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Effect of Alkali Silica Reaction Expansion and Cracking on Structural Behavior of Reinforced Concrete Beams

by Shenfu Fan and John M. Hanson

A laboratory study was carried out to investigate the effect of deleterious ASR expansion on the structural behavior of reinforced concrete beams and on mechanical properties of cylinders made with the same concrete. The specimens were conditioned by immersion in a cyclically-heated alkali solution for one year to accelerate ASR. To simulate in-service conditions, two beams were held under load that caused flexural cracking while being conditioned.

Cracks were first observed in the cylinders at an age of 125 days. Before ASR cracking occurred, changes in the mechanical properties of the reactive cylinders were minor. After cracking, the compressive strength, splitting tensile strength, and dynamic modulus of the cylinders were significantly reduced. Cracks were first observed on top of the beams after 6 months of conditioning and were oriented in the direction parallel to the reinforcement. However, after conditioning for one year, the flexural strength of the reactive beams which experienced ASR cracking was nearly the same as that of the nonreactive concrete beams. Effect of ASR on flexural strength of the pre-loaded and cracked beams was also insignificant.

Keywords: alkali-silica reaction (ASR); compressive strength; cracking; dynamic modulus; expansion; flexural loading capacity; reinforced concrete beams; splitting tensile strength.

INTRODUCTION

Alkali-silica reaction (ASR) of concrete has been studied for more than fifty years since it was first recognized in 1940 by Stanton.[1] Deleterious ASR expansion may lead to cracking, surface pop-outs and spalling, distortion of elements, and loss of concrete strength and elasticity. Methods to prevent or minimize ASR deterioration, as described by Stark,[2] include: 1) avoiding use of reactive aggregates, 2) limiting the alkali content of cement, and 3) incorporation of pozzolans and other admixtures. Although knowledge of ASR has been expanding for several decades, the number of existing structures exhibiting ASR deterioration is apparently increasing. In recent years, extensive cracking has been found in several bridges in North Carolina. The number may continue to increase because of difficulties in obtaining low alkali cement, dwindling good quality aggregate supplies, and use of high-strength concrete with a high cement content. ASR deterioration in existing structures has raised concerns over serviceability.

RESEARCH SIGNIFICANCE

A few laboratory tests[3-6] on reinforced concrete beams and other structural elements affected by ASR have been conducted in recent years. However, the effect of reinforcement on ASR expansion and cracking, and the structural behavior of ASR-deteriorated elements, is not fully understood. In this research, beams were tested after one year of ASR accelerated conditioning. Two of the beams had been pre-loaded to cause cracks on their tension face, and then subjected to the conditioning while maintaining the cracking load to simulate service conditions. The results of this research provide information on the structural behavior of reinforced concrete beams exhibiting ASR deterioration.

EXPERIMENTAL PROGRAM

Constituent materials

An ordinary portland cement (ASTM Type I/II) with an alkali content of 0.65 percent equivalent sodium oxide ($Na_2O + 0.658 K_2O$) was used. Gold Hill coarse aggregate from a quarry in North Carolina was chosen as a reactive aggregate. This aggregate had been used in the James Garrison Bridge, which has experienced extensive cracking due to ASR. This aggregate has been confirmed to be highly reactive by the North Carolina Department of Transportation.[7] For comparison, a nonreactive coarse aggregate with long-time good field performance was selected. Natural sand used in this laboratory test program as fine aggregate was also nonreactive. Physical properties of the aggregates are described in Table 1. Coarse aggregate was sieved and particles larger than $^3/_4$ in. (19 mm) were removed. Grade 60, No. 3 and No. 5 deformed steel bars were used for tensile reinforcement. Tensile strength tests on two No. 3 bars were carried out and the yield strength of the bars was 62.8 and 63.4 ksi (433 and 437 MPa), respectively. The No. 5 Bars were not tested.

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Test specimens

Two concretes were produced, one with the reactive aggregate and the other with the nonreactive aggregate. The mix proportions are shown in Table 2.

Test specimens are listed in Table 3. Six 60 in. (1500 mm) long reinforced concrete beams were made, three with the reactive concrete indicated by *R*, and three with nonreactive concrete indicated by *N*. All beams had a 6 × 10 in. (150 × 250 mm) rectangular cross section and were designed as singly-reinforced. The beams had a reinforcement ratio of either 0.004 (using two No. 3 bars) or 0.01 (using two No. 5 bars). Stirrups made with D-5 wire were placed in the shear spans of all the beams. Details of the beams are shown in Fig. 1. In addition, 4 × 8 in. (100 × 200 mm) cylinders representative of concrete in the beams were also cast. The average strength of the reactive and nonreactive concrete at 28 days was 5030 and 5210 psi (34.7 and 35.9 MPa) with a standard deviation of 67 and 60 psi (0.46 and 0.41 MPa), respectively.

