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ASSESSMENT AND REPAIR OF DAMAGED CONCRETE STRUCTURE

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ABSTRUCT

Since a crack in a concrete structure was detected to be due to alkali silica reaction (ASR) in 1982 in Japan, various investigations have been carried out.

This paper presents the results of the investigations with regards to:

(1) damaged concrete structures due to ASR,
(2) the level of the damages such as cracking characteristic, loss of compressive strength and elastic modulus, expansion of concrete and corrosion of steel,
(3) assessment of the damages by loading test on concrete models and existing structures,
(4) various repair methods such as coating of concrete, injection of resin or mortar into cracks and adhesion of steel plate and
(5) effect of these repair methods.

From these investigations, it was found that the damage of concrete structures in Japan due to ASR had not yet reached a significant level at the present stage. However, appropriate repair techniques are needed to safeguard damaged structures from further deterioration.

1. DAMAGED CONCRETE STRUCTURES DUE TO ASR

Appearance of ASR in Japan is considered to be due to several causes, such as the use of reactive mountain gravel or crushed stone, increase of alkali content in cement, increase of cement content in concrete and use of sea dredged sand.

The damage of a concrete structure due to ASR depends upon many factors such as the characteristic and the amount of reactive aggregate used, alkali content of the concrete, ambient temperature and moisture content, size and shape of the structure,degree of restraint of the structural member, existence of initial cracks etc.

Figure 1 shows 47 locations in which ASR was detected by the author in the last 5 years. The investigation was focused on the southwest area of Japan. ASR has been also detected in the northeast area of Japan. The damaged structures include various types of structures such as pier and abutment of road bridge, sea defence, retaining wall, box culvert, tunnel lining, water tunk,building etc.

Table 1 shows the construction time of these structures. According to this investigation, ASR in Japan seems to begin suddenly from the late 1960th. This result looks reflecting the following facts begun at that time.

-Wide use of crushed stone and sea dredged sand due to the lack of river stone and river sand,
-Increase of alkali content in cement due to the change of the production process and
-Increase of alkali content in concrete due to increase of the cement content as a

result of adoption of pump placing.

Among 47 locations, bronzite andesite was found at 34 locations, slate and chert at 6 locations, tuff at 5 locations and opal at 2 locations as the reactive aggregate. Bronzite andesite is the most popular reactive aggregate in Japan.

Figure 2 to 8 are examples of damaged concrete structures due to ASR. Figure 2 is a typical example of ASR cracks in reinforced concrete beam where the main cracks are developing in the horizontal direction. Figure 3 shows cracking in a bridge column where vertical cracks are developing. Figure 4 shows cracking at the

Table 1 Construction time of ASR damaged concrete structures

Construction time	No. of locations
1966～1968	2
1969～1973	25
1974～1978	11
1979～1983	4
unknown	5
Total	47

