EFFECT OF AAR ON SHEAR CAPACITY OF BEAMS, WITHOUT SHEAR REINFORCEMENT

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Test data from a series of beams are compared with predictions of capacity based on a familiar formula used for compliance with the U.K. code based method of design. The beams were made from a concrete with a strength usual for bridges. AAR was promoted by the use of a high cement content, added alkali salts and by storing members under hot water. Amongst the parameters considered are: end anchorage of bending steel; free expansion of the concrete; shear span; size of cross-section; the effects of repeated loadings; and the orientation of AAR induced cracking.

INTRODUCTION

In the early 1980s, engineers faced with managing structures that were showing signs of concrete deterioration due to the effects of AAR in the U.K. called for a co-ordinated research programme to provide data on remaining shear capacity. A forum called by the DOE and the SERC determined that: tests should be conducted on specimens manufactured to a common mix design and subjected to a common conditioning regime; members with different expansion levels should be tested; shear capacity in members with no shear reinforcement was of prime concern.

Common mix design and conditioning regime

At the outset, it was determined that each research centre would use the C50 BRE (Building Research Establishment) 'bridge mix', which gave a twenty eight day cube strength of 50 N/mm^2, and allowed specimens up to full size to be made. The SERC and BRE co-ordinators decided that variability would be minimised if the BRE was made responsible for providing the materials for manufacture of the specimens. For the same reason, they also determined that the specimens for the two studies sponsored by the SERC would be cast by the British Cement Association. An accelerated reaction was ensured by using a cement content of 400 kg/m^3; a total reactive alkali content of 7 kg/m^3, by dissolving sodium and potassium sulphates in the mixing water; and by conditioning the specimens under water at 38°C after twenty eight days of normal curing.

When the research programme started, it was expected that the beams would be placed flat in the hot water tanks. Expansion levels of cylindrical specimens would be monitored, until predetermined expansion levels were reached, and the beams would then be removed for testing.

Strains on the beam elevations were also monitored. The results from the first series of tests showed that there was great scatter, not only between the average expansion of cylinders, but also between the gauge lines on a particular cylinder. Attempts to relate the average expansions of cylinders with the measured surface strains of beams were only partially successful, Cope and Slade (1). The results suggested that there was a non-linear relationship between expansion due to AAR and stress level. Functions relating these quantities and an analytical procedure for predicting stress levels are described in an accompanying paper, May, et al (2).

In this paper, the average expansion of cylinders is referred to as the free expansion for the concrete in a beam. It should be recorded, however, that cracking of the beam and cylinder concretes was different in nature. All of the concrete in the relatively small cylinders was extensively cracked. In the beams, macro-cracking was found to be confined to an outer layer, the depth of which appeared to be independent of the beam size.

The mix itself provided a surprise, in that the cube strengths at the times of testing were up to about 50% higher than the twenty eight day values. This may have led to the involvement of near exhausted bond or bending capacities in the failures observed in a number of tests.

Data on the shear capacities of members with expansions up to 2.5mm/m was urgently required by the Forum. Members with simply supported ends, no shear reinforcement, no hooks or turn ups on main bars and with round, mild steel reinforcement (which was commonly used in pre 1973 bridge structures) were judged by the Forum to be most at risk.

OUTLINE OF TESTING PROGRAM

The testing program consisted of an examination of shear failure in beams, studies of dowel and aggregate interlock actions and a study of members with varying width over their shear spans. Here, attention is concentrated on the beam studies.

Most of the tests were on beams with a 125mm x 250mm cross-section, 1.48% mild steel soffit reinforcement and no shear reinforcement. These beams are in series 1. The parameters investigated included: free expansion of concrete; end detail - hooked or straight bars and overhang length; shear span to effective depth ratio; and position of reinforcement during casting. A small number of additional tests was conducted to explore the effects of: conditioning under load; repeated applications of loading; and sustained loading on shear capacity. Details of the beams and loadings are given in Figures 1 and 2.

Two series of larger beams, 200mm x 400mm cross-section, 1.35% mild steel soffit reinforcement and no shear reinforcement, were constructed for shear and dowel action studies. The purpose of the shear studies, Series 2 beams, was to determine whether the anticipated AAR effect on shear capacity would scale, or whether there was a size effect. For the dowel action studies, Series D specimens, part of the central, soffit portion of the beam was isolated from the rest of the member and loaded in the manner described by Taylor (3), in a study of dowel action in members with sound concrete. Details of these beams are given in Figures 1b and 3c.