The test specimens were kept in the forms used for casting for 2 days. After demolding, the specimens were cured in a standard moisture room for 14 days. Custom-made stainless steel Demec studs were then bonded on the specimens using epoxy for length expansion measurements, as shown in Fig. 2. The studs were $^1/_8$ in. (3.1 mm) thick, $^1/_2$ in. (12.5 mm) diameter disks having a small, partial-depth hole in the center.

ASR accelerated conditioning

Three galvanized steel tanks having dimensions of 3 × 3 × 8 ft (0.9 × 0.9 × 2.4 m) were used for ASR accelerated conditioning. All specimens were placed in the tanks, which contained a 0.5 N concentration alkali solution. The solution was made by adding 10 grams of sodium hydroxide (NaOH),

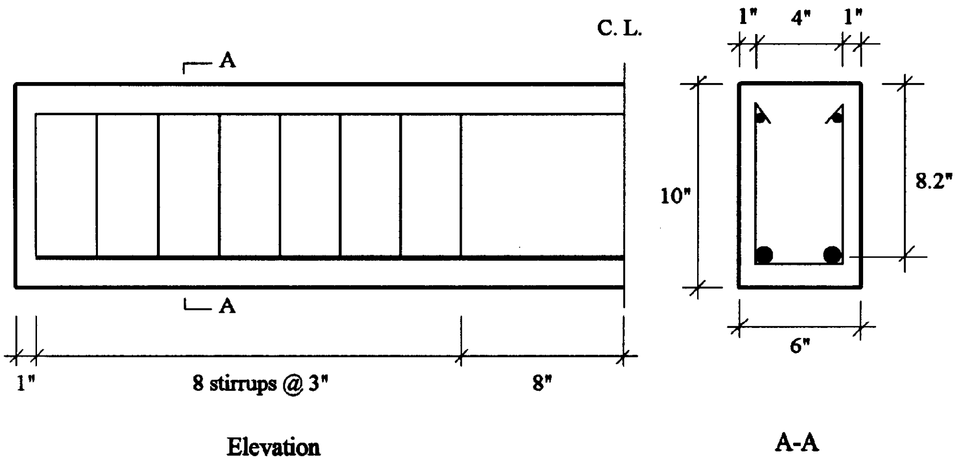

Fig. 1—Reinforced concrete beams

Table 1—Physical properties of aggregates

Aggregates		Reactive coarse aggregate	Nonreactive coarse aggregate	Nonreactive fine aggregate
Classification		Meta-argillite	Silica sand	Granite quartz
Bulk specific gravity, SSD		2.78	2.64	2.57
Dry-rodded unit weight, lb/ft^3		94	92	—
Absorption, SSD percent		0.52	0.60	1.10
Fineness modulus		—	—	2.66
Grading, percent (Passing sieve size)	1 in.	100	100	N/A
	$^3/_4$ in.	90	97	
	$^1/_2$ in.	71	84	
	$^3/_8$ in.	40	68	
	No. 4	23	51	
	No. 8	7	34	
	No. 16	3	17	