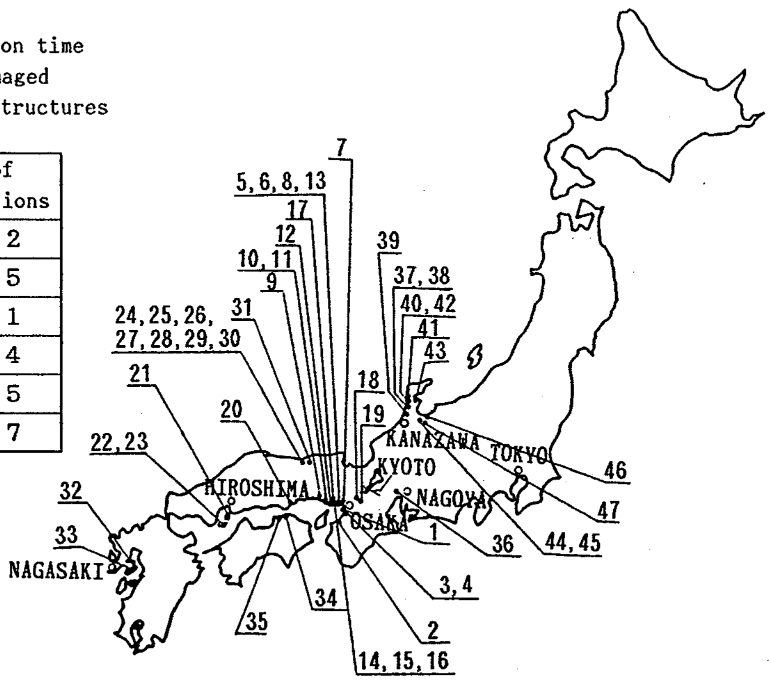

Figure 1 Locations of damaged structures due to ASR

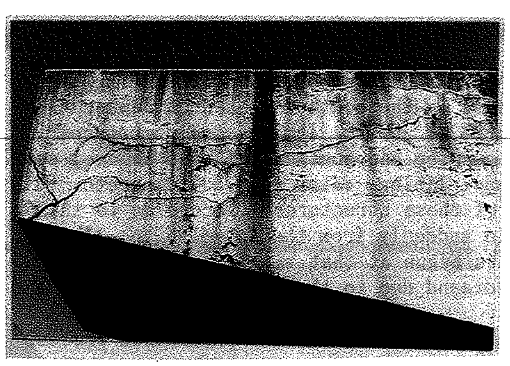

Figure 2 Cracking in a beam

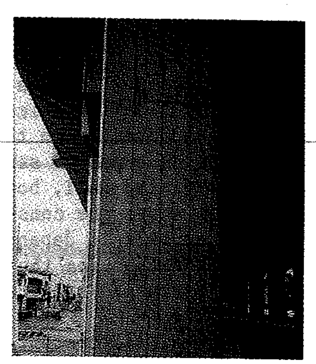

Figure 3 Cracking in a pier

top of a box culvert where horizontal cracks are developing. Figure 5 shows cracking in a retaining wall where horizontal cracks are dominant. Figure 6 is an example of deformed structures due to ASR. Figure 7 is an example of cracking in a building. Figure 8 shows cracking in a column of a tower where vertical cracks are dominant but map like cracks are also developing.

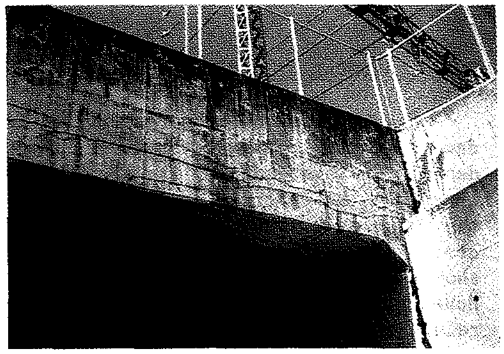

Figure 4 Cracking in a culvert box

figure 5 Cracking in a retaining wall

Figure 6 Deformation of a parapet

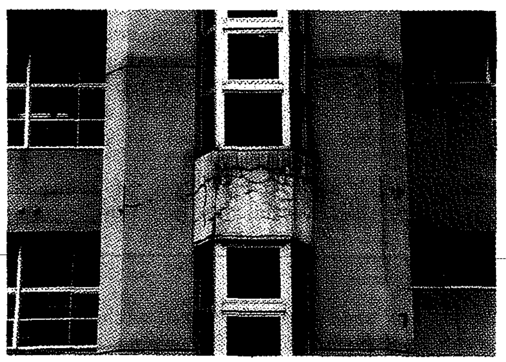

Figure 7 Cracking in a building

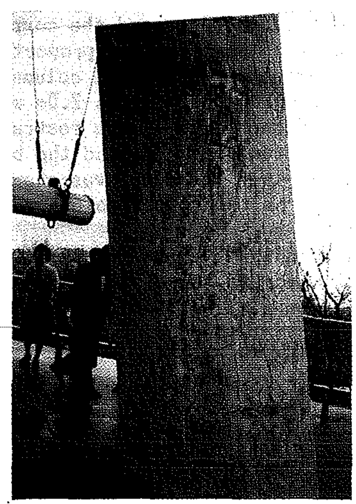

Figure 8 Cracking in a column of a tower

2. ASSESSMENT OF DAMAGE

2.1 Assessment by concrete core

Figure 9 is one of cores drilled from damaged concrete structures to see the crack depth. In this example, the crack depth was more than the concrete cover. Figure 10 shows the relation between the crack depth and the crack width at the surface. These results show a tendency for wider cracks to be deeper. However,the crack depth in most of the reinforced concrete structures investigated was within the range of the concrete cover. Figure 11 shows the degree of expansion obtained from cores of various concrete structures in which bronzite andesite is used with different mix proportions. These results indicate that the extent of damage and cracking due to ASR depends upon the amount of bronzite andesite as a proportion of the total coarse aggregate and the alkali content. The amount of expansion of the cores from concrete structures exhibiting significant cracking exceeded 500×10^{-6}.

