 μs and then analyzed by the fast Fourier transform to obtain the response function of specimens. The signals were analyzed up to the frequency of 102.3 KHz. The transducers with the resonance frequency of 54 KHz were used.

4. TEST RESULTS AND DISCUSSIONS

4.1 Part I

Figure 3 shows the changes of expansive strain, pulse velocity and energy of mortar bars while the reaction was accelerated. In this study, the energy is defined as the integral of energy spectral density of response function. This is the total energy of response function integrated in the frequency domain from DC to 102.3 KHz. The pulse velocity of deteriorated mortar bars decreased with the expansive strain. The energy of two deteriorated bars (Mix **MR1** and **MR2**) decreased abruptly to about 50% of the non-reactive bar **N**, and the decrease of the energy for **MR1** was slowly than for **MR2**. The pulse velocity and the energy of the deteriorated bars decreased until a minimun value and then recovered. It may be concluded that this recovery is caused by the silicate gel filling the cracks.

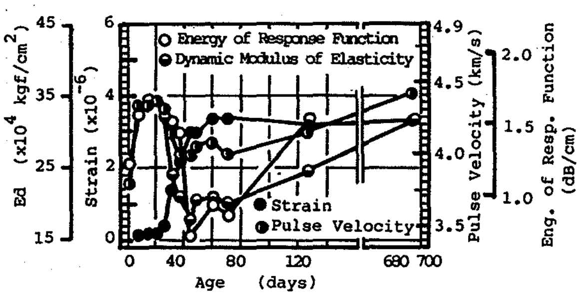


Figure 4 Comparison of Tests Results for Mortar made by Mix **MR2**

Figure 4 compares the changes of expansive strain, pulse velocity, energy and dynamic modulus of elasticity for **MR2**. The change of dynamic modulus of elasticity with the expansion due to ASR was significantly large, but dynamic modulus of elasticity is not used to assess the deterioration of concrete structures. The change of the energy due to expansion was larger than that of pulse velocity.

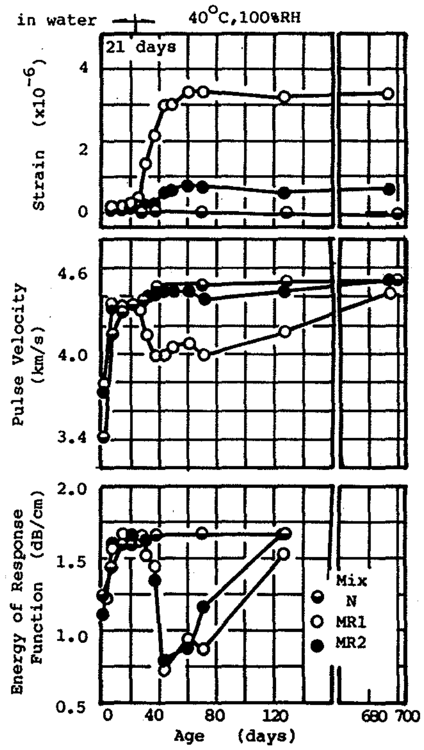


Figure 3 Changes of Strain, Pulse Velocity and Energy for Mortar Bars

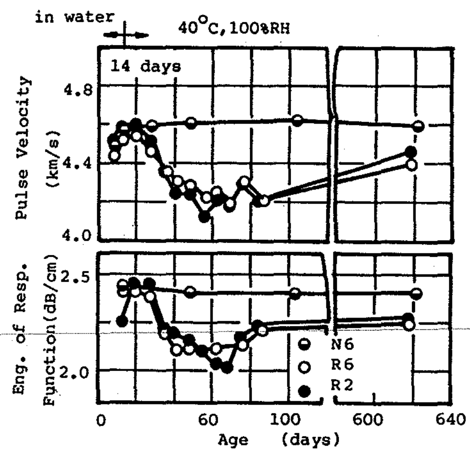


Figure 5 Changes of Pulse Velocity and energy of Concrete

4.2 Part II

Figure 5 shows the changes of pulse velocity and energy of the concrete specimens while the reaction was accelerated. Although the energy and the pulse velocity of mortar bars decreased immediately after the reaction was accelerated, these of concrete cylinders decreased after one week from the start of accelerated cure. It may be influenced by the difference of volume-surface area ratio (V/S) because pulse velocity of cube specimens decreased about one week slow than that of cylinder as shown in **Figure 7.** The V/S ratios of cube and cylinder were, respectively, 3.3 cm and 2.0 cm.

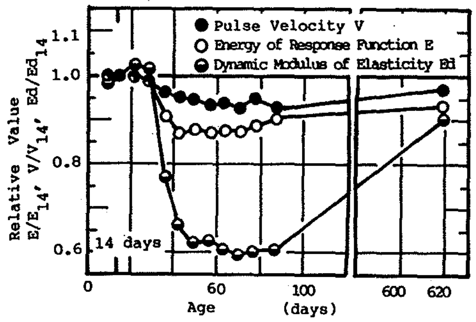


Figure 6 Comparison of Test Results for Mix **R6**

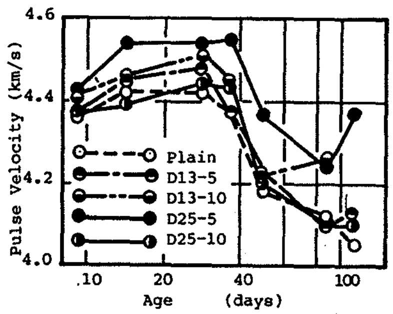


Figure 7 Changes of Pulse Velocity of Concrete

Figure 6 illustrates the results of non-destructive testing for concrete cylinders of Mix **R6** as the value at the age of 14 days is one. Although the decreases of energy and pulse velocity were occurred almost simultaneously, the energy and the pulse velocity of deteriorated concrete were about 88% and 95% of that at the age of 14 days respectively. Although pulse velocity method is easily than ultrasonic spectroscopy method, the energy calculated by ultrasonic spectroscopy may be advantageous for assessing the deterioration of concrete structures due to ASR.

Figure 7 shows the changes of pulse velocity of concrete made by **Mix R6** encaged by axial steels and stirrups while the reaction was accelerated. The pulse velocity of D25-5 which had most reinforcement was larger than that of

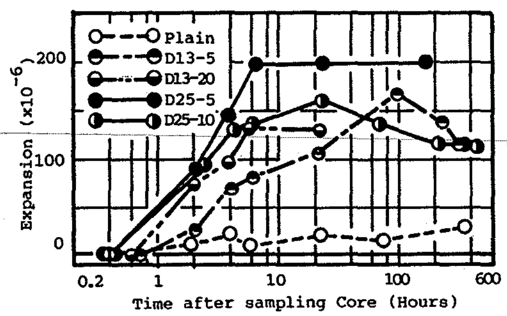


Figure 8 Changes of Residual Expansion of Concrete

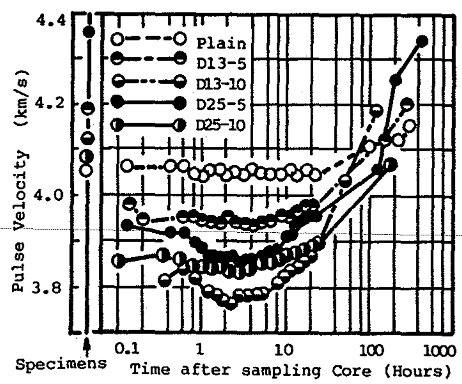


Figure 9 Changes of Pulse Velocity of Concrete Cores

another specimens, and the decrease of pulse velocity due to expansion was most slowly. It may be concluded that the expansion of this specimen due to ASR was restricted largely by reinforcement more than another specimens.