N/A = Not applicable

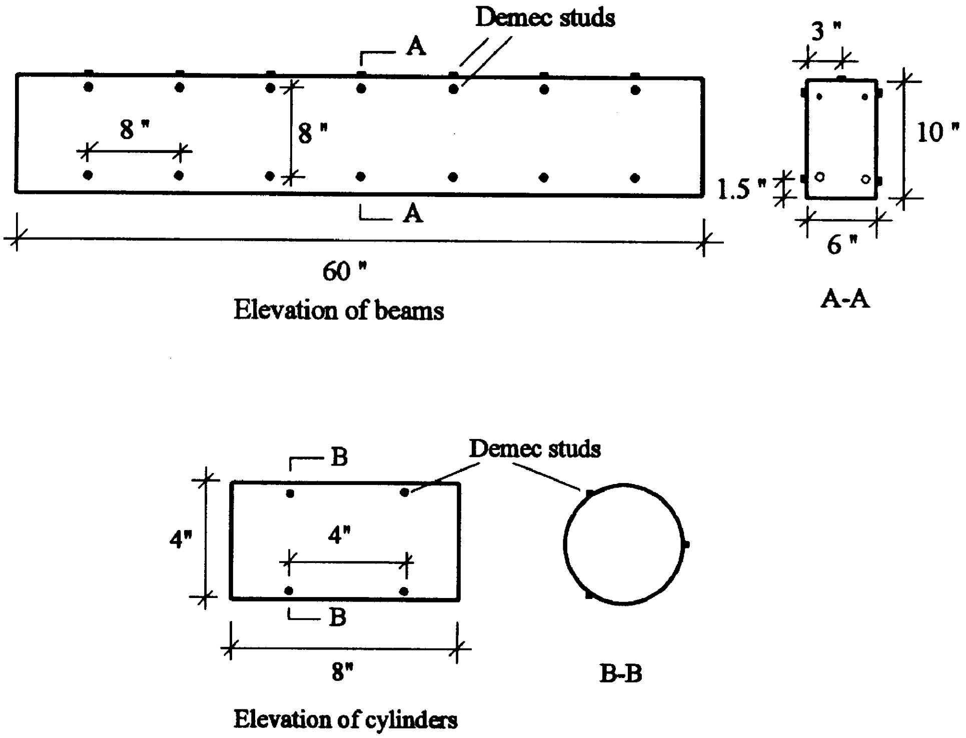

Fig. 2—Arrangement of Demec studs on specimens

14 grams of potassium hydroxide (KOH) and 0.1 grams of calcium oxide (CaO) per liter of tap water, based on procedures proposed by the British Cement Association.[8] The solution in each tank was heated to 100 F (38 C) by a temperature-controlled heater and circulated by using an immersed chemical resistant pump, to maintain a uniform temperature. After 5 to 7 days of heating, the alkali solution was allowed to cool to room temperature of around 75 F (24 C) for 2 days. The cooling to room temperature took about 6 hours. This heat-up-cool-down cycling procedure continued until the end of the tests.

TEST RESULTS AND DISCUSSION

Expansion and cracking due to ASR

Length expansion was measured with a Demec mechanical dial gage with a least count accuracy of 0.00008 in. (0.002 mm) using the stainless steel studs mounted on the surface of the specimens. Initial readings were taken just before the specimens were put in the alkali solution. All subsequent measurements were taken immediately after the specimens were temporarily taken out of the solution, during the period when the solution was at 100 F (38 C). As soon as the measurements were completed, the specimens were returned to the tanks for the continuation of ASR accelerated conditioning. In addition, the specimens were periodically inspected for cracks with the aid of a magnifying glass and a crack comparator. Cracks were traced on the specimens and maps were made of the cracking.

The results of the length expansion measurements on the beams are shown in Fig. 3 and 4. Initially, in the first few days after immersion, the beams expanded about 250 to 300 microstrain, regardless of whether they were made with reactive or nonreactive concrete. This expansion is believed to be caused by the increase in temperature and absorption of moisture, because the initial readings were taken in air at room temperature before the beams were immersed in the alkali solution. For the nonreactive beams, subsequent expansion was insignificant. For the reactive beams, the subsequent expansion due to ASR started about 5 months after immersion in the alkali solution. At an age of one year, the longitudinal expansion on the top of beams #3R and #5R1 was approximately 1400 and 1700 microstrain, respectively. Due to the restraint from the reinforcement, the longitudinal expansion at the level of the reinforcement was greatly reduced. Expansion in the transverse direction was also reduced. Cracking development and growth is shown in Fig. 5 and 6. ASR cracks first appeared on the top of the beams longitudinally when the expansion reached approximately 800 microstrain, including the expansion due to temperature elevation and concrete saturation during the first several days. The cracks propagated and became connected to each other, forming main longitudinal cracks. Then, cracks transverse to the reinforcement occurred and joined the main longitudinal cracks. There was little difference in the rate and extent of ASR expansion and cracking between beams reinforced with No. 3 bars and beams with No. 5 bars.

To investigate the ASR expansion of beams while under simulated service loads, two beams, the reactive beam #5R2 and the nonreactive beam #5N2, were first set up in tandem for loading, as shown in Fig. 7. Two steel blocks with dimensions of 1.5 × 1.5 × 6 in. (37.5 × 37.5 × 150 mm) were placed 16 in. (400 mm) apart in the center between the beams. A loading system was mounted at both ends of the beams. The loading was gradually increased using two hydraulic rams until cracks appeared having a width of about 0.008 in. (0.2 mm) on the tension face of the beams. The calculated stress

Table 2—Concrete proportions

Type of concrete	Reactive	Nonreactive
Ratio of water to cement	0.47	0.47
Mixing water, lb/yd^3	320	320
Cement, lb/yd^3	680	680
Coarse aggregate, lb/yd^3	1612	1577
Sand, lb/yd^3	1246	1201
Water reducer, oz/cwt	4	4
Air entraining agent, oz/cwt	0.5	0.5