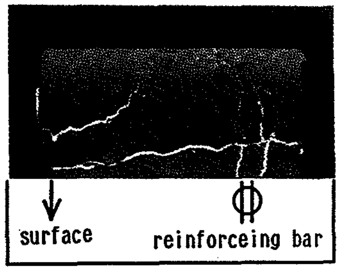

Figure 9 Concrete core

Figure 12 shows compressive strength and Young's modulus of cores drilled from damaged concrete structures by ASR. These results indicate that the compressive strength of concrete decreases due to ASR. The compressive strength of concrete in some structures was only 120 to 150 kgf/cm^2. These results also indicate that Young's modulus of the ASR affected concrete is proportional to its compressive strength and that it is very low. One of the measured values was only 5×10^4 kgf/cm^2. However, Young's modulus calculated from the deflection of the ASR damaged concrete beams was not so low (See Table 4). The difference between Young's modulus of the concrete structure as a whole and that of cores drilled from the structure might be due to the different degree of restraint, but it has not been clearly understood.

Figure 13 shows the relation between the compressive strength of concrete core and the depth at which the core was taken. These cores were drilled from ASR damaged bridge column and the corresponding test model. The model was made of concrete with the same mix proportion, same reactive aggregate and same reinforcement as the actual column. The size of the model are also same as the column of which diameter is 2.0m and height is also 2.0m.The difference between the column and the model is the restraint at the top and the bottom of the column. The top of the model is free and the bottom is restrained only by the friction with the base concrete. The model was stored in the room of RH100% and 40℃. The age of the core from the actual column was 7 years when tested and that from the model was 1 year.

According to these results, the concrete strength near the surface of ASR damaged concrete structure is not always lower than that of the inside. The reason why the compressive strength of the upper part of the model is lower than that of the lower part would be due to poorer mix proportion and higher degree of the reaction.

Test cylinders with same mix proportion as the model concrete but without reactive aggregate were also stored in the room of RH 100% and 40℃ until tested. The compressive strength was 476 kgf/cm^2 at the age of 1 year. These results also indicates that the compressive strength of concrete is lowered by ASR and that the influence extends well into the inside of structures.