Control specimens

Four ways of obtaining control specimens were considered: storing the members under water at 20°C; omitting the additional alkali salts and storing under water at 20°C, or at 38°C; and changing the fine aggregate to a non-reactive material. For most of the initial control specimens, the first option was adopted. Later, the second option was adopted. The change to the mix was made because of reports that the addition of alkali salts could reduce the compressive and tensile strengths of the concrete, Shayan et al (4).

Tests on cubes and cylinders. Tensile strengths were determined by splitting cylinders. Values of Young's modulus were determined from strain measurements for axial loading of cylinders to one third of their anticipated compressive strengths. The values recorded are the averages from three gauges, equally spaced around the circumference, and from three load cycles. It should be noted that values of Young's modulus determined in this way, the standard method for sound concrete, were dependent on the number of surface cracks crossing a gauge length. Sample cube and cylinder properties from different batches of concrete are shown in Table 1.

Series 1 : beam tests

Two, 16mm, round, mild steel bars were provided longitudinally, with 25mm cover. Transversely, on top of these bars, 9, 100mm long, 16mm tor-bars were placed at 300mm centres, to enable the beams to model strips of slabs with two way reinforcement. Lifting hooks were cast in, 750mm from each end. 25mm cover was provided to the main bars.

TABLE 1 - Sample cube and cylinder properties

Specimen reference	Alkali salts	Free expansion (mm/m)	fcu (N/mm2 (days))	ft (N/mm2 (days))	E (kN/mm2)
1A	Yes	0.11	45.5(28), 57.8(150)	4.1	
7	Yes	0	56.5(28), 77.0(200)	4.9	
AG1	No	-0.33	79.9	4.9	48.3
AG2	No	-0.05	77.2	4.6	44.8
AG10	Yes	1.16	71.6	3.3	16.9
AG9	Yes	1.77	72.6	3.5	19.7
AG5	Yes	1.79	72.5	3.1	
AG8	Yes	3.01	70.6	3.0	25.4
AG6	Yes	3.29	69.8	3.2	21.6
AG4	Yes	3.59	70.5	3.3	
AG7	Yes	4.03	67.7	2.3	18.6
AG3	Yes	4.14	69.4	3.0	19.2
13	Yes	5.35	54.0(28), 67.6(245)	3.4 (433)	
2B	Yes	6.05	53.5(28), 70.7(210)	3.8	

AAR cracking. Figure 3 contrasts crack patterns on beams conditioned with and without external loading applied. The crack pattern on the side of beam 9A shows little variation along the length of the member. The restraining effect of the reinforcement prevents cracking in the soffit region and biases the crack direction to the horizontal in the region immediately above the soffit. Away from the reinforcement, the crack directions appear to be random. The loading on beam 13A (see Figure 2b) has influenced cracking. In the unloaded end regions A, the pattern of cracking is similar to that of beam 9A. In zones B and C, the cracks tend to follow principal stress directions.

Figure 4 shows stress trajectories predicted by finite element analyses, using the non-linear AAR expansion limit-stress model proposed by May et al (2). For these analyses, the model used took the form of a cubic polynomial, with expansion stopped completely by a compressive stress of $4N/mm^2$. The stress trajectories shown in Figure 4a are for a beam conditioned with no external loading. The analysis predicts a relatively large, horizontal compressive stress in the concrete close to the reinforcement and small compressive stresses above the bars. The trajectories shown for beam 13A are first for the conditioning load, and then for the conditioning load modified by the AAR induced expansion. In the central section of beam 13A, the horizontal compressive stress has a more uniform distribution over the depth of the section. This is consistent with the cracking shown in Figure 3.

<u>Shear capacity</u>. Test results for the series 1 beams are summarised in Table 2. In Figure 5, the capacities of the beams are compared with predictions based on the BSI design code (5).

