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PROPERTIES OF AAR-AFFECTED CONCRETE STUDIED OVER 20 YEARS

G E Blight and Y Ballim
University of the Witwatersrand, Johannesburg
Private Bag 3
WITS 2050
South Africa

ABSTRACT

In 1978, cracking of a motorway portal structure was identified to be the result of AAR. At this time the structure was 15 years old. The structure was monitored continuously over the following 13 years and was twice proof tested to establish its state of safety. Finally, the upper beam of the portal was demolished and reconstructed in 1991. Prior to demolition, it had been established that expansion of the concrete of the beam was continuing, even 28 years after construction. Five and a half years after construction, strain measurements showed that expansion was still continuing in the concrete that had not been replaced.

Prior to demolishing the portal beam in 1991, cores had been taken and subjected to elastic modulus measurements as well as free swell, swell-under-load and creep tests. Seven years later, in 1998 the same cores were retested. The complete set of elastic modulus, swell, swell pressure and creep data will be presented and analysed to show how consistent the properties of the concrete remained over the 20 year period of investigation, and to illustrate that the potential for swelling caused by AAR may continue for more than 35 years after construction.

Keywords: Alkali-aggregate reaction, reinforced concrete, portal frame, long term behaviour.

INTRODUCTION

Johannesburg, South Africa, lies at an altitude of 1,800 m at 26° south latitude. It has a semi-arid climate with average annual rainfall of 750 mm and A pan evaporation of 1,600 mm. The rain falls in summer and there is an excess of evaporation over rainfall throughout the seasons. Minimum temperatures fall to 0° and maximum temperatures are 28 to 30° C.

In 1978 it was discovered that a series of reinforced concrete motorway structures that had been completed between 1963 and 1965 were showing signs of severe cracking. The investigation (reported to the 5th ICAAR in 1981, Blight et al.) concluded that the cause of the cracking was AAR. Although the cracking was widespread and affected a variety of types and components of structures, the damage to one structure was of particular concern. This was a two-level portal frame that had suffered severe cracking of the upper beam, a cantilever nib supporting longitudinal bridge beams, and both columns. An elevation of the structure is shown in Fig. 1 which also records the location of the cracking and the dates and types of remedial measures carried out between 1981 and 1991. Unfortunately, the construction records had been lost by 1978. However, an analysis of cores showed that the fine and coarse aggregate in the concrete was quartzite. The content of ordinary portland cement was 330kg/m^3 and the equivalent Na_2O content was 8.6kg/m^3 (2.6 per cent of cement) (Blight et al. 1981).

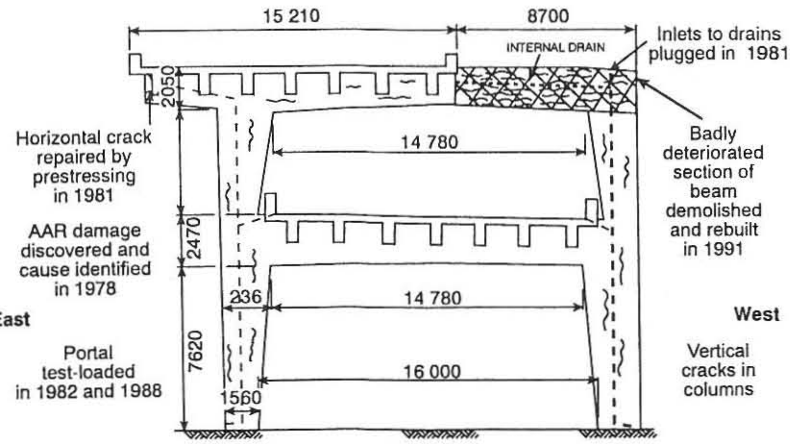

Fig. 1: Main dimensions (mm) of the two-level portal frame showing various repair procedures and tests (1981 to 1991) (elevation looking south)

The structure was monitored continuously for the next 14 years and was twice load tested to its static design load (Blight et al. 1983; Blight et al. 1989). Despite the fact that these proof tests showed very consistent results and proved that the portal's structural integrity remained unimpaired, in 1991 the owners decided to demolish and rebuild the deteriorated upper beam. Fig. 2 is a 1982 photograph of the western haunch of the portal showing the appearance of the concrete. (The hole visible in the concrete was cut to expose the reinforcing for strain gauging.)