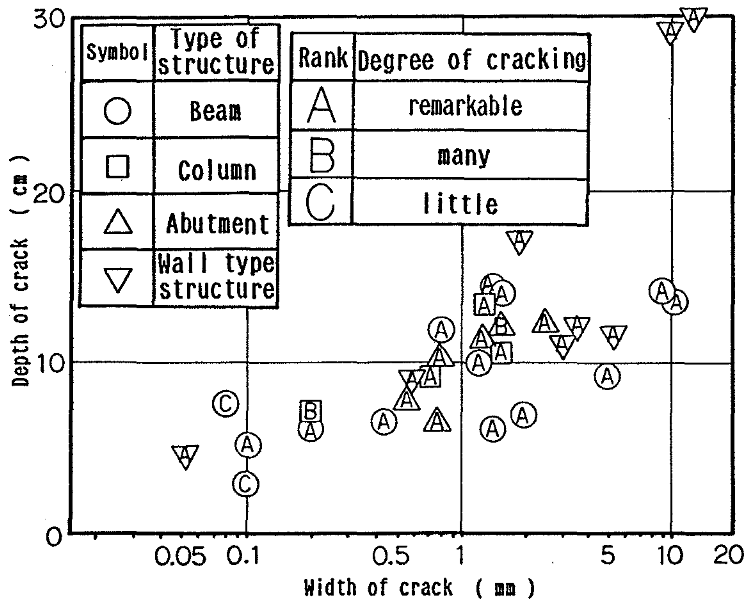

Figure 10 Relation between crack width and depth

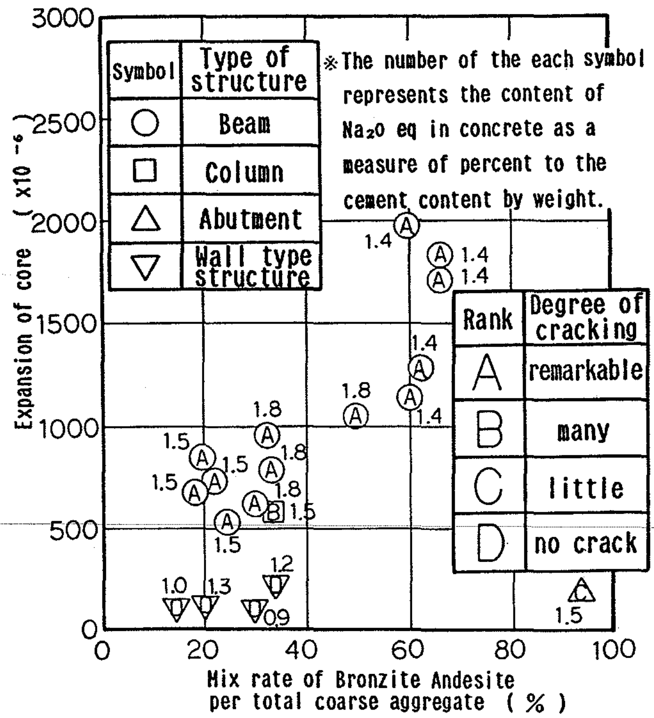

Figure 11 Degree of cracking and expansion of the drilled cores

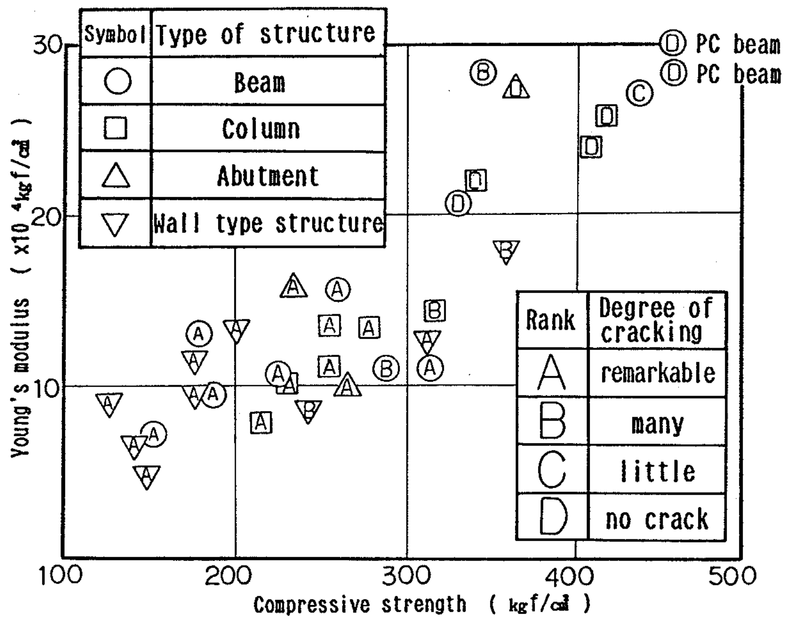

Figure 12 Compressive strength and Young's modulus of drilled cores

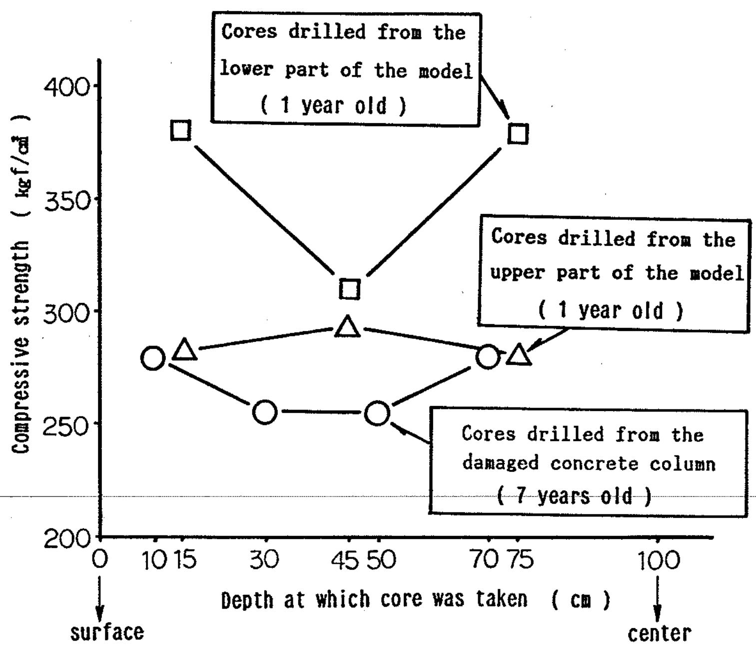

Figure 13 Reaction between compressive strength of concrete core and depth at which core was taken