Figures 8 and 9 show the residual expansion and the changes of the pulse velocity of concrete cores sampled after the age of four months respectively. The expansion of plain concrete was very small, and the pulse velocity was almost constant. On the other hand, the residual expansion of reinforced concrete grew large with the area of reinforcement, and the pulse velocity immediately after drilling the concrete core was decreased very largely. Furthermore, the pulse velocity of the core that was taken from the cube with large area of reinforcement decreased until a minimum velocity, and then recovered to near or more than that test specimen. It may be concluded that this recovery of pulse velocity is caused by the silicate gel filling the cracks.

5. CONCLUSIONS

The following conclusions may be drawn that was obtained from this study.

(1) By the deterioration due to ASR, dynamic modulus of elasticity of mortar and concrete was decreased extremely. The energy of the response function and the pulse velocity of deteriorated concrete were decreased to about 88% and 95% of the non-reactive concrete respectively.

(2) The energy of the response function was significantly affected by the change of the texture due to ASR more than for the pulse velocity. Although the pulse velocity method is easily than the ultrasonic spectroscopy method proposed in this study, the energy calculated by ultrasonic spectroscopy may be advantageous for assessing the deterioration of concrete structures due to ASR.

(3) The larger the reinforcement, the larger the residual expansion. On the other hand, the velocity of concrete core slightly decreased due to the residual expansion after sampling the core, and then it increased to more than the velocity of the test specimen because the silicate gel formed by the reaction may be filled in the cracks.

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CHARACTERISTICS AND SIMULATION OF CONCRETE CRACKS CAUSED BY AAR

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1. INTRODUCTION

Concrete cracks caused by alkali aggregate reaction (AAR) relate, more or less, to the mechanical bahavior and safety of structures. Many studies have been made on the subject and there are many reports on the typical crack pattern caused by AAR from qualitative aspect [1,2,3,4]. The present study has two main objects. One is to analyze quantitatively the effect of reinforcement and prestress on the crack characteristics due to AAR. The other is to simulate mathematically the crack occurence and its propagation process with three dimensional finite element method, which will be developed as a diagnosis technique of concrete cracking in the future.

2. CRACK ANALYSIS

Quantitative crack analysis was made for six kinds of reinforced concrete (RC) beam specimens and four kinds of prestressed concrete (PC) specimens. In RC specimens, kinds of concrete and the amount of compression reinforcement were varied. In PC specimens, amount of prestressing force was changed.

As shown in Fig. 1, RC beam specimen had rectangular cross section and its dimensions were 10x18x170 cm. Tensile reinforcement ratio was kept constant at p=1.7% (2D13), while three levels were selected for compression; i.e. p'=0% (none), 0.9% (2D10) and 1.7% (2D13). Two kinds of concrete, normal concrete (NC) and steel fiber reinforced concrete (SFRC), were used. The kinds of RC specimens are listed in Table 1. Yield strength of re-bars of D10 and D13 were 37.1 and 36.0 (kgf/mm^2), respectively. Strength and failure properties of the beams were already reported [2].

The size of the PC prism specimens was 10x10x40cm, which is shown in Fig. 2. Prestressing tendon was a high strength bar (SBPR 122/131) of 19mm in diameter. The bar was arranged at the center of the specimen through a plastic sheath (polyvinyl chloride) with 34mm outer diameter and was held in length by two steel plates of 40mm thick each at both ends of the specimen with nuts. One kind of normal concrete was used. Four levels of induced stresses were chosen, i.e. 0, 20, 40 and 80 kgf/cm^2 and the specimens were named AP-0, AP-20, AP-40 and AP-80, respectively.

Mix proportions of the concretes were about the same where w/c=0.5 and C:S:G = 1:2.1:2.7. For coarse aggrgate as a whole for RC specimens, alkali reactive crushed gravel (bronzite andesite: max. size 20mm) was used and one half of the whole amount was used for PC specimens. In the latter case, the other half of the coarse aggregates was replaced by non-reactive one, because

Table 1 Kinds of RC specimens

Name	Reinf. ratio and position			
	p(%)	d(cm)	p'(%)	d'(cm)
A−1	1.66	15.3	0	
A−2	1.66	15.3	0.93	2.5
A−3	1.66	15.3	1.66	2.7
SA−1	1.66	15.3	0	
SA−2	1.66	15.3	0.93	2.5
SA−3	1.66	15.3	1.66	2.7

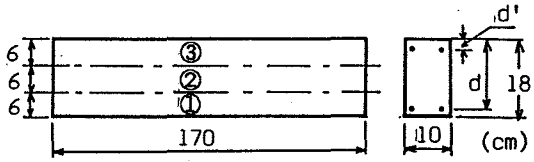

Fig. 1 Size and divisions of RC specimen

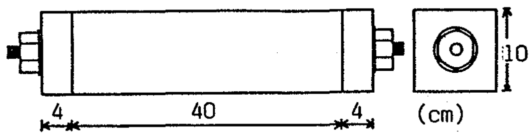

Fig. 2 Shape of PC specimen

the pessimum amount of this gravel was taken into consideration. Non reactive river sand was used. Sodium hydroxide was added to regulate the equivalent sodium oxide content so that it will be 2.3 % of cement content for RC specimens and 3.3 % for PC ones. In case of SFRC, indented steel fibers of 0.5x30 mm (aspect ratio: 60) were added as one volume percent of the concrete.

RC beam specimens were demoulded one day after casting and then stored in a room of 22°C and 80% R.H. for 14 days. Then, the room temperature was raised to 40°C and the specimens were kept under wet condition. PC prism specimens were also treated in the same manner as described above untill 14 days. Then, prescribed amount of prestresses was induced. At the age of 28 days, the stresses were re-induced and then, the room temperature was raised to be 40°C.

Cracks due to AAR were transcribed manually on tracing papers and crack maps were made for each of the opposite sides of every specimen; i.e. totally four maps were made for two specimens under each testing condition. The maps for the analysis were made at the ages of ca. 100 days for PC specimens and ca. 150 days for RC specimens. The maps for RC specimens were optically reduced to one thirds with a copying machine. Cracks on the maps were continuously traced with digitizer, which had an effective trace area of 380x260 mm and resolvability of 1.0 mm. Co-ordinates of the points on the cracks were read and put into a personal computor for every 1.0 mm. Total length of the cracks (CL) as well as the projection length on a beam axis (called as x-axis component, XL, hereafter) and that in a perpendicular direction to the beam axis (called as y-axis component, YL) were calculated for each crack map. The length of a line element for the calculation of components was 1.0 mm. For RC specimens, each map was divided into equally three regions in height as shown in the Fig. 1, and the crack values CL, XL and YL were also obtained for each division.

3. RESULTS AND DISCUSSIONS ON THE CRACK ANALYSIS

Digitized crack data put into the personal computor were reproduced again as crack maps with X-Y plotter. The reproduced maps are shown in Fig. 3 for RC and in Fig. 4 for PC specimens, where one side for each kind of the specimens is shown as an example.