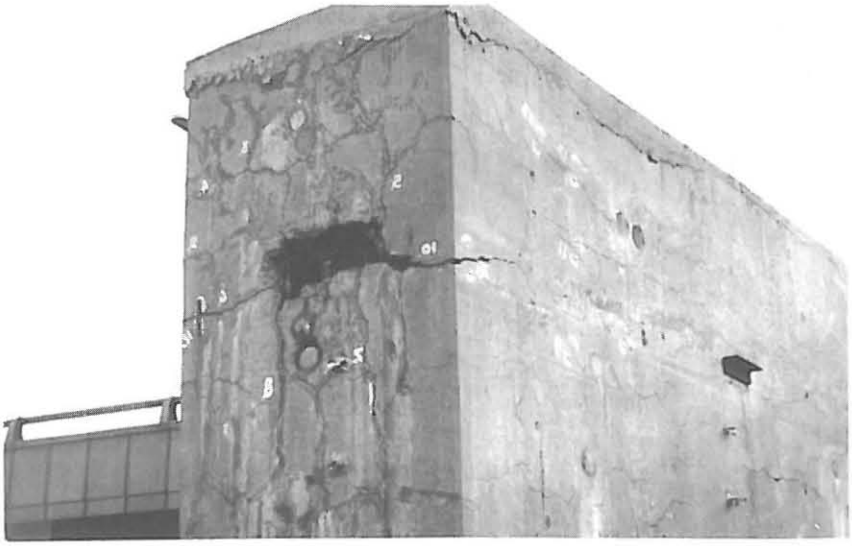

Fig. 2: Deterioration of concrete in portal as result of AAR

In situ tests performed shortly before the demolition (Blight 1991) had shown that concrete composing the portal was still expanding. Prior to demolition, several cores were taken from the beam and subjected to laboratory testing (Alexander et al. 1992). These tests included elastic modulus measurements as well as free swell, swell-under-load and creep tests.

Five and a half years after reconstruction of the portal beam, strain measurements showed that expansion was still continuing in the original concrete that had not been replaced in the reconstruction (Blight and Lampacher 1998). Seven years after the tests on the pre-demolition samples (in 1998/99) the same cores were retested, and this paper will report the results of these tests.

A SUMMARY OF PRE-DEMOLITION TEST RESULTS

Swelling and Swell-Under-Load

Fig. 3 shows strains measured on the surface of the portal beam during 1990 (Alexander et al. 1992) when the portal was 25 years old. An attempt had been made to seal the surface of the beam by means of a cementitious coating to prevent the ingress of water. The coating was applied in April 1990, at the end of the wet season. As a result, moisture was sealed into the concrete which almost immediately started to swell. Swelling in some areas exceeded 800 microstrain seven months later and the "water proof" coating had cracked extensively.

Recent in situ measurements on the portal beam (Blight and Lampacher 1998), revealed swelling strains approaching 1,000 micro-strain in the old concrete where it abuts the reconstructed portal beam. Moisture appears to have been sealed into the old concrete by the covering of new concrete. The old concrete has swelled whereas the new has shrunk. The results of these measurements are shown in Fig. 4. These (temperature corrected) swelling strains are likely to damage the repaired beam, if they continue to increase.