Table 2 shows the results of field investigations with respect to the depth of carbonization, half cell potentials of the reinforcing bars and corrosion of these reinforcing bars. These results indicate that ASR does not always accelerate carbonization of concrete and that corrosion of reinforcing bars in these ASR damaged concrete structure has not reached a significant state yet. The value of half cell potentials does not seem to be a good measure of jugging the corrosion of the reinforcing bars in these cases.

Table 2 Corrosion of reinforcing bar

Structure	ASR or Normal	Concrete cover (cm)	Depth of carbonization (cm)	Half cell potentials of reinforcing bar in concrete (mV)	Corrosion of reinforcing bar
Sea defence	A	8.5	0.3	350	Spot corrosion
	A	12.0	0.3	330	Spot corrosion
	A	11.5	0.2	310	Spot corrosion
	A	12.5	0.5	210	Spot corrosion
	A	8.0	0.4	140	Spot corrosion
	N	11.5	0.6	———	Nil
Abutment	A	9.2	0.7	230	
	A	7.2	1.0	190	Surface corrosion
	A	7.0	0.6	160	Spot corrosion
	A	7.0	1.0	110	Surface corrosion
	A	9.0	0.9	110	Spot corrosion
	A	7.5	0.4	90	Spot corrosion
	N	9.0	0.7	———	Nil
Beam of pier	A	10.0	1.0	340	Spot corrosion
	A	11.0	1.1	440	Nil
	A	14.0	0.9	390	Nil
	A	4.0	0.9	280	Spot corrosion
	A	5.5	1.0	170	Nil
Column of pier	A	6.5	1.0	290	Spot corrosion
	A	5.5	1.0	250	Surface corrosion
	A	5.8	0.9	160	Spot corrosion
	N	———	1.3	———	———
	N	10.0	1.2	———	Nil

2.2 Assessment by concrete model

It is usually difficult to evaluate strength of a concrete structure being used. Therefore, the influence of ASR to the strength of concrete structures has been accessed by testing of the concrete models.

Following is one of these tests. In this test, reinforced concrete beams of 20 cm × 20cm × 160cm were tested in flexure to evaluate the loss of the strength due to ASR. The amount of reinforcement and stirrup was varied for each beam. Bronzite andesite was used as the reactive aggregate. The mix ratio of the reactive aggregate to the total aggregate was 50% and the equivalent alkali content was 8 kg/m^3. Normal concrete beams correspond to the ASR beams were also made. These beams were stored in the room of RH100% and 40℃ until tested. ASR and normal concrete specimens of ϕ15×30cm were also made and stored in the same room.

The expansion of the ASR beams with the steel ratio of 1.20% converged 761 to 849×10^{-6} and that with the steel ratio of 0.77% converged 1014 to 1088×10^{-6} at the age of 160 days. Many cracks developed dominantly in the axial direction and the crack depth reached 3 to 4cm which was the concrete cover. After conversion of the expansion, flexural test of these beams was performed. The shear span to effective depth ratio, a/d, was varied to be 1.5, 2.0 and 2.5 in the test.

Table 3 shows the test results. The compressive strength of the ASR concrete obtained from the cylinder specimens was 195 kgf/cm² and that of the normal concrete was 467 kgf/cm². The compressive strength of cores drilled from these test beams was 213 kgf/cm² for the ASR concrete and 488 kgf/cm² for the normal concrete.

According to these results, the loss of the strength of these beams due to ASR was measured to be 1.5% to 12.7%. On the other hand, the evaluated loss of the beam strength in bending due to the loss of the compressive strength is 5.9% from the core strength and 6.7% from the cylinder strength.

From these results, it is inferred that the strength of reinforced concrete beams does not affected so much by ASR although the compressive strength of concrete decreases less than half.