The vertical axis units of the graphs in Figure 5 are the moment of resistance divided by the beam dimensions. Horizontal lines towards the tops of the graphs indicate ultimate moment capacities based on the initial yield of the reinforcement, My, and the maximum sustainable steel stress Mf. The horizontal axis units give the shear span divided by the effective depth. With these units, the sloping line to the origin represents the code based reduction in ultimate moment due to shear failure. The lower horizontal line takes into account the code based enhancement to shear capacity for sections close to a support. The slope of the steeper line to the origin represents the effect of the maximum specified shear stress to prevent concrete crushing. The factors of safety have been set to unity when constructing the failure envelope.

Results for the control beams are shown in Figure 5a. It can be seen that the capacities were all in the range where yielding of reinforcement at the loaded section was predicted. Adding alkalis to the mix tended to reduce the capacity, but the effect is not significant.

Results for members conditioned with no external loading applied are shown in Figure 5b. As may be expected from shear tests, the results show considerable scatter. The trend indicated by the predicted failure envelope is followed. Within the range studied, there is no direct relationship between degree of free expansion and member capacity. Beams 9A and 9B were subjected to sustained loadings, see Figure 2c, before being loaded to failure. The effect of this has been to reduce the shear capacity for the tests with the shorter shear spans. Both tests on beam 9B resulted in bending failures.

Five sections of beams with reinforcement provided with end hooks were tested. Although the free expansions were in the highest range, none of the moments sustained was below the line based on first yield of reinforcement at the loaded section.

Results for members with reinforcement cast in the tops of beams are shown in Figure 5c. These beams were included to investigate the effect of the poorer compaction sometimes found in

TABLE 2 - Series 1 beam results

Beam	Alkali Salts	Free exp. (mm/m)	av/d	an (mm)	fcu (N/mm2)	Vult (N/mm2)	Comments
1A	Yes	0.11	3.0 3.0	250 250	57.8 57.8	1.69 1.63	Tested at BCA
1B	Yes	1.85	2.7 2.1	70 100	62.6 62.6	1.67 2.50	Damaged Tested at BCA
2A	Yes	2.52	0.9 0.9	100 100	70.7 70.7	4.12 6.54	Bond Shear
2B	Yes	6.05	3.0 2.1	100 100	70.7 70.7	1.88 2.35	Bending and shear
3A	Yes	1.09	2.7 2.7	150 160	75.6 75.6	2.20 1.98	
3B	Yes	1.03	3.0 3.0	100 100	75.6 75.6	1.94 1.88	Bending and shear
4A	Yes	-0.22	0.9 0.9	150 200	70.2 70.2	5.82 5.85	Bond Shear
4A1	Yes	-0.22	1.8 0.9	100 220	70.2 70.2	2.65 7.16	Bond
4B	Yes	1.22	2.7 3.0	70 100	73 73	2.04 1.93	Bond
5A	Yes	1.39	3.0 3.1	250 220	72.7	1.49 1.58	Top cast steel
5B	Yes	1.81	3.0 3.0	100 100	72.7	2.12 1.93	Top cast steel
6A	Yes	1.97	1.8 1.8	100 100	72.7	1.98 1.78	Top cast steel
6B	Yes	4.78	1.8 2.1	100 100	72.7	2.26 1.74	Top cast steel
7A	Yes	0.09	2.1	250	76.9	2.57	
7B	Yes	0	2.1 1.8	100 31	76.9	2.86 2.71	Bond
11A	No	-0.30	0.9 2.7	200 210	72.9	7.77 2.30	
11B	No	-0.39	3.0 1.8	145 200	72.9	1.82 3.27	
12A	Yes	4.61	0.9 1.8	185 145	72.4	4.22 2.50	Conditioned under load
12B	Yes	5.01	1.8 2.7	200 210	72.4	2.61 2.14	Conditioned under load
13A	Yes	6.06	1.8	290	68.6	2.82	Conditioned under load
13B	Yes	4.63	3.0 1.8	120 350	66.6	1.78 2.79	Conditioned under load
14A	Yes	5.86	3.0 1.8 2.1	250 150 150	57.6	1.72 3.29 3.08	With end hooks
14B	Yes	6.15	1.8 0.9	145 150	57.6	2.82 5.91	With end hooks
15A	Yes	4.40	1.8 0.9	150 150	65.1	2.62 4.73	Top cast steel
15B	Yes	4.19	1.8 2.1	150 150	65.1	2.89 2.24	Top cast steel

hogging regions of continuous members. The results suggest that sections in such regions are likely to have a lower capacity than those with bottom cast reinforcement. However, all of the results are above the code based failure envelope.