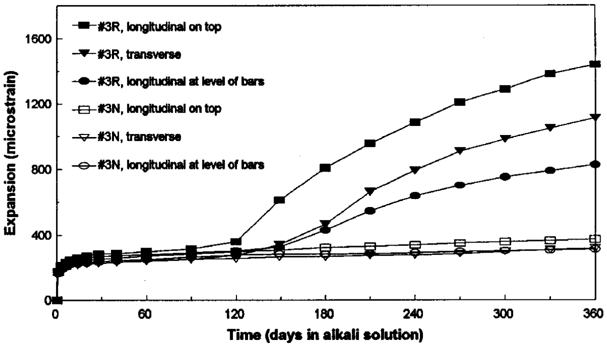

Fig. 3—Length expansion of Beams #3N and #3R

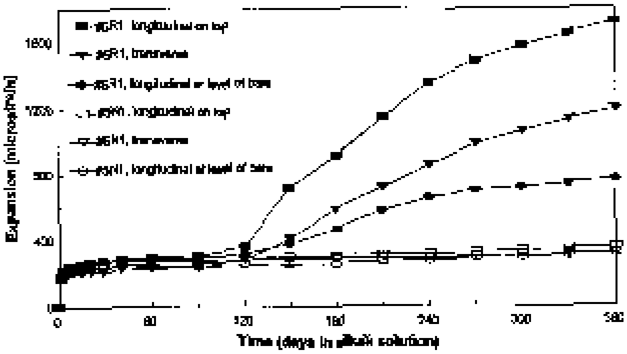

Fig. 4—Length expansion of Beams #5N1 and #5R1

in steel at the cracking load was 13 ksi (89.7 MPa). After that, the cracking load was maintained by tightly fastening the screws on the steel rods, and the loading system was removed. The pre-cracked beams were then immersed in the alkali solution for conditioning while under the cracking load, as shown in Fig. 8. During the conditioning, the applied load was checked regularly by measuring steel strain using Demec studs installed on the rods, and adjusted if necessary.

After one year of conditioning, cracks had developed on the compression face of the pre-cracked reactive beam (#5R2). However, the cracks transverse to the reinforcement were suppressed due to the retained applied load, compared with those of the similar beam that was not pre-cracked (#5R1), as shown in Fig. 9. The pre-existing cracks did not extend during immersion in the alkali solution. In addition, even though the pre-cracked beam #5R1 was under the applied load, only a few cracks due to ASR were found on the tension face, and these cracks were smaller than on the compression face.

Because the electrical resistance strain gages bonded on the bars survived only 4 to 5 months, strain readings on bars induced by ASR expansion was not obtained.

Mechanical tests of concrete cylinders

Compressive and splitting tensile strength tests of 4 × 8 in. (100 × 200 mm) cylinders were carried out in accordance with ASTM C39[9] and C496,[10] respectively. Nondestructive dynamic modulus tests were also carried out on two reactive and two nonreactive cylinders. Longitudinal frequencies of concrete were measured by using an impact resonance method in accordance with ASTM C215-91.[11] Dynamic modulus of elasticity was then calculated based on the frequencies, specimen geometry, and mass. After each measurement was completed, the cylinders were quickly returned to the alkali solution for continuation of ASR accelerated conditioning.

Mechanical properties of the nonreactive cylinders did not change significantly with time. Results of compressive strength, splitting tensile strength, and dynamic modulus tests, and length expansion measurements of the reactive cylinders are summarized in Fig. 10. It was found that the change in the mechanical properties was closely related to the ASR expansion. The compressive strength, splitting tensile strength, and dynamic modulus were not affected significantly up to 90 days. However, at an age of 125 days, just after ASR cracks were found, the concrete mechanical properties were greatly reduced. At an age of 180 days, the loss of the compressive strength, splitting tensile strength, and dynamic modulus of the cylinders was 24, 38, and 31 percent, respectively, compared with the corresponding 28-day

Table 3—Test specimens

Concrete specimens		Specimen size	Coarse aggregate	Tension reinforcement	No. of specimens
Beams	#3R	6 × 10 × 60 in.	Reactive	2 No. 3 bars	1
	#3N		nonreactive	2 No. 3 bars	1
	#5R1		Reactive	2 No. 5 bars	1
	#5N1		nonreactive	2 No. 5 bars	1
	#5R2*		Reactive	2 No. 5 bars	1
	#5N2*		nonreactive	2 No. 5 bars	1
Cylinders	R	4 × 8 in.	Reactive	None	40
	N		nonreactive	None	40