Fig. 5 shows the average total crack length and crack density of one side of the RC specimens. The crack density was defined as the ratio of the total crack length to total area of the specimen. The crack length and density of

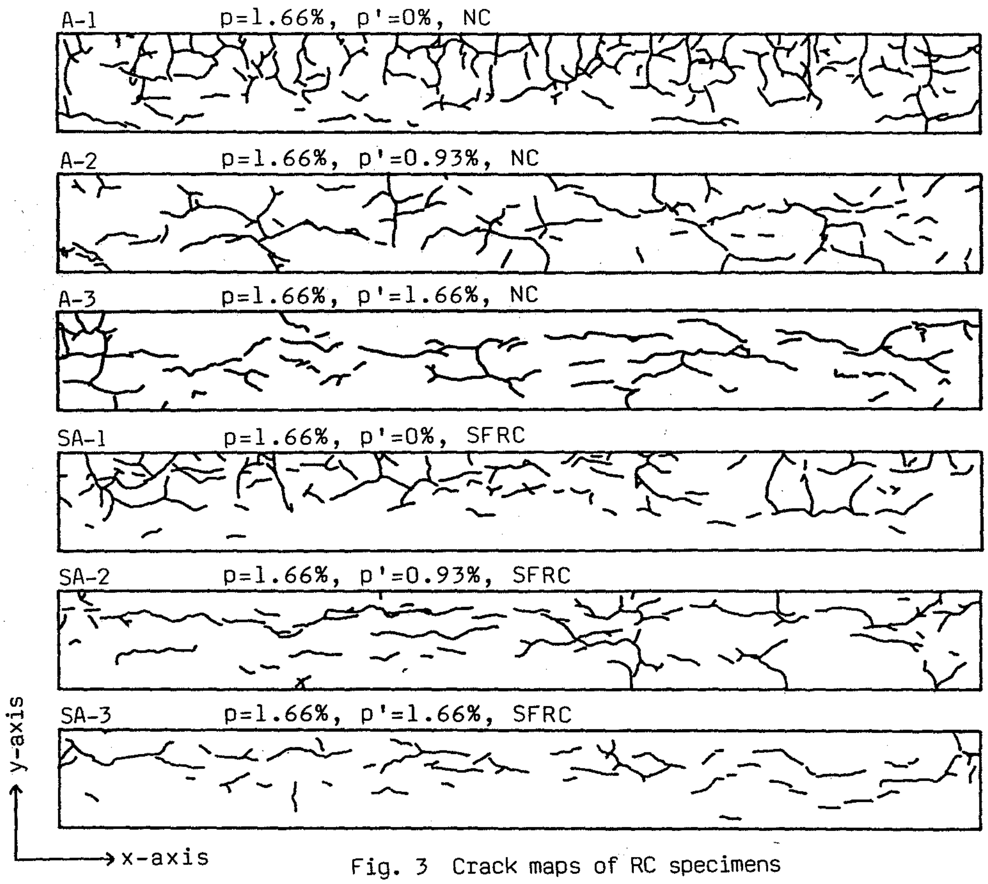

Fig. 3 Crack maps of RC specimens

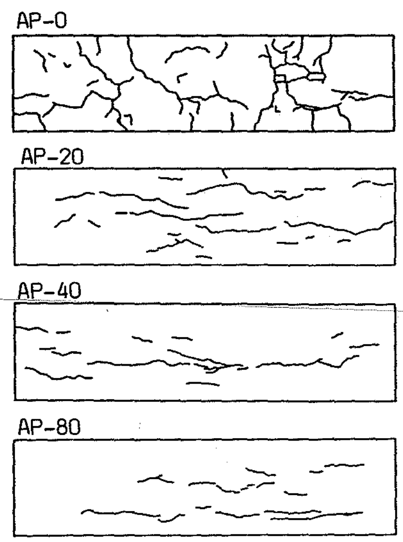

Fig. 4 Crack maps of PC specimens

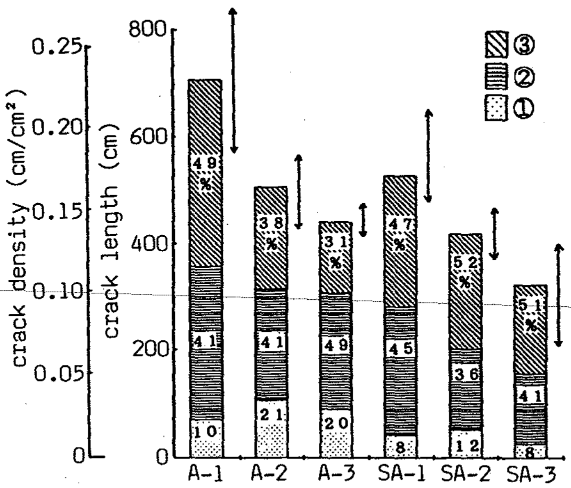

Fig. 5 Crack length and crack density (RC)

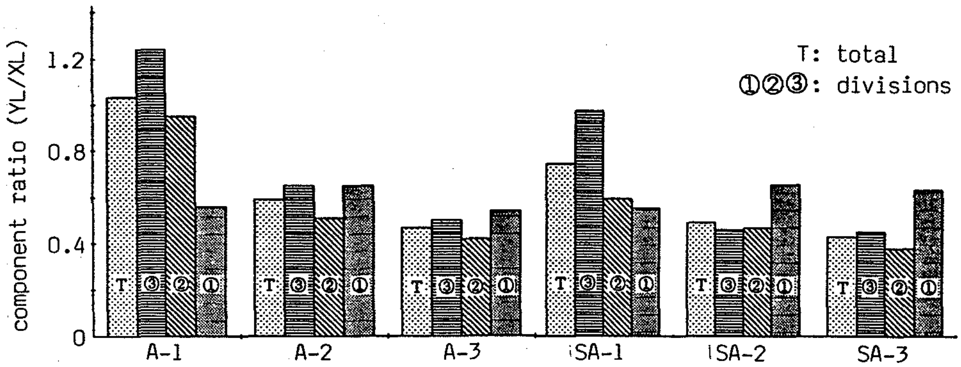

Fig. 6 Component ratio of cracking (RC)

the portion in each division defined in Fig. 1 are also shown in Fig. 5. In addition, the range of measured crack value is shown as an arrow on the right corner of the bar chart of total crack length. In case of RC beam specimens, total crack length decreased as compression reinforcement increased. For normal concrete without steel fibers, the crack lengths in region 1 and 2 changed only slightly when compression reinforcement increased. Only the length in region 3 decreased considerably. In case of SFRC, total crack length and that in each layer were smaller than that of corresponding normal concrete specimen. The maximum crack width was found to be 0.25 mm for A-1 specimen and it was less than 0.1 mm for other specimens.

Fig. 6 shows the ratio of the crack length of y-axis component to that of x-axis component (YL/XL: called as component ratio, hereafter). The component ratio of each of three divisions is also presented at the same time. When the compression reinforcement increased, the y-axis component as well as the component ratio decreased. For example, 1.03 for A-1 specimen and 0.45 for S-3 specimen. Because the reinforcement restrained the occurence of cracks perpendicular to the beam axis, cracks in the beam axis direction became dominant. In the division 3 and 2 of singly reinforced A-1 specimen, the component ratios were 1.24 and 0.95, respectively. The ratio 1.0 means that the cracks occurred quite at random. In the division 1, i.e. in the bottom division, the ratios were less than 0.6 and the cracks occurred mainly in the direction of the beam axis. Except the single reinforcement beams, the component ratio fell between 0.4 to 0.6 and the specimens were restrained as a whole by the reinforcement both in tension and compression. When the reinforcement ratio remained the same, the component ratio became smaller for SFRC. It is most likely that this was caused because the adoption of steel fibers raied the restrained effect of the reinforcement.