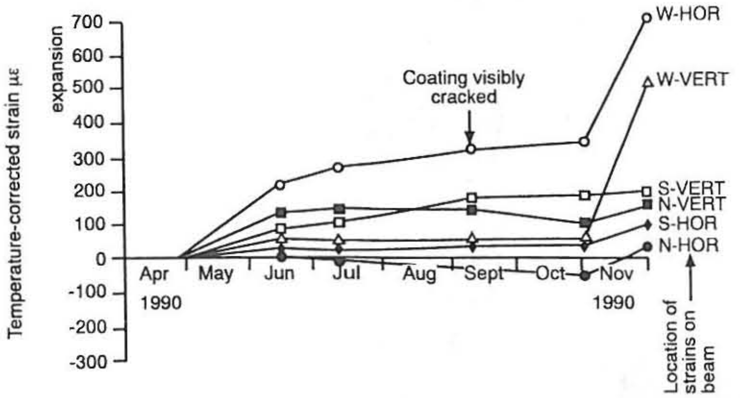

Fig. 3: Measured expansion of the surface of a 25 year old R.C. portal beam

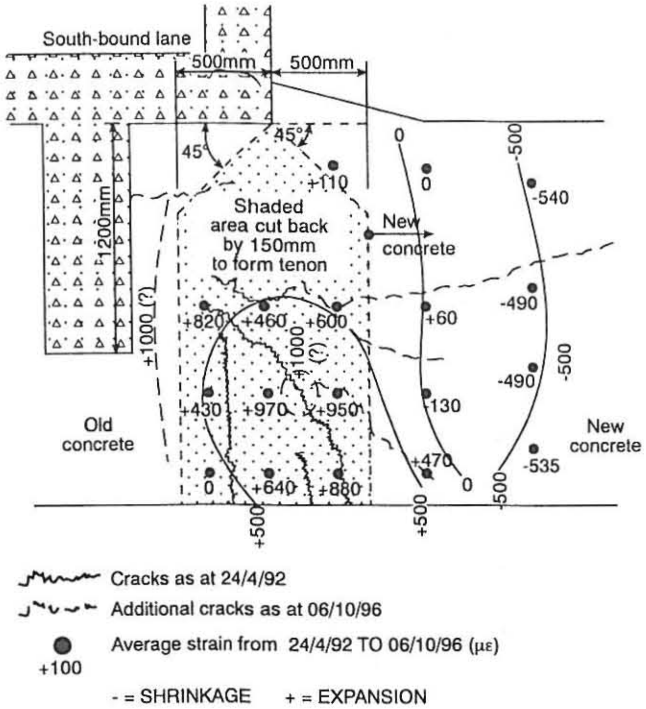

Fig. 4: Average strains and cracking recorded 5½ years after repairing the portal beam

The results of free swell and swell-under-load tests on the pre-demolition cores are shown in Fig. 5. Free swells exceeding 450 x 10^{-6} over a period of 130 days were observed. These were of the same order as the in situ surface strains shown in Fig. 3, four months after sealing the surface of the portal beam. Field and laboratory swell measurements were therefore consistent.

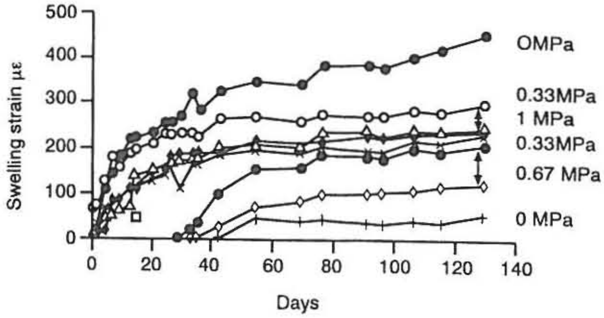

Fig. 5: Swell-under-load versus time relationships for pre-demolition cores of 25 year old concrete

Elasticity and Creep Properties

The pre-demolition tests (Alexander et al. 1992) had included measurements of the static elastic modulus of the concrete at different depths through the thickness of the portal beam. These results are shown in Figure 6, where the modulus values have been plotted at the positions occupied in the beam by the respective cores. The horizontal dashed lines in the figure mark two standard deviations from the mean value.

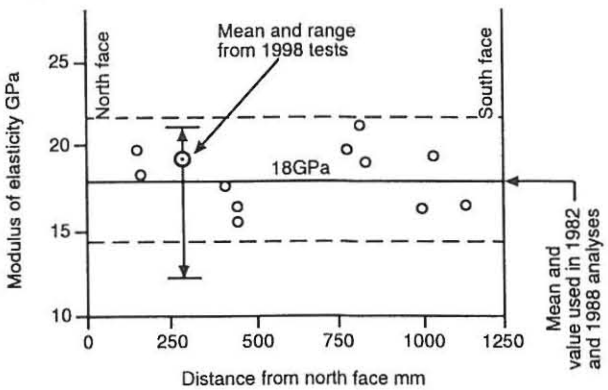

Fig. 6: Modulus of elasticity measurements on pre-demolition cores from portal beam

Probably the best measurement of the elastic modulus of the portal concrete can be derived from the results of the two full-scale load tests. Based on these, the "best" overall value of the static elastic modulus is 18 MPa which coincides with the mean value of the core tests.

One of the early (1981) repairs to the portal consisted of applying a prestress to close a horizontal crack and provide support to a cantilever nib on the upper eastern side of the portal (see Fig. 1). The applications of prestress in the repair were designed on the basis of laboratory creep tests on cores taken from the nib. Figure 7 shows the results of the laboratory creep tests superimposed on the field creep values established by monitoring strains in the prestressing rods restraining the nib (Blight 1990).