Table 3 Test results

*1 Beam No.	*2 Steel ratio (%)	*3 Stirrup (%)	Expansion ($\times 10^{-6}$)	a/d	Maximum load (tonf)	Loss of strength (%)
A1 N1	1.20	0.32	848	2.5	12.9 13.1	1.5
A2 N2	1.20	——	815	2.0	14.5 16.6	12.7
A3 N3	1.20	0.073	761	2.0	15.0 16.4	8.5
A4 N4	1.20	0.32	849	2.0	13.7 14.9	8.1
A5 N5	0.77	——	1088	2.0	11.4 12.8	10.9
A6 N6	0.77	0.073	1014	2.0	11.6 12.9	10.1
A7 N7	1.20	0.073	827	1.5	21.9 22.7	3.5
A8 N8	1.20	0.32	836	1.5	20.1 21.1	4.7

*1 A denotes ASR concrete and N denotes normal concrete
*2 Steel ratio of the compression side is same as the tension side
*3 The spacing is 10cm apart

2.3 Assessment by loading test

Both ASR damaged and normal concrete beams of a highway pier were loaded statically and dynamically by a truck as shown in Figure 14. These beams are identical except ASR. The total weight of the truck was 59.6 tons for the static loading and 32.3 tons for the dynamic loading. Displacement of these beams was measured at the end by an optical method.

Table 4 shows the summary of the test results. The rigidity of the beam is assumed to be inversely proportional to the displacement and proportional to the square of the natural frequency. The Young's modulus of the concrete was calculated from the displacement by the use of the finite element method and the framed structure analysis.

According to these results, the loss of rigidity due to ASR was measured to be only 13 to 15%. However, it should be noticed that the loss has occurred within only 3 years although this highway is designed for 50 years of life time. Therefore, it is required to make a careful and constant inspection to damaged concrete structures by ASR, particularly to load bearing structures such as highway pier.

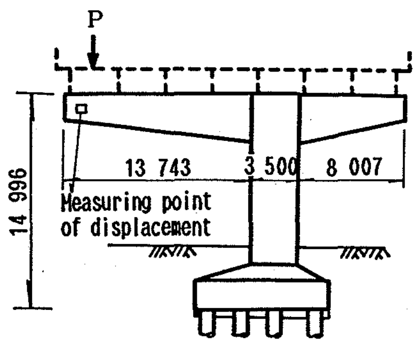

Figure 14 Loading test

Table 4 Results of loading test

	Normal	ASR
Static displacement of the beam end (P=59.6ton)	2.40 mm	2.75 mm
Rigidity ratio	1.0	0.87
Back analyzed Young's modulus of concrete. ( ) is measured by the core	40.5×10^4 kgf/cm²	35.3×10^4 kgf/cm² (11.2×10^4 kgf/cm²)
Dynamic displacement of the beam (P=32.3ton). Speed of the car=60km/h	1.47 mm	1.69 mm
Rigidity ratio	1.0	0.87
Natural frequency	2.73 Hz	2.52 Hz
Rigidity ratio	1.0	0.85

3. REPAIR

Repair of concrete structures damaged by ASR has just started in Japan. Many investigations have been carried out to establish an effective method to inhibit ASR. This section introdues mainly coating method and examples of repair done for damaged concrete structures.

3.1 Repair by coating

3.1.1 Laboratory test Many test specimens were made by ASR concrete and repaired by coating after cracking. These specimens were a prisum of $10\times10\times40$cm. Bronzite andesite was used as the reactive aggregate and the equivalent alkali content was 2.0% of the cement content or 6.9kg/m³. The specimens were stored in the room of RH100% and 40℃ until coating.

Coating was performed when the specimens cracked fully as shown in Figure 15. At this stage, the average expansion was about 1000×10^{-6}. For the coating materials, polyurethane and epoxy resin were selected as waterproof type of coating and polymer cement were selected as aeration type of coating. After coating, the specimens were stored 3 weeks in the room of RH100% and 40℃ and 1 week in the open air, repeatedly.

Expansion of specimens coated by the polyurethane and the epoxy resin reached about 2000 to 2500×10^{-6} within 6 months after coating and cracked again. But expansion of specimens coated by the polymer cement have ceased for 2 years.