Total crack length (CL) and the crack density of PC specimens are shown in Fig. 7. In Fig. 8, the x- and y-component and the component ratio of PC specimens are illustrated. In case of PC specimens, the crack density decreased when the induced prestress increased when the amount of the initial prestress was 40 kgf/cm^2 and less. To the contrary, the total length of the crack, especially that of the x-axis component, increased when the initial prestress increased from 40 to 80 kgf/cm^2. Directions of cracks came to coincide gradually with those of prestressing as the amount of prestress increased. Fig. 9 shows the relation between the expansion and the ages from prestressing. In case of AP-0 and AP-20, specimens expanded with the ages in a hot and humid condition. In case of AP-40 and AP-80, contraction occurred at

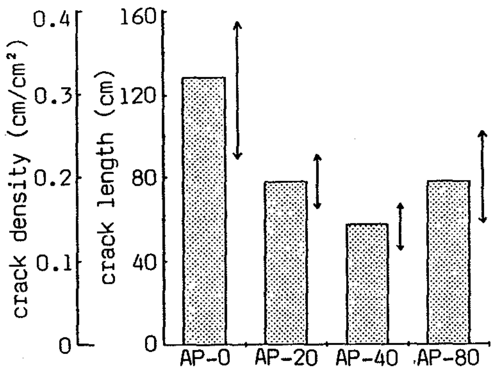

Fig. 7 Crack length and crack density (PC)

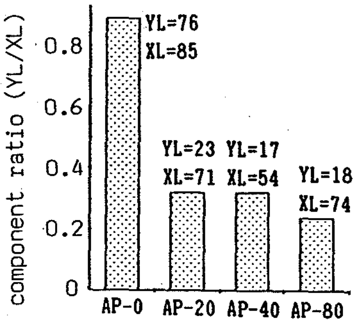

Fig. 8 Component ratio of cracking (PC)

first because creep deformation occurred due to the prestressing. After ca. 40 days, it turned to expand due to AAR. Almost of all the expansion was restrained by prestressing tendon and stress in the specimen raised in addition to the initial stress. This is probably because AP-80 had larger crack length than AP-40 had. The maximum crack width was less than 0.15 mm in any case.

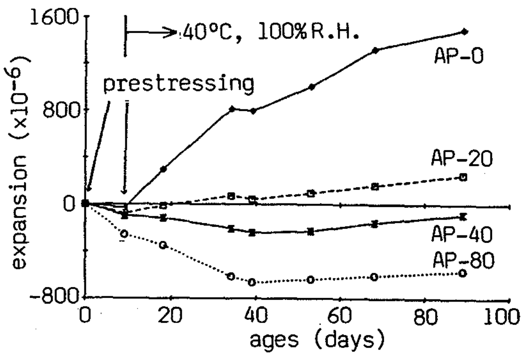

Fig. 9 Expansion of PC specimen

4. SIMULATION ANALYSIS OF CRACKING

Crack initiation and propagation process in concrete due to AAR expansion were simulated with three dimensional FEM (finite element method) analysis [5]. Concrete was modeled as the composition of 64 (4x4x4) elements as shown in Fig. 10. The size of an each element was 5 cm cubic, i.e. the model was 20x20x20 cm. An expansive element, a shaded one, was assumed to be located in the inside of the model concrete. Cracked element was considered to have an orthotropically heterogeneous property in the direction of the cracked surface. In the consitutive equation for the direction in perpendicular to the cracking, tension softening was taken into account. Crack band model was adopted and the tension softening curve was assumed to be straight as shown in the stress strain curve of Fig. 11, where assumed tensile strength and Young's modulus were 30 and $3x10^5$ kgf/cm^2, Poisson's ratio was 0.16 and fracture energy G_f was 0.1 kg/cm. The maximum principal strain theory was adopted as a fracture criteria. The expansive element was considered to increase its volume uniformly and continuously, however, in the calculation, the expansion was given stepwise with the increment of $50x10^{-6}$. Cracking process on the surface nearest to the expansive element (the front surface) was shown in Fig. 12, where total expansive strain e at each stage is given in the figure. The crack was thought to initiate at the strain of $100x10^{-6}$ and it did not necessary to agree with the maximum stress point. At the last stage in Fig. 12, the maximum surface strain was ca. $300x10^{-6}$. Though it is insufficient in the size of the model and the extent of the calculation, it would be valuable to develop the simulation technique in analyzing the nature of cracking.

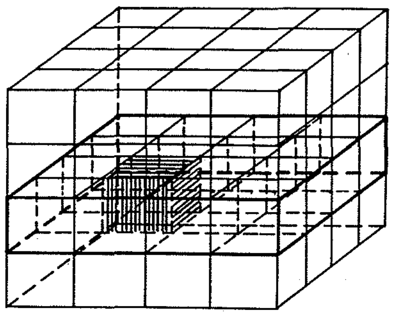

Fig. 10 Concrete model and expansive element

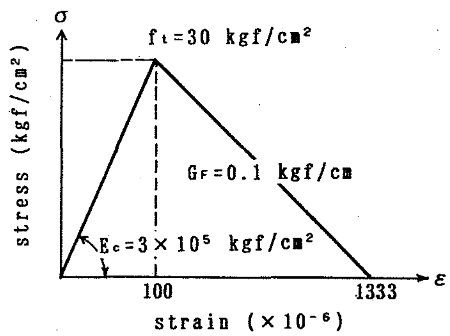

Fig. 11 Tensile stress strain curve

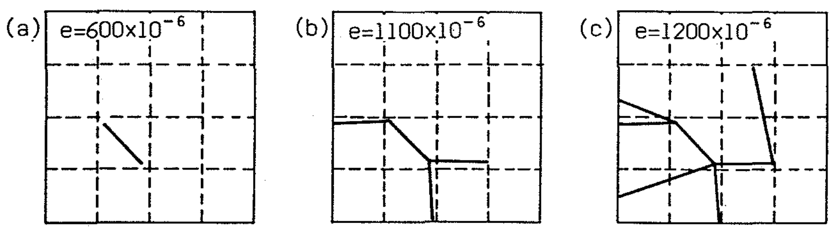

Fig. 12 Cracking process with simulation analysis

5. CONCLUSIONS

The character of concrete cracks caused by AAR was analyzed quantitatively, especially in relation with reinforcement and prestressing. Simulation analysis of crack pattern was also made. Principal conclusions derived are as follows;

(1) In case of RC beam specimens, total crack length and crack component perpendicular to the beam axis decreased when compression reinforcement increased and cracks in the direction of the axis became dominant. Addition of steel fiber also made crack length shorter.

(2) In case of PC specimens, cracks in the stressing direction became dominant gradually when the amount of prestress increased. Crack density decreased with the increase of prestress within 40 kg/cm^2. It increased, however, when the prestress became 80 kg/cm^2.

(3) Cracking process could be simulated, though insufficient, where tension softening was taken into consideration.