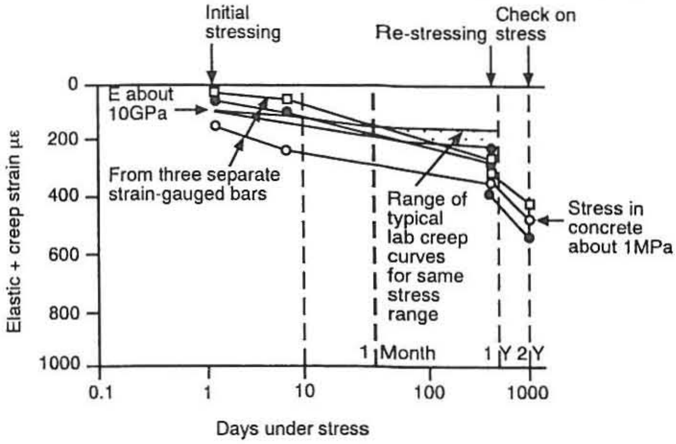

Fig. 7: Field creep curves measured on cantilever nib repaired by prestressing

SECOND SERIES OF TESTS ON PRE-DEMOLITION CORES

After the pre-demolition cores had been used for the free swell and swell-under-load tests, they were stored in a drying environment in the laboratory until 1998, at which time they were approximately 33 years old. The cores were then subjected to the following tests:

Static Elastic Modulus

The cores were tested for static elastic modulus in their air-dry condition and again after they had been soaked in a water bath for 18 days.

Eight cores were tested, four in dry and four in wet conditions. The average "dry" elastic modulus (see Fig. 6) was 19.0 GPa (range 12.0 to 20.9) and the average "wet" elastic modulus was 13.5 GPa (range 12.3 to 16.3). The elastic modulus of the concrete was significantly reduced by re-wetting. Normally the static elastic modulus of wet concrete is higher than that of the same concrete tested dry (e.g. Neville, 1981). This indicates that the effect of moisture content in reducing the elastic modulus of the AAR-affected concrete results from the presence of reaction products in the concrete. The dry elastic modulus values agree very well with the values shown in Fig. 5.

Free Swell Upon Re-wetting

The cores were placed in a water tank controlled at 23°C. Strains were then regularly measured up to 18 days.

To provide a basis for comparison, a set of air-dried, two year old 100 x 100 x 200 mm prisms, made with nominal 30 MPa concrete containing quartzite aggregate, were also monitored for free swell upon re-wetting.

The results of the swell measurements are shown in Fig. 8. The AAR-affected cores showed 18-day swelling strains from 325 to 510 microstrain. However, only one of the samples showed particularly high swelling strains and, after six days, the average of the results for the AAR cores was only 16% higher than the swelling strain of the sound concrete.

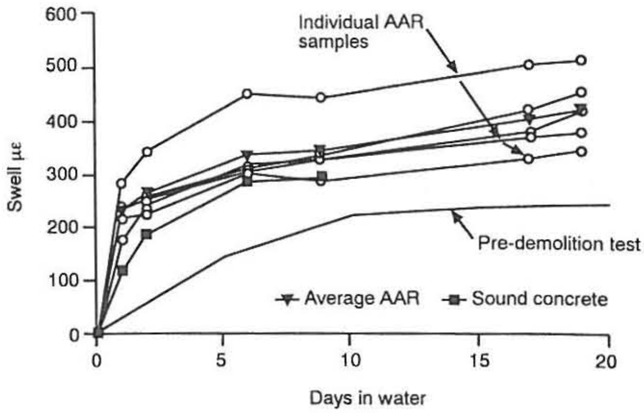

Fig. 8: Free swell of cores in 1998 tests compared with pre-demolition test (Fig.4)

After 18 days of swelling, the 1998 free-swell strains of AAR-affected cores were almost double those obtained in the pre-demolition test when the concrete was 25 years old (see Fig. 5). However, two of the cores were left in the water tank for 453 days, when the swelling strain was $1{,}004 \times 10^{-6}$. A linear interpolation to 125 days indicates that swelling strains in the present tests were quite similar to those obtained in the 1991 tests.

Fig. 8 also shows that the initial rate of swell of the AAR-affected cores was considerably higher than that of the sound concrete (only nine days of measurements). This is probably because of a higher rate of uptake of water by the products of AAR. For the sound concrete, expansion resulted from re-absorption of water by the c-s-h gel, which is less permeable at the micro-level and less fractured at the macro level than for the AAR-affected concrete.

Creep of Dry and Soaked Cores

After the free-swell measurements, three of the cores were coated with a bitumen emulsion and then wrapped in aluminium foil. This allowed the creep behaviour of the concrete to be determined without drying. Three other cores were tested unsealed and air-dry.

Fig. 9 shows the results of the creep tests. The figure also shows creep results for the 1980/81 tests (Fig. 7) and for an unsealed 30 MPa quartzite concrete.