Results for members conditioned under load are shown in Figure 5d. Details of the conditioning load arrangement are shown in Figure 2b. Although this loading induced AAR cracking parallel to likely shear failure planes, see Figure 3b, the capacity of sections does not appear to have been reduced. The lowest test value shown is for a beam which had also been subjected to repeated applications of loading, before the test to failure.

Size effect. The larger beams were approximately 1.6 scale versions of the smaller beams. The test results for these beams suggest that AAR had a lesser effect than on the smaller beams. Examination of cores and cross-sections suggested that penetration of AAR induced surface cracking is limited to a surface zone with a depth of about 25mm. This is probably the cause of the relatively greater effect of AAR on the unit shear capacity of the smaller members.

CONCLUSIONS

The differential expansions of the test beams produced macro-cracking in a thin outer shell and micro-cracking in the core. The AAR and crack patterns were strongly influenced by prevailing stress conditions, including those produced by restraint to the expansion. The test data provided suggest that AAR, perhaps by prestressing the concrete, enhances shear capacity. The degree and distribution of prestress in a member cannot be determined reliably with current knowledge. Use of current code procedures and formulae for reinforced concrete, gave conservative estimates of failure. It is, therefore, suggested that this well understood and simple method be used for assessment purposes of bridge standard concrete, unless members show particular signs of weakness.

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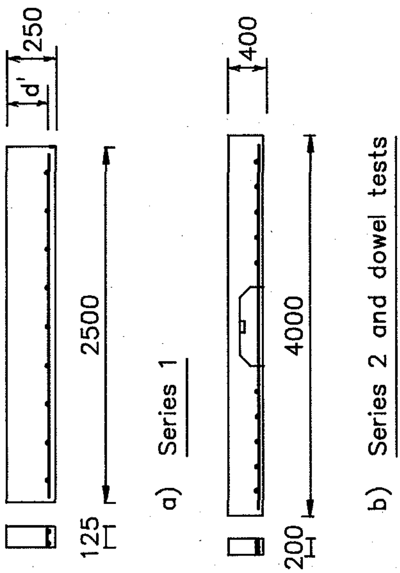