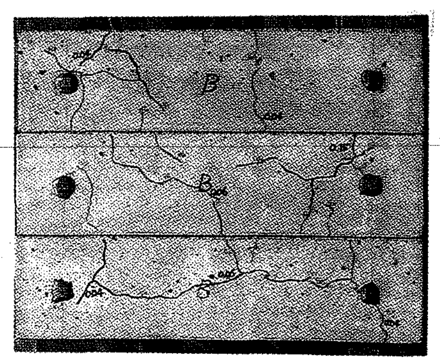

Figure 15 Cracked specimens

3.1.2 Field test Concrete piers damaged by ASR were repaired by various types of coating materials and the observation was made after coating. Following results were obtained.

The beams repaired by polyurethane and epoxy resin cracked again within a year (Figure 16 and 17).

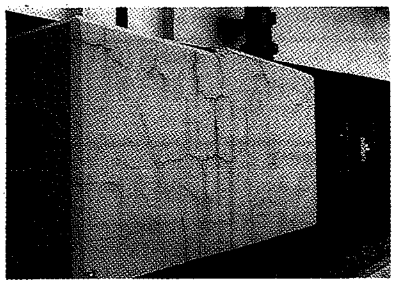

Figure 16 Beam repaired by polyurethane resin

Figure 17 Beam repaired by epoxy resin

The column repaired by polybutadiene cracked at 2 years after coating and gel was exuded (Figure 18).

In spite of the large residual expansion of 1400×10^{-6}, the beams repaired by the polymer cement ceased to expand and have not cracked again for 2 years after coating.

Many more investigations are required to find out an effective coating method. However, from these laboratory and field test, the polymer cement type of coating used in this test is expected to be one of effective methods to inhibit ASR.

Figure 18 Column repaired by polybutadiene

3.2 Examples of repair

Figure 19 shows repair of bridge pier by the polymer cement coating. In this

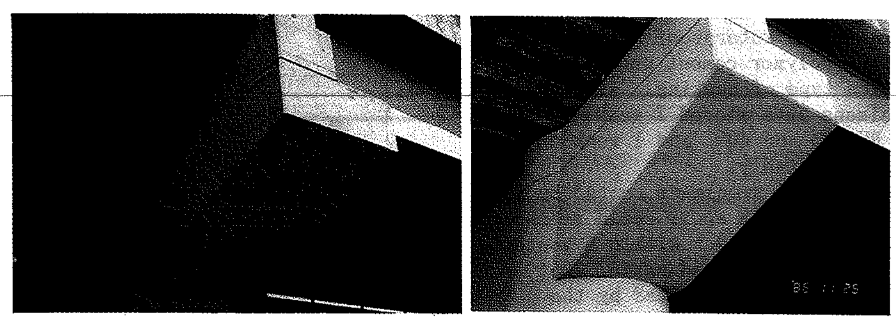

Figure 19 Beam repaired by the polymer cement

repair, cracks wider than 0.3mm were injected by epoxy resin before coating. After coating the expansion of the pier by ASR has ceased.

Figure 20 shows repair of river defence. About 10cm of concrete was placed on the top of the river defence since cracking occurred only at the top.

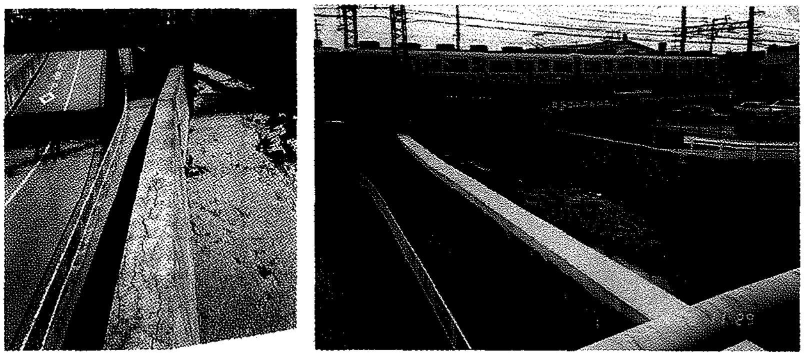

Figure 20 Repaired river defence

Figure 21 shows an example of repair by steel plate. The steel plate was adhered to the damaged concrete surface by epoxy resin.

Figure 22 shows a railway pier. In this beam, a diagonal crack occurred near the centre of the beam in addition to ordinary ASR cracks. Therefore, the beam was tighed up in the axial direction by prestressing bars and the anchor frames(Figure 23). After prestressing, concrete lining was made around the beam (Figure 24).