* = Pre-cracked

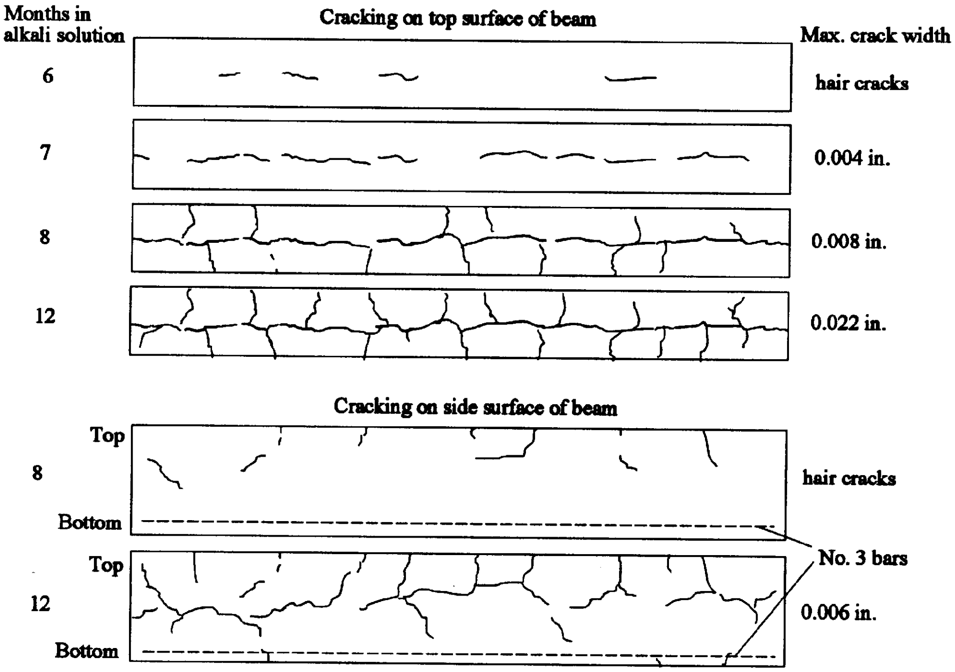

Fig. 5—Cracking of Reactive Beam #3R

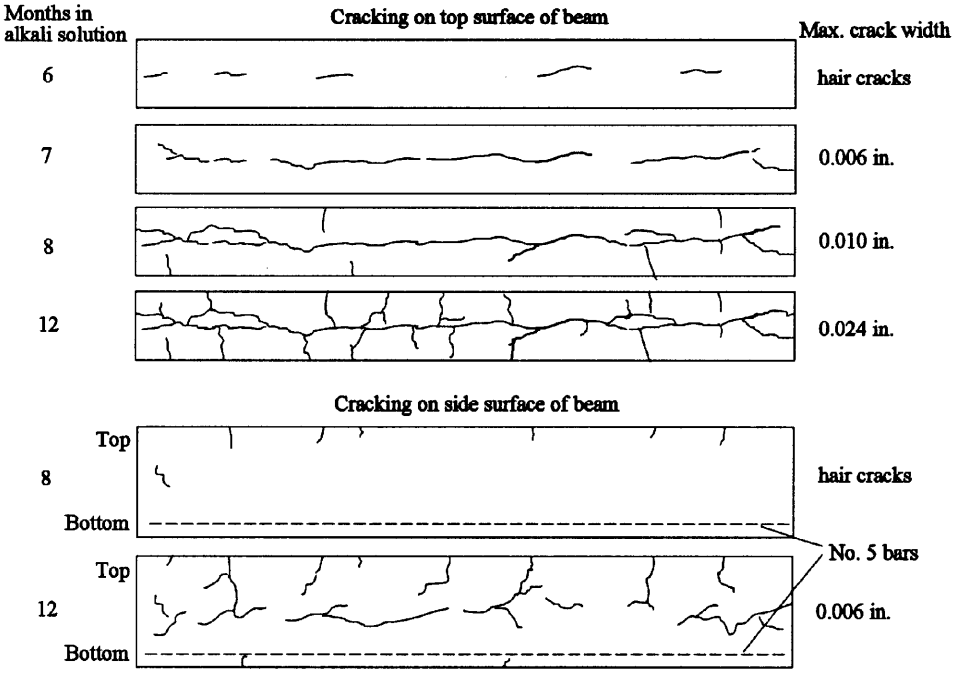

Fig. 6—Cracking of Reactive Beam #5R1

values. It was also observed that the reduction of the splitting tensile strength was higher than that of the compressive strength and dynamic modulus. At an age of one year, further reduction was small.

Beam flexural loading tests

After one year of ASR accelerated conditioning, flexural loading tests were carried out to investigate the structural behavior of the reinforced concrete beams. The setup for the flexural loading tests is shown in Fig. 11. Cracks due to ASR were first marked so as to be distinguishable from the cracks due to the applied load. Deflection at the midspan of the beams was measured using a Linear Variable Differential Transformer (LVDT). Curvature in the constant moment region was obtained by measuring strain at both the top and bottom faces using two LVDTs held by a mounting frame. The beams were loaded symmetrically at two points spaced 16 in. (400 mm) apart, at a rate of 0.0036 in./min (0.09 mm/min)