Figure 1 Beam details

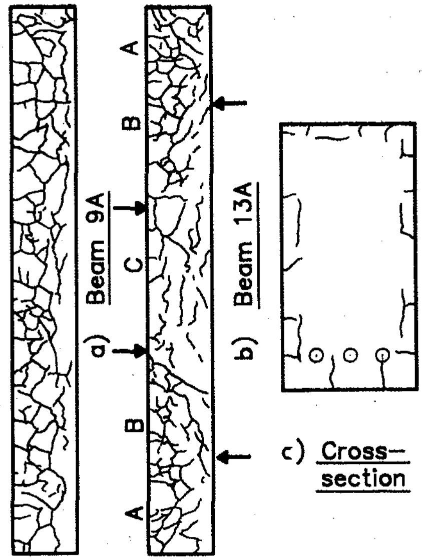


Figure 3 AAR cracking

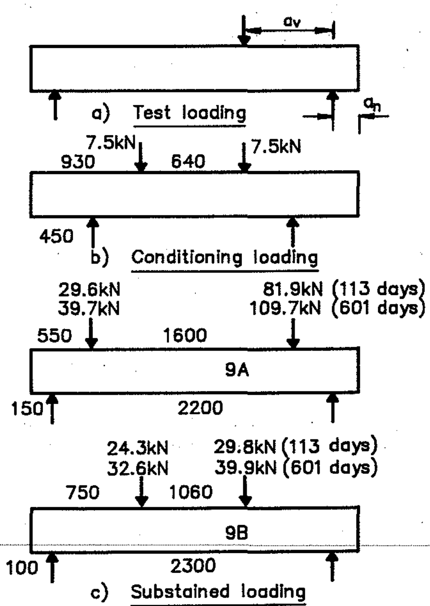


Figure 2 Loading arrangements

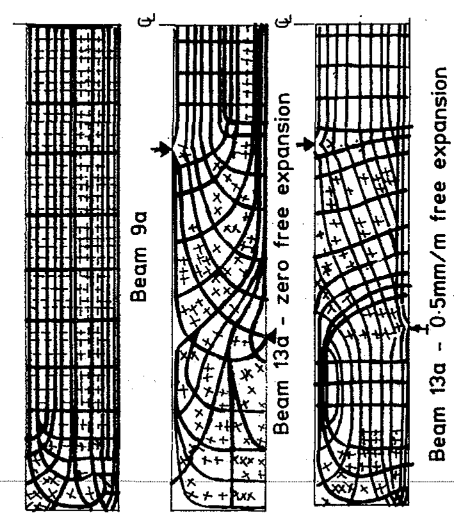
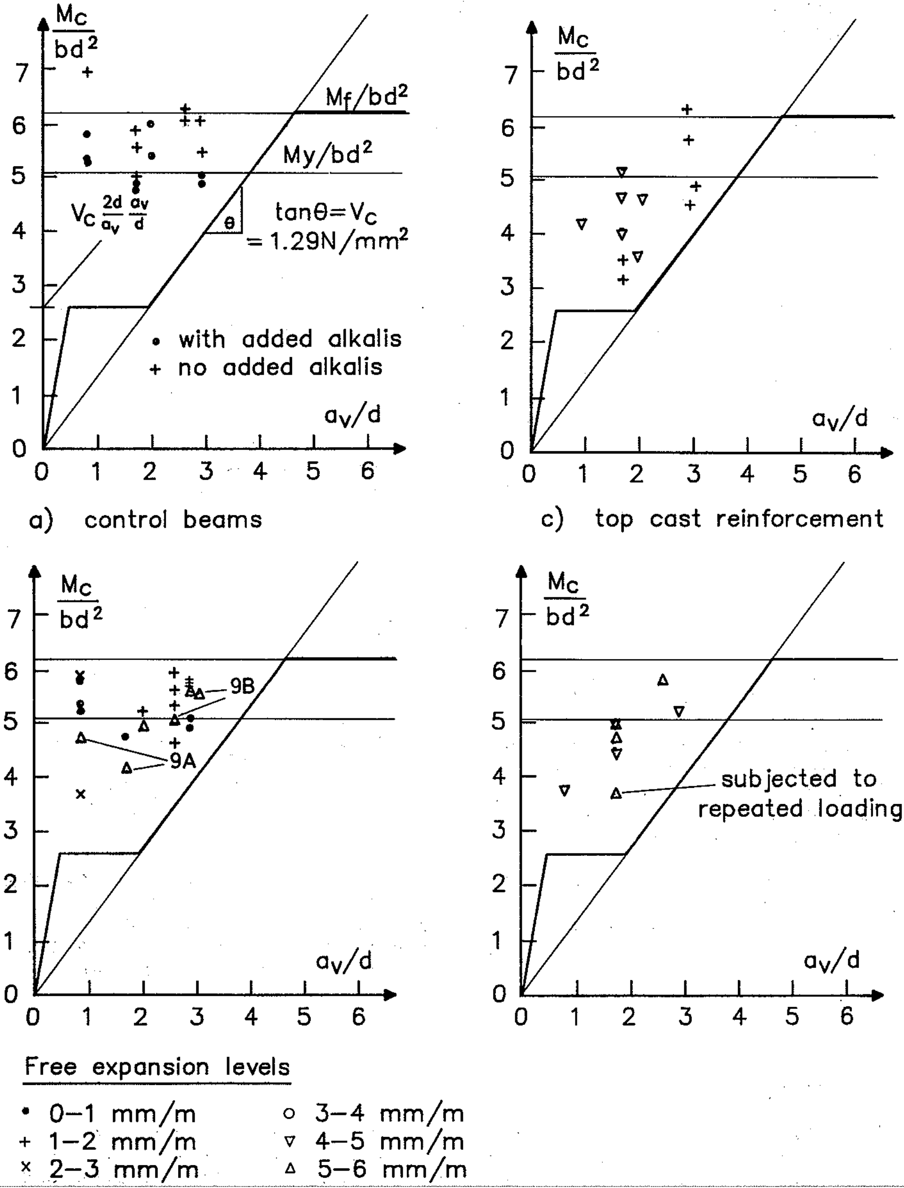


Figure 4 Predicted stress trajectories



b) no conditionig load

d) conditioned under load

Figure 5 Comparison of shear capacities