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RECENT DIAGNOSIS AND REPAIR TECHNIQUES FOR DAMAGED CONCRETE STRUCTURE BY ASR —A GUIDELINE FOR PUBLIC WORKS STRUCTURE—

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ABSTRACT

A concrete structure damaged by the alkali silica reaction become less durable, because rain water and carbon dioxide in the air penetrate into the cracks which results in the neutralization of the surrounding concrete and erosion of reinforcing rods. For this reason, a study of development of repairing materials for concrete structures damaged by the alkali-silica reaction was conducted, and based on the results "Guideline for repairing concrete structure damaged by ASR(draft)" was prepared. The guideline (draft) includes diagnosis, repair design, repairing materials, and follow-up survey. The major contents of the repair guideline are described.

INTRODUCTION

Concrete structures damaged by the alkali-silica reaction spread widely in the country. Japan island were mainly formed from volcanic activities and upheaval from the sea bottom. Volcanic rocks and chert with characteristic properties distribute widely, which are knows as aggregates vulnerable to the alkali-silica reaction. Meteorologically Japan has high temperature and high humid summer over the country, and much snow and rainy winter along the Japanese Sea coast. As a result, in concrete structures which are always wet, the reaction proceeds rather quickly. Up to the present (as of 1988) , more than 40 concrete structures have been repaired. The details of the structures , the time of repair,and repair methods are summarized in Table 1.

Table 1 Repaired Concrete structures

TYPE OF STRUCTURE	number
bridge foot,abbut	36
concrete wall	7
guarder	2
pc beam (girder)	1
concrete box	2
REPAIR WORK	
1983	2
1984	5
1985	14
1986	9
1987	10
INJECTION(GROUT) METHOD	
epoxy	28
sealant	2
prepacked concrete	1
non	11
SURFACE COATING MATERIALS	
polyurethane	14
epoxy	6
polybutadien	6
acrylurethane	7
polymer cement	3
cement mortar	2
silan	1
silicone	2

REPAIR GUIDELINE

"Guideline for repairing concrete structure damaged by ASR(draft)" was prepared. These guidelines are applied to repair and reinforcement design/implementation of civil engineering concrete structures damaged by the alkali-silica reaction are composed of as in Figure 1. The essential parts are briefly described.

START
DIAGNOSIS AND CLASSIFICATION OF CRACK
  periodical inspection
   appearance inspection
  detailed inspection
   crack propagation
    concrete surface strength
   non destructive investigation
    infrared thermography
    steel corrosion
   concrete core sampling
    expansion test/ aggregate reactive test
    gel analysis
   load bearing test of concrete members
DESCISION FOR REPARING OR NOT
REPAIR DESIGN
  patching and injection
  coating
REPAIR WORK, WORK CONTROL AND INSPECTION
FOLLOW UP INSPECTION
END

Figure 1 Flow of diasgnosis and repair

DIAGNOSIS AND CLASSIFICATION OF CRACK

In a preliminary investigation(Table 2), the degree of advance of the cracking of structures was divided into two groups, cracking progress division A where cracking is in progress, and cracking progress division B where the progress of cracking has discontinued . In addition, in consideration of repair costs, 2 classes of 0.2-5.0mm crack width and more than 5.0m crack with set. In a general environment, when the width of a crack is less than 0.2mm the structure is not subjected to repair.

With respect to repair techniques, most cases are made by a combination of crack injection, water proofing by coating of the concrete surface. Depending on the purpose and the surrounding environment of a structure only coating may be applied.

REPAIR DESIGN

Based on past results and findings, the basic requirements for inhibiting aggregate reaction consist chiefly of : Drying the concrete, blocking off supply of moisture, a substance essential to reaction, and providing corrosion protection for the reinforcing bars.

The framework of the present repair work(Figure 3), consisted of removing all loose sections on the concrete surface and performing patching for the sections where concrete had spoiled off, injecting resin into the cracks, sealing off the supply

Table 2 ASR Judgment from Concrete Core

test		items
concrete core		gel observed. reaction rim observed.
core expansion test		expansion ratio > 0.05%/3months
aggregate test	chemical test	reactive or non reactive
accelerated test		gel observed
gel analysis		silica gel analysed

of moisture by coating the concrete.

Table 3 Crack Classification and Repair Design

	crack	crack width (mm)	crack sealing		surface coating
			injection		
A	crack progressed	0.2~0.5	epoxy grout		soft, high build type coat
		>0.5		sealant	soft type coat
B	crack stabled	0.2~0.5	epoxy grout		soft or hard type grout
		>0.5		sealant polymer cement	soft type coat hard type coat

PATCHING AND CRACK INJECTION

Patching has three objectives, one being, to eliminate the danger of concrete fragments falling off. this is achieved by removing those parts of the concrete surface which have come loose and turned brittle by chipping them off. The second objective is to obtain smooth surface. Through surface treatment performed with a disc grinder, or sand bluster, a firm concrete substrate for crack injection and to ensure that the coating applied to the concrete will not spoil off. The third objective is to obtain a level surface configuration in order to restore the overall appearance of the structure to the original one, by patching those irregular sections where concrete is missing.

There is a large variety of methods for injecting resin into cracks(Table 4). Resin materials used for crack injection include the standard hard type which is designed for integration with the concrete, and the soft (flexible) type designed to relax crack expansion to suppress crack generation in other parts of the concrete. In consideration of the fact that ASR was still in progress, the soft type which has an elongation capacity was used. In the present repair work, epoxy resin to have a rupture elongation rate of 100%, or more, was selected as the resin material for injection: injection for cracks measuring 0.3mm, or more.

COATING AND MATERIALS

A coat placed on a concrete surface can control as not to be intruded on surface from factors to progress the alkali-silica reaction such as water supplied from the outside, and thus it plays an important role in controlling the alkali-silica reaction.

Coats are required to be those which have permanent and satisfactory adhesiveness and the ability to prevent and water penetration from the outside(Table 5). When cracking is in a fairly advanced stage, coats have to be those which expands with the progress of cracking and prevent water penetration through cracks.

In coating, concrete which has been seriously deteriorated and resultantly raised to the surface has to be removed, because it hinders adhesion of the coating material. At this time, any corroded steel parts have to be subject to corrosion-proofing treatment, patching and coated.

Table 4 Specification of Injection Grout

	epoxy	soft type epoxy	high soft type epoxy	polymer cement	sealant
viscosity (cps)	1000	1000	1000	10000	
pot life time (hour)	16	16	24	16	24
crack adaptability (%)	—	50	100	—	800
harden shrinkess (%)	0.1	0.1	0.1	0.1	0.1
adhesiveness to concrete (kg/cm²)	60	60	60	60	60

Table 5 Specification of Coating Materials

	A	B		
	soft, high build type	soft type	hard type	
crack adaptability	100% or more	50% or more		tensile elongation
waterproofing	20mℓ/m²·day or less	30mℓ/m²·day or less	20mℓ/m²·day or less	imperme-ability
bond strength	10kg/cm² or more	10kg/cm² or more	10kg/cm² or more	bond with concrete
alkali resistant	30days	30days	30days	$Ca(OH)_2$
weathering	300hours	300hours	300hours	sun-shime accelerating weathering

Waterproofing coatings used for coating the concrete surface can be divided into three groups, coating systems that have adaptability to cracks, coating systems that perfectly seal off moisture and water vapor, and those that seal off water but permit water vapors to permeate.