The AAR-affected samples loaded dry show lower creep strains than the control concrete, because of the considerably later age at loading and the absence of a drying creep component. These samples showed very similar creep strains to those in Fig. 7, although the specific creep (microstrain/MPa) was slightly lower.

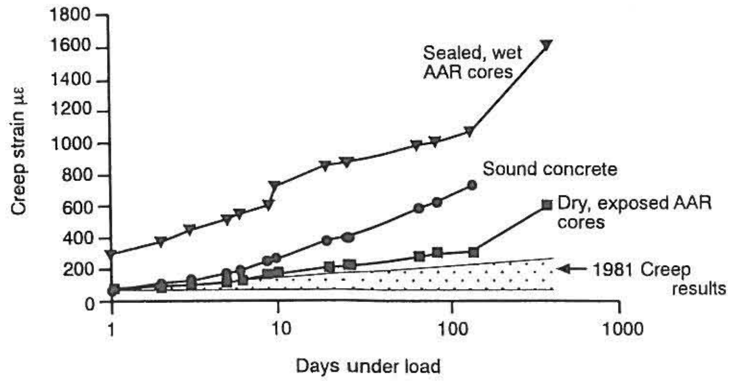

Fig. 9: Comparison of 1998 and 1981 (Figure 7) creep results

The influence of moisture content is demonstrated by the creep of the wet cores. In this case, the specific creep at one year was three times that of the dry cores.

Optical Microscopy

A thin section was prepared from a disc cut from one of the cores for optical examination using a geological microscope. This was aimed at qualitatively assessing the nature and extent of AAR damage. In preparing the thin section, a blue dye was added to the impregnation epoxy to highlight cracks and discontinuities in the concrete. The section showed extensive networks of cracks occurring along the cement-aggregate interface, passing through aggregate particles and bridging through the cement paste matrix from one aggregate particle to another. AAR gel deposits and ettringite were visible throughout the section. Simultaneous occurrence of both gel and ettringite in AAR-affected concrete has been noted previously by Clark et al. (1992). Fig. 10 shows typical deposits of gel at the interface between cement paste and aggregate, while Figure 11 shows ettringite deposited at the aggregate/cement paste interface.

CONCLUDING SUMMARY

The 35 year-old concrete studied over the past 20 years has shown consistent characteristics, which have changed very little since the study began. The concrete has continued to swell when wetted or given access to water (Figs. 3, 4, 5 and 8) and swell strains measured in the field have been consistent with those measured on cores. Elastic moduli have changed very little and moduli for air dry cores are consistent with moduli established by means of full scale loading tests (Fig. 6). Similarly, creep properties established for cores 18 years ago, and which were shown to be consistent with creep measured in situ (Fig. 7) are very similar to creep properties recently established for cores (Fig. 9).

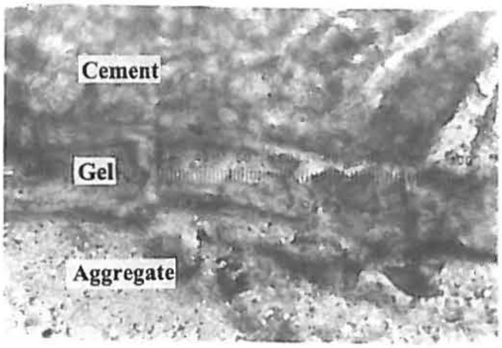

Fig. 10: AAR gel deposit at the interface between cement paste and aggregate (1 division = $1\mu m$)

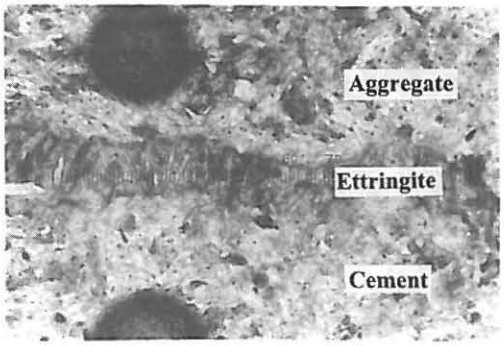

Fig. 11: Ettringite deposited at the aggregate/cement paste interface (1 division = $2\mu m$)

The results also indicate that progressive drying of an AAR-affected structure should be an important objective of any planned repair strategy (see also Blight 1991). This will have the benefit of limiting swelling caused by the AAR as well as limiting elastic and long term deflections.

Optical microscopy studies have shown that this AAR-affected concrete contains both AAR gel and ettringite (Figs. 10 and 11).

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