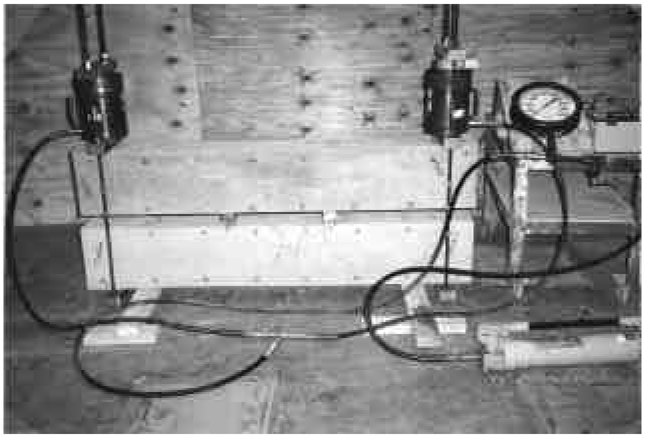

Fig. 7—Beam setup for loading at both ends

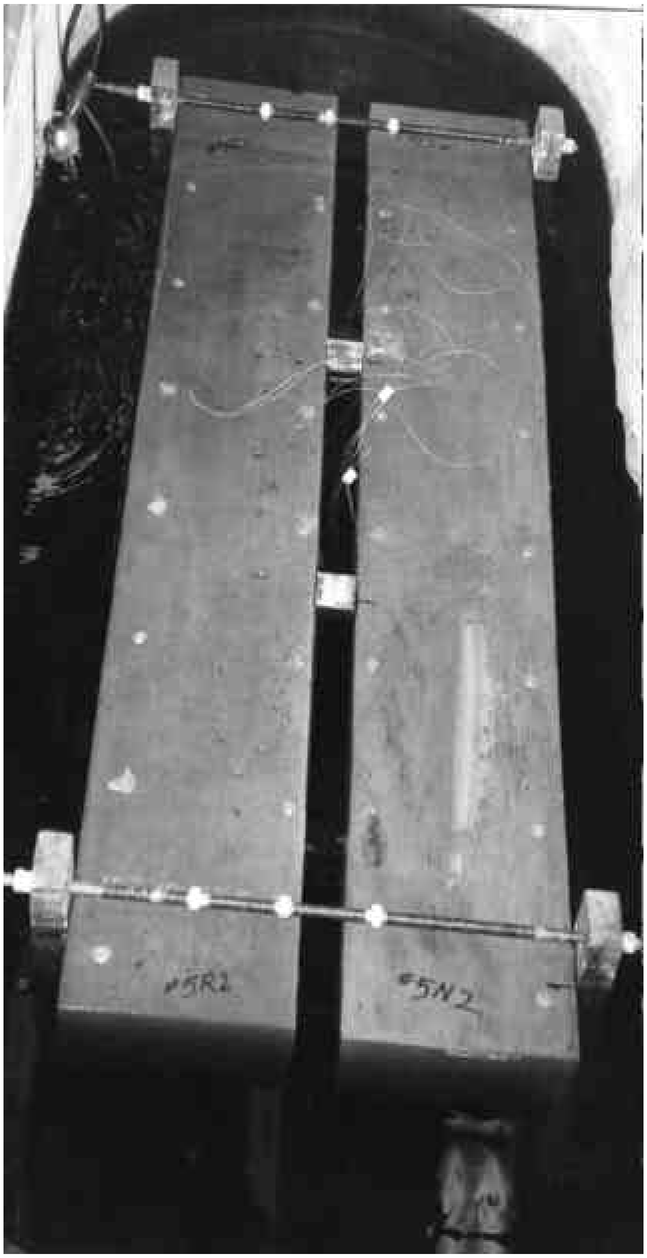

Fig. 8—Immersion of pre-cracked beams in alkali solution

until concrete crushing occurred on the top face of the compression zone.

Results of the tests are shown in Fig. 12 to 14. There was little difference in the load at first cracking and steel yielding between the reactive and the nonreactive beams. Although the deflection and curvature of the reactive beams after yielding were slightly lower, their flexural strength was nearly the same as that of the nonreactive beams.

It was also found that the flexural strength of beam #5R2, which was pre-cracked and under loading while being conditioned, was nearly the same as that of the similar beam #5R1. Therefore, the effect of ASR on the flexural strength of the pre-cracked beam was insignificant.

In addition, it was also seen that the existing ASR cracks of the reactive beams were apparently not enlarged in width and length during the loading tests, and most cracks due to the applied load did not connect with the ASR cracks.

The ASR expansion and cracking did not reduce the flexural loading capacity of the concrete beams, despite the substantial reduction in the compressive strength, splitting tensile strength, and dynamic modulus of the concrete cylinders, as described previously. In other words, ASR had a much more detrimental effect on the mechanical properties of concrete cylinders than on the structural behavior of reinforced concrete beams. One reason may be that the detrimental effects of ASR were limited to the beam surface. Another reason is that the beams were under-reinforced, so that the change in concrete strength from around 5000 to 3500 psi (34.5 to 24.2 MPa) made little difference to the beam flexural strength.

CONCLUSIONS

1. Cracking was observed on reinforced concrete beams made with the reactive aggregate after 6 months of immersion in an alkali solution alternately heated to 100 F (38 C) and then cooled to room temperature, when length expansion due to ASR reached approximately 500 microstrain. The dominant cracks were oriented in the direction parallel to the reinforcement.

2. Before visible ASR cracks occurred, change in mechanical properties of concrete cylinders was insignificant, but after cracking, substantial reduction was found. At an age of 6 months in ASR accelerated conditioning, the compressive strength, splitting tensile strength, and dynamic modulus were reduced by 24, 38, and 31 percent, respectively, compared with the corresponding 28-day values. At an age of one year, further reduction was small.