In the present repair work such performances as, having excellent adaptability to cracks, and being capable of sealing off intrusion of water from the outside were set as significant assessment items, in consideration of the progressive state of ASR. Further, a coating system capable of causing the moisture contained inside the concrete to escape outside and disperse was also assessed, though it had only a small number of application results.

The following coating systems were selected for example. A thick-film type flexible epoxy resin paint of the full-seal-off type, which features adaptability to cracks, an elasticity-imparting type polymer cement-based paint, which, in addition to having adaptability to cracks, causes the moisture inside concrete to escape outside and disperse, was selected. The standard film thickness for both coating systems was set to 500-1,000(um).

Table 6 Examples of Coating Systems

coat	material	No. of coats	film thickness (μm)	volume (kg/m^2)	appli-cation	interval (20℃)
pre treat-ment	surface preparation with power tool					
leveling	epoxy primer	1	–	0.15/coat	brush	16hr-7days
	epoxy putty	1	–	0.40/coat	spatula	16hr-7days
middle	high build type epoxy	2	500	0.40/coat	spatula	16hr-7days
top	polyurethane	2		0.12/coat	brush	8hr-3days

coat	material	No. of coats	film thickness (μm)	volume (kg/m^2)	appli-cation	interval (20℃)
pre trea tment	surface preparation with powerfool					
leveling	putty	1	–	0.12/coat	brush	1hr-7days
top	polymer cement	3	1000	0.70/coat	spatula	16hr-7days

In order to obtain full repair effects, thorough control was effected for each process during repair work.

In conducting crack repair, thoroughgoing construction work was carried out for each process, working in conformity to the control items and control standards for crack injection. Following the completion of crack injection work, cores were sampled to confirm the state of injection and crack depth. In coating concrete, thorough on going work was carried out for each process, working in conformity to the control items and standards. On completion of coating work, the film thickness was measured to confirm the formation of a film thickness as specified in the coating specifications.

FOLLOW UP INSPECTION

The follow-up inspection consist of;easurement of the crack width and crack length conducted on an appearance investigation bases, follow-up measurements of the crack widths using such means as a contact gauge, external observation of the film (blistering, cracking, flaking), and investigation of the bonding strength of the film.

CONCLUSION

In the present repair work, the waterproofing measures taken to inhibit ASR for the concrete by supply of water from outside is most promisible system. however,the effect and control of the internal moisture on the reaction is not clarified. At present, there is no alternative but to rely on the effects gained by applying elasticity-imparting polymer cement based paint to the cement, thereby causing the internal moisture to escape and disperse.

In Japan, it has not been long since the ASR phenomenon has been discovered, and examples of repair works are still a few. Meanwhile, research on repair technology is still progressed day after day, and repair methods have not been established perfectly although we have new reccomendarion "guideline for repairing concrete structure damaged by ASR(draft). Therefore, we make it common practice to conduct a follow-up investigation after each repair work has been completed, not only to confirm the repair effects but also together practical data for use in developing repair methods of even higher efficiency.

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Photograph Repaired Concrete Structure

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STUDY ON THE REDUCING REACTIVITY RATE OF CONCRETE USED WITH REPAIRING MATERIALS IN LABORATORY AND FIELD

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ABSTRACT

The lack of established repairing techniques for deteriorated concrete structures due to ASR urgently requires the establishment of such techniques. For this purpose, Public Works Research Institute, Ministry of Construction, and Hanshin Expressway Public Corporation conducted a joint research for development of injection materials and coating materials.

This paper reports results of promotion testing in laboratory and exposure testing in the field for repairing materials selected by a performance test of various materials, for the purpose of elucidation of their repair effect.

INTRODUCTION

When a concrete structure deteriorated due to alkali-silica reaction (ASR) is left as it is, then rain water and carbon dioxide penetrating into cracks neutralize the surrounding concrete, resulting in corrosion of the reinforcing bar. Therefore, development of repairing techniques of concrete structures deteriorated due to ASR is urgently requested. The authors performed testing of various injection materials and coating materials as repairing materials, and promotion testing in laboratory and field exposure testing using concrete specimens,in order to identify the effect of these repairing materials.

For repairing materials selected through performance testing of various materials, their repair effect is identified through promotion testing in laboratory and field exposure testing.

ACCELERATED TEST METHOD IN LABORATORY

Coating materials selected through sealing test, crack adaptability test and crack control test were applied to concrete specimens as coating materials for the promotion testing in laboratory.

Sixty 150 x 150 x 500 mm specimens of reinforced concrete with W/C=50% were tested, and 2 types of andesites and 1 type of chart which are reactive to coarse aggregates were adopted. Alkali was adjusted by sodium hydroxide to the alkali level of 8kg/m^3 in R_2O conversion. The specimens were subjected to promotion curing at 40°C and approximately 100% humidity for 6 months for cracking, and then coated in the specification shown in Table 1 for the promoted deterioration testing in the same condition with that of the promoted curing.

Result of observation of appearance of the cost 240 days after the promotion testing was :epoxy resin (flexible type) coated specimens (200um) and acryl resin coated specimens (100um) of coat thickness was set to be thin because the comparison got swollen, while specimens coated with polyurethane as silane (30um) yielded gel.
However, all other specimens which were coated rather thickly didnot have any defects such as swelling.

Table 1 Coating specifications

coating materials	thickness (μm)
① epoxy resin (flexible type)	200
② polyurethane (flexible type)	200
③ epoxy resin	500
④ rubber (flexible type)	500
⑤ cloth mixed epoxy resin (flexible type)	1,200
⑥ cloth mixed rubber (flexible type)	1,200
⑦ glass flake mixed epoxy resin	500
⑧ glass flake mixed vinylester resin	500
⑨ polymer cement (flexible type)	500
⑩ polymer cement mortar	15,000
⑪ silane-polyurethane	30
⑫ acryl resin	100

ACCELERATED TEST METHOD OF THE LARGE SPECIMENS

Large reinforced concrete specimens repaired by either injection or coating were subjected to promoted deterioration, and the change in deformation of reinforcing bar with time was measured.

Eleven specimens being 50 x 50 x 140 cm beams were made at 0.5% tension reinforement ratio and 0.2% stirrup in accordance with the placement of bars shown in Figure 1. specified concrete strength was set at 210kgf/cm^2 and mix proportion shown in Table 2 was set. At that time, andesite which is reactive to coarse aggregate was mixed at the rate of 50%. In addition, alkali was adjusted with sodium hydroxide to 8kg/m^3 in R_2O conversion unit.