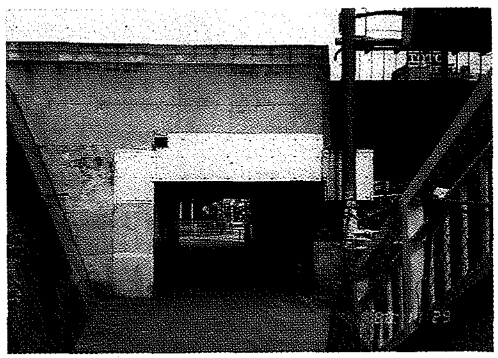

Figure 21 Repair by steel plate

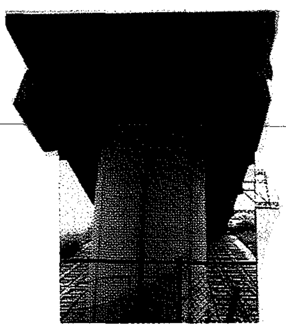

Figure 22 Pier before repair

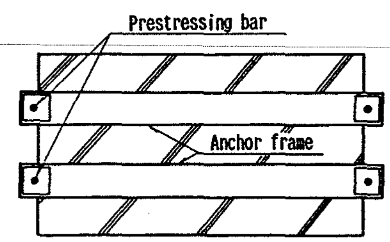

Figure 23 Repair by prestressing

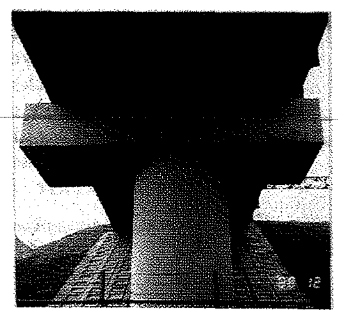

Figure 24 Repaired pier

Figure 25 shows deformed parapets by ASR. These parapets were constructed in 1975 and various investigations were made in 1986. Figure 26 shows the relation between the horizontal displacement at the top and the average crack width of each span of the parapets. The group A and B parapets of which horizontal displacement was more than 5cm were replaced by new one. The group C was coated by the polymer cement. The group D was left for future observation.

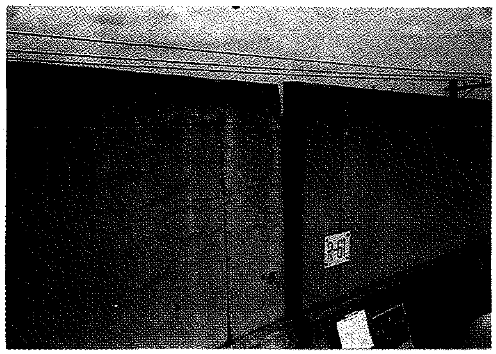

Figure 25 Deformed parapet

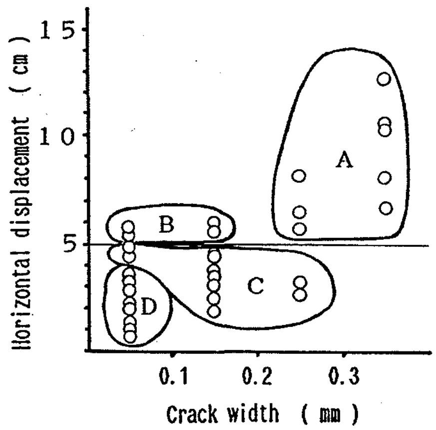

Figure 26 Crack width and displacement of parapets

4. CONCLUDING REMARKS

From the investigations introduced in this paper, following tentative conclusions may be drawn.

(1) Sudden increase of ASR in Japan started about 20 years ago.
(2) Damaged structures by ASR can be seen at many places in Japan.
(3) Bronzite andesite, chert, slate, tuff, sandstone, opal etc. were identified as reactive aggregate.
(4) The degree of damage by ASR depends mainly upon the characteristic and content of reactive aggregate used, the alkali content, degree of restraint of the structure, ambient temperature and moisture content.
(5) According to the facts that cracks due to ASR have not reached deep enough and that reinforcing bars in damaged structures have not corroded severely yet, damaged structures in Japan due to ASR seem to have not reached a severe stage yet. However, successive loss of concrete strength and rigidity of the damaged structures seem to be giving a warning for future deterioration of these structures. Therefore, constant inspection or investigation is essential for these structures.
(6) An effective method to inhibit ASR of damaged structures is needed to prolong their life.