3. Even though the reactive reinforced beams experienced visible cracking due to ASR, their flexural loading capacity was nearly the same as that of the nonreactive beams. Flexural strength of the beam which was pre-cracked and then maintained under the cracking load during ASR conditioning was also the same as that of the beam which was not pre-cracked.

ACKNOWLEDGMENTS

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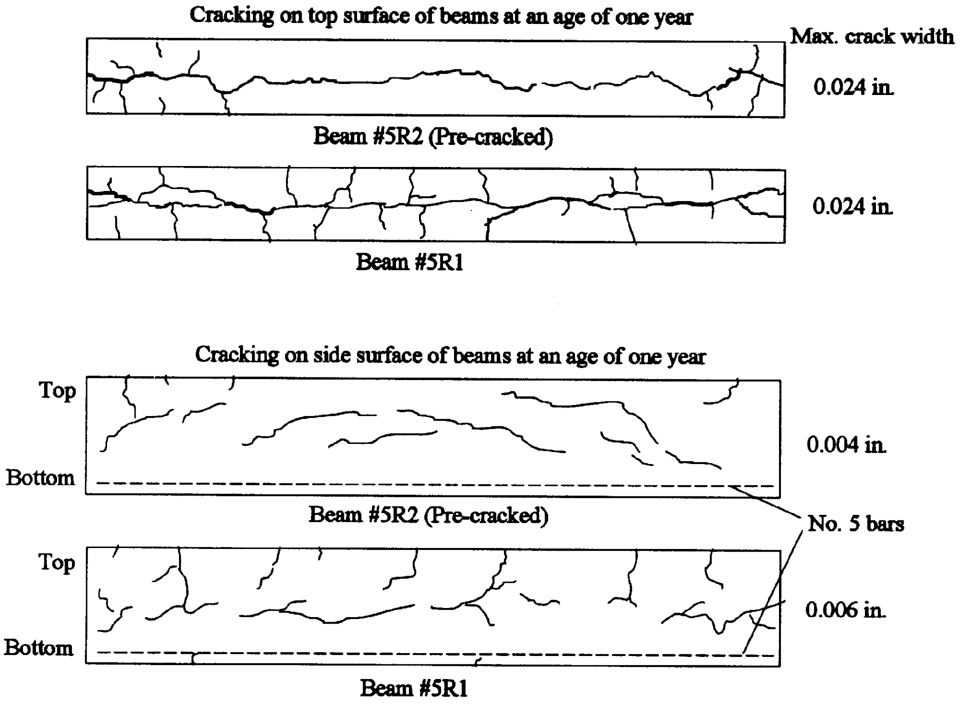


Fig. 9—Comparison of cracking between Beams #5R2 and #5R1

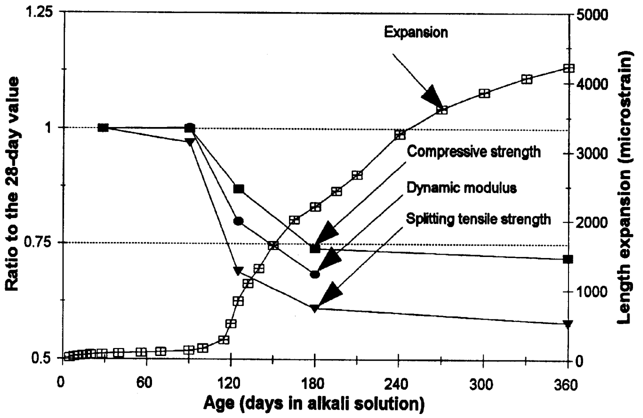


Fig. 10—Change in mechanical properties of reactive cylinders with ASR expansion

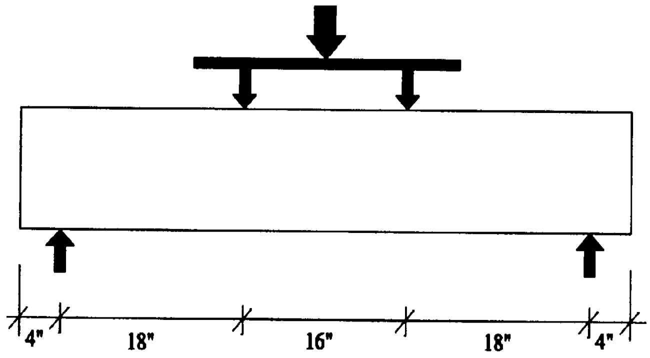
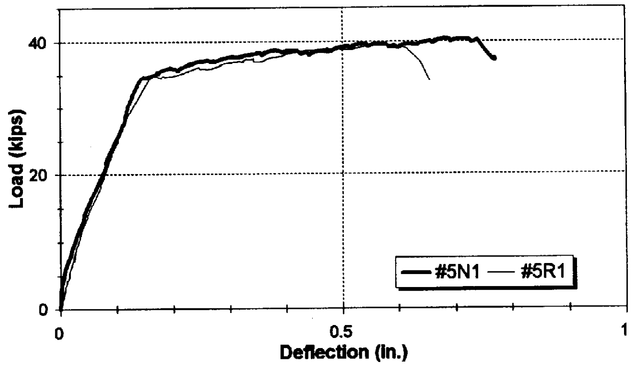
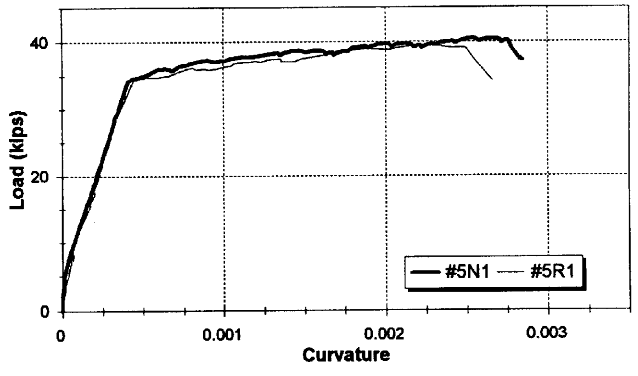