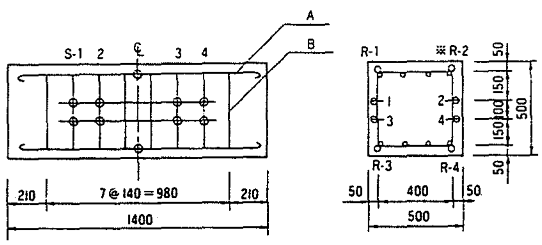

Fig.1 Drawing for re-bar arrangement

Table 2 Table of mix proportion

maximum size of coarse aggregate (mm)	slump (cm)	air content (%)	water cement ratio (%)	sand-coarse aggregate ratio, sand percentage (%)	quantity of material per unit volume of concrete (kg/m^3)				
					water	cement	fine aggregate	coarse aggregate	sodium hydroxide
20	10	2.0	60	48	180	300	877	947	7.81

These specimens were subjected to ASR promotion curing a condition of 40^{o}C and approximately 100% humidity for about 5 months, and then repair works were carried out in accordance with the specification shown in Table 3. After that, the specimens were placed again in the same environment with one before repaired to identify the repair effect by measuring the elongation of the specimens with contact gauge. The injection was conducted only for those with crack of more than 0.2mm width. The deformation of the reinforcing bar of specimen at the time of repair was approximately 1,000um for the main

Table 3 Repair specifications

	injection materials	coating materials
①	epoxy resin (hard type)	polyurethane (100μm)
②	epoxy resin (flexible type)	polyurethane (100μm)
③	polyurethane resin	polyurethane (100μm)
④	polymer cement (acryl resin)	polyurethane (100μm)
⑤	slag (colloidal type)	polyurethane (100μm)
⑥	—	rubber (flexible type)
⑦	—	cloth mixed epoxy resin (flexible type)
⑧	—	epoxy resin (flexible type)
⑨	—	glass flake mixed vinylester resin
⑩	—	polymer cement (flexible type)
⑪	—	—

reinforcing bar, and approximately 1,200 - 1,700um for the stirrup, as shown in Figure 2.

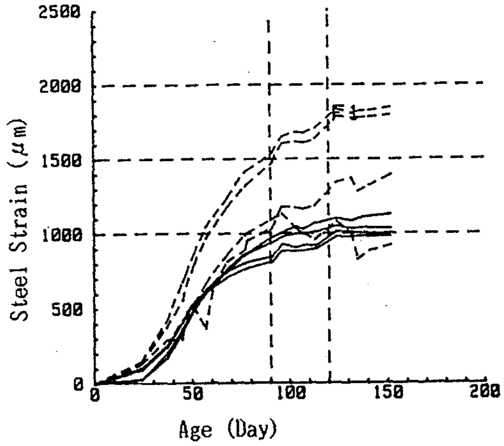

Fig.2 Steel strain before the repair

The elongation of specimens after being repaired was determined by using a contact gauge with the measuring precision of 1/1,000nm. Figure 3 shows the position where the elongation of specimens was measured. The elongation values are those at the front towards the longitudinal direction for sections from 1 to 4 of specimen (1,400 x 500mm), those at the front towards the transverse direction for 5 and 6, and those at the side for 7. A comparison of elongations of specimens at different positions indicate that the elongation is greater in the longitudinal direction where more reinforcing bars are placed on the whole than in the lateral direction where the volume of reinforcing bars is smaller. A tendency is recognized that the elongation gets greater at the end of the beam (section 5) and at the side face (section 7) than at the center of the beam (section 6) in the lateral direction. This is assumed because the restrict due to concrete as

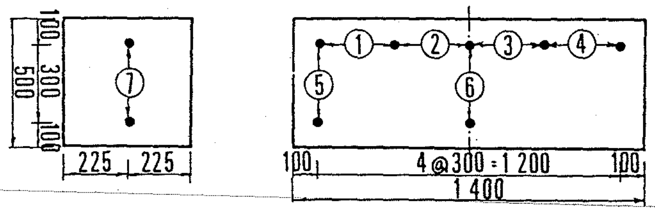

Fig.3 Elongation measuring point

well as restriction due to the elongated reinforcing bar function is more effective at the center of the beam than at the beam ends.

The elongation restriction effect at the time of 250 days is compared for each repairing materials with respect to the elongation between section 5 and section 7 where the restriction due to reinforcing bar is less; as to injection materials specimens into which epoxy resin (hard type) (NO.1) and epoxy resin (flexible type) (NO.2) were injected has less elongation. On the other hand, as to coating materials, relatively hard materials such as cloth mixed epoxy resin (flexible type) (NO.7), vinyl ester resin (NO.9) and polymer cement (NO.10) showed the restriction effect.

FIELD EXPOSURE TEST METHOD FOR REPAIR EFFECT

In order to know the long-term repair effect of injection materials and coating materials, concrete specimens were manufactured for field exposure testing.

The concrete specimens are 25cm x 25cm x 100cm reinforced concrete square pillars with W/C=50%, in which the alkali quantity was set at 3 levels of 1.5, 2.0, and 2.3% (8kg/m^3)in R_2O conversion unit with sodium hydroxide. Coarse aggregates are mixtures of two kinds of andesite and; a 3 kinds of reactive aggregates at 50%. The specimens were subjected to promotion curing at 40°C and approximately 100% humidity for about 6 months. The top 70cm section excluding the bottom 30cm of the above mentioned cracked reference specimens

Table 4 Field exposure test

aggligate	A								B			C		
alkali(R_2O) (%)	1.5			2.0		2.3			2.3			2.3		
injection materials														
① epoxy resin (hard type)	○	○		○		○			○			○		
② epoxy resin (flexible type1)	○	○		○		○			○			○		
③ epoxy resin (flexible type2)	○	○		○		○			○			○		
④ epoxy resin (flexible type3)	○	○		○		○			○			○		
⑤ polyurethane	○	○		○		○			○			○		
⑥ polymer cement (acryl resin)	○	○		○		○			○			○		
⑦ polymer cement (epoxy resin)	○	○		○					○			○		
⑧ slag(colloidal type)	○	○		○		○			○			○		
⑨ polisulfide	○	○		○					○			○		
coating materials (µm)														
① epoxy resin (flexible type) 200							○			○			○	
② polyurethane (flexible type) 200										○			○	
③ epoxy resin 500	○					○	○		○	○		○	○	
④ rubber (flexible type) 500							○			○			○	
⑤ cloth mixed epoxy resin (flexible type) 1200							○			○			○	
⑥ cloth mixed rubber (flexible type) 1200							○			○			○	
⑦ glass flake mixed epoxy resin 500							○							
⑧ glass flake mixed vinylester resin 500							○			○			○	
⑨ polymer cement (flexible type) 500							○			○			○	
⑩ fluorine resin 200										○			○	
⑪ silane 30							○							
⑫ acryl resin 100							○			○			○	
no repair			○		○			○			○			○

was repaired in accordance with the specification shown in Table 4. The specimens were buried perpendicularly under the ground surface at the depth of 30cm in consideration of uncoated bridge piers, and the exposure was initiated in Oct. 1987.

Summarizes results of appearance observation of the specimens after 1 year exposure. Cracks developed and a part of the coat of the silane-polyurethane showed cracks, just other specifications had no change.

SEASHORE EXPOSURE TEST FOR ASR PREVENTION

Concrete specimens were manufactured for exposure at the seaside for the purpose of understanding the effect of sodium from the outside (sea breeze and; splash of sea water) on ASR and the ASR prevention effect of coating materials.

The concrete specimens are 15cm x 15cm x 100cm reinforced concrete square pillars, and they were adjusted to W/C=50%, slump of 8cm, air volume of 1.5%, and the alkali quantity of a $3kg/m^3$ and $5kg/m^3$ in R_2O conversion unit with sodium hydroxide. Coarse aggregate was made by mixing reactive andesite at 50%. Cement was low alkali portland cement and portland blast-furnace slag cement(B type).

As to ASR preventive measure, 4 specimens of silane-poly urethane,high density mortar(8mm), epoxy resin and acryl resin were applied. The details of the specimens are shown in Table 5.

