PRE-DEMOLITION TESTS ON STRUCTURAL CONCRETE DAMAGED BY AAR

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A reinforced concrete double storey portal frame that was damaged by AAR was recently rehabilitated by demolishing part of the upper beam and reconstructing it. After outlining the history of the portal and the various investigations made on it in the past 13 years, the paper describes the results of a series of pre-demolition tests carried out on the concrete.

INTRODUCTION - THE HISTORY OF THE CONCRETE

In 1977 it was discovered that several of the reinforced concrete portal frames supporting an overhead section of the Johannesburg urban motorway system were severely cracked (1). An examination of cores taken from the damaged structures, as well as the form of the crack patterns, resulted in a diagnosis of alkali aggregate reaction as the cause of the damages. One portal (number C15), was particularly badly damaged and became the subject of intense investigation. Figure 1 shows the dimensions of portal C15 and records the various tests and remedial measures carried out in the period of 13 years that the portal was under investigation.

The concrete is made of Witwatersrand quartzite fine and coarse aggregate. This quartzite is a mine waste rock that has been brought to the surface from depths of 1000 to 3000m. An analysis of the rock showed that its equivalent Na_2O content varies between 0.1 and 0.7 percent. Analysis of the AAR-affected concrete, allowing for the alkali content of the aggregate, indicated that the equivalent Na_2O content of the cement varied between 0.3 and 2.6 percent with a mean for 13 analyses of 1.3 percent and a standard deviation of 0.8 percent. As the concrete was 15 years old when the damage was discovered, and the construction records had been destroyed, the source of the abnormally high alkali cement has never been discovered.

Full-scale Load Tests

Because of concern for the structural safety of portal C15, a full-scale load test was carried out in 1982 (2). Part of the preparation for the test consisted of an elastic finite element analysis to predict the behaviour of the structure under load. The elastic modulus used in the analysis was based on laboratory measurements on cores taken from the structure, and