Fig. 11—Beam setup for flexural loading tests

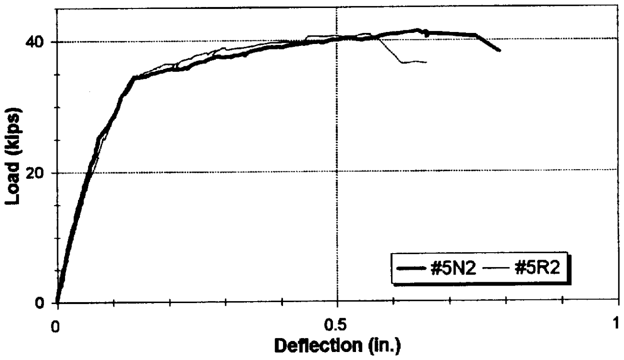


Deflection at Center vs Load

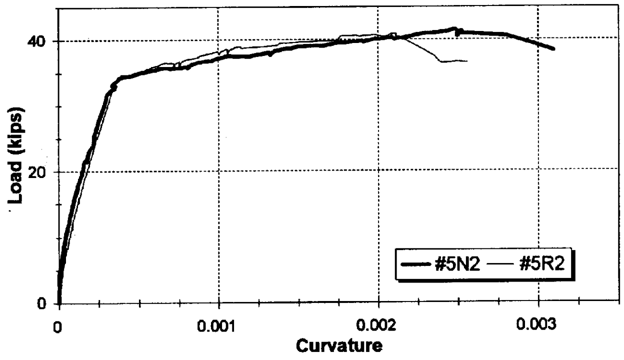


Curvature vs Load

Fig. 12—Results of flexural loading tests of Beams #5N1 and #5R1

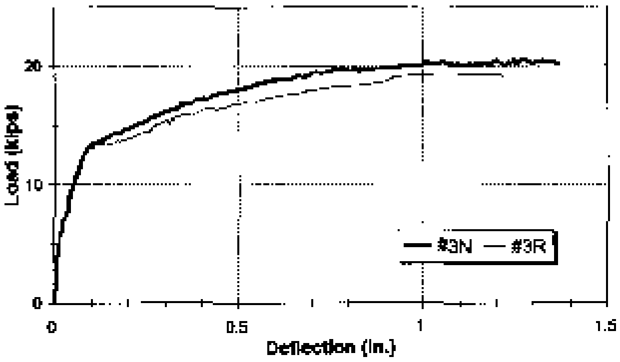


Deflection at Center vs Load

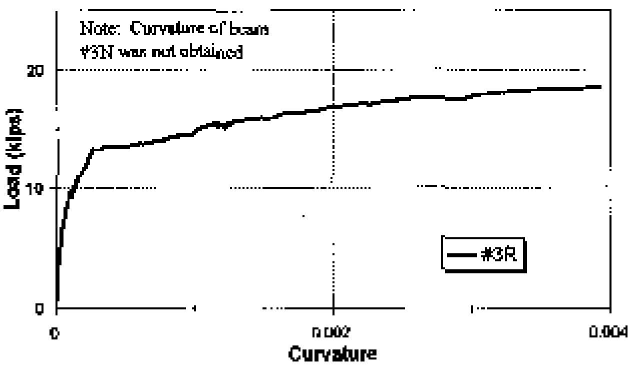


Curvature vs Load

Fig. 13—Results of flexural loading tests of Pre-cracked Beams #5N2 and #5R2



Deflection at Center vs Load



Curvature vs Load

Fig. 14—Results of flexural loading tests of Beams #3N and #3R

CONVERSION FACTORS

1 in. = 25.4 mm
1 lb/ft^3 = 16 kg/m^3
1 lb/yd^3 = 0.59 kg/m^3

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