Table 5 Seashore exposure test

	surface treatment				
	no-coating	silane-polyurethane	hight density mortar	epoxy resin coating	acryl resin coating
low alkali portland cement	2	2	2	2	2
portland blast-furnace slag cement (B type)	2	2	2	2	2
alkali gross weight $3\ kg/m^3$	2	2	2	2	2
alkali gross weight $5\ kg/m^3$	22	2	2	2	2

Exposure of the specimens was commenced along the shoreline in Noto Peninsulas in Ishikawa Prefecture in Nov. 1986.

Observation of the appearance of the specimens 2 years after the start of exposure suggests that no macroscopic cracking wasseen even for the specimens without the preventive measure of coating. Of the specimens coated with organic compounds such as epoxy resin and acryl resin did not develop cracking, and their coat layer was very sound. However, the coat exposed to violent splash of sand was partly exhausted.Silane-polyurethane developed fine cracks in the edge parts.

SUMMARY

Various material tests in laboratory, promotion tests in the laboratory and exposure tests in the field for injection materials and coating materials for repair, indicate the following.

To cut supply of water into the inner part of concrete is a key for ASR repair works. For this purpose, appropriate materials are injected or filled depending on the width of existing cracks, and then the outside of the structure is coated with materials with excellent water insulation. However, when aggregate in the structure is in progress of the reaction, the effect of cutting water supply from the outside does not manifest itself so quickly. As a result, the water left in the inner concrete may slowly promote the reaction. Therefore, materials for repair require elongation for the remaining expansive power, and strong adhesiveness of concrete. On the other hand, for a structure in which the reaction is almost over, it is important to prevent corrosion of reinforcing rods due to neutralization of concrete caused by water entering cracks and carbon dioxide in the air. For this reason,materials for repair do not necessarily require elongation, but strong adhesiveness to concrete is called for.

Results of the bearing test for reinforced concrete already cracked due to ASR show that cracking does not result in a decrease in durability and so reinforcement is practically unnecessary at this stage. When reinforcement is made, implementation in accordance with that for general reinforced concrete structures is good enough.

AFTERWARD

As a measure to prevent the ASR reaction, injection and coating are very effective in repairing structures already cracked due to ASR. Thus, the authors have proposed Repair/Reinforcement Guidelines (draft) based on the results from various material performance tests, promotion tests, and field exposure tests for injection materials for repair and coating materials which are for concrete structures already cracked due to ASR.

Since the effect of these repairing materials is that only for less than 2 years, the long-term repair effect is not yet known. Therefore, follow-up study will be conducted for a long time with respect to the field exposure tests to elucidate their repair effect.

ACKNOWLEDGMENT

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8th Internationl Conference
on Alkali-Aggregate Reaction

REPAIR OF CONCRETE STRUCTURES DAMAGED BY ALKALI-SILICA REACTIONS AND ITS EFFECTS

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1. INTRODUCTION

In Japan, cracks in concrete have been repaired by injection of epoxy resin or cement mortar into the cracks and cracked concrete slub of bridge has been repaired by adhering steel plate to the bottom surface of the slub by epoxy resin.

This paper reports two test results to confirm the efficiency of such repair methods to be applicable to concrete structures damaged by ASR as well.

Reinforced concrete model beams were used for the first test and reinforced concrete continuous slubs with 2 spans were used for the second test. These ASR models were stored in the room of 40℃ and 100%RH. Strain measurement and crack observation were done periodically. When the expansion reached a certain level and cracks developed widely, the cracks of the beams were repaired by injecting epoxy resin and the slubs were repaired by the steel plate adhering method. After repaired, these models were stored again in the same room. When the expansion of these models almost converged, they were loaded up to failure.

2. EFFECT OF REPAIR BY EPOXY RESIN INJECTION

2.1 Test

2.1.1 Models Eight reinforced concrete beams of 500×500×3150mm were made. Their tensile steel ratio was 0.516% and stirrup ratio was 0.217%.

Figure 1 shows the beams and the cross section. Among 8 beams, 4 ASR beams were repaired by 2 different types of epoxy resin and 2 ASR beams were left unrepaired. The left of 2 beams were normal concrete beams (Table 1).

Table 2 shows the mix proportion of the concrete. Bronzite andesite was used as the reactive aggregate. The sand used was non-reactive. The alkali content was 8.0 kg/m³ and the mix rate of reactive aggregate (GR) to the total aggregate (G) was 50% which is close to the pessimum.

Table 3 shows the property of two different types of epoxy resin injected. The characteristic of these epoxy resin is their ability of large extensibility which is expected to play a role of absorbing strain caused by successive ASR.

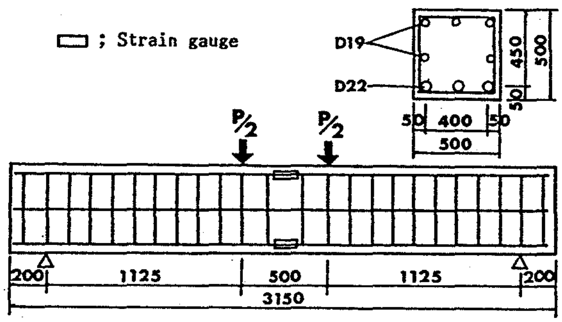

Figure 1 Cross-section of reinforced concrete beam

Table 1 Test beams

Beam NO.	Concrete	Injected epoxy resin	Storing condition
A-1	ASR	Super flexible type	①→②
A-2			①
A-3		Flexible type	①→②
A-4			①
A-5		no repair	①→②
A-6			①
N-1	Normal	no repair	①
N-2			①→②

① In the room of RH 100% and 40℃
② In the atomosphere
Change of storing condition was done at the age of 213 days

Table 2 Mix proportion of the concrete

	Gmax (mm)	Slump (cm)	Air (%)	w/c (%)	s/a (%)	W (kg)	C (kg)	S (kg)	G(kg) GR	G(kg) GN
ASR concrete	20	9.5	3.5	50	44	176	352	778	509	509
Normal concrete	20	9.0	3.6	50	44	176	352	778	—	1018

Table 3 Property of the epoxy resin used

Property	Test values	
	Super flexible type	Flexible type
Specific gravity	1.18	1.15
Tensile strength (kgf/cm²)	98	124
Extensibility (%)	190	130
Tensile shearing strength (kgf/cm²)	80	105

2.1.2 Storing and repair The beams were cured for 13 days at 20℃ and 80%RH and then stored in the room of 40℃ and 100%RH for 200 days. Thereafter, the beams A-1, A-3, A-5 and N-2 were exposed to the outdoor and the others were kept in the same room until loading test. At the age of 289 days, cracks of the beams A-1, A-2, A-3 and A-4 were repaired by the epoxy resin.

2.1.3 Loading test The beams were loaded statically up to failure by two-point loading with shear span to effective depth ratio of 2.5. Deflection and steel strain of the beams were measured at the center of span.

2.2 Results

2.2.1 Strength of the concrete Table 4 shows the compressive strength, Young's modulus and bending strength measured at the age of 407 days. The compressive strength and Young's modulus were measured by cylinder specimens of $\phi 10\times20$cm and bending strength were measured by prism specimens of $10\times10\times40$cm.

Table 4 Strength of the concrete

	Compressive strength (kgf/cm²)	Bending strength (kgf/cm²)	Young's modulus ($\times10^4$ kgf/cm²)
ASR	348	35	21.1
Normal	589	59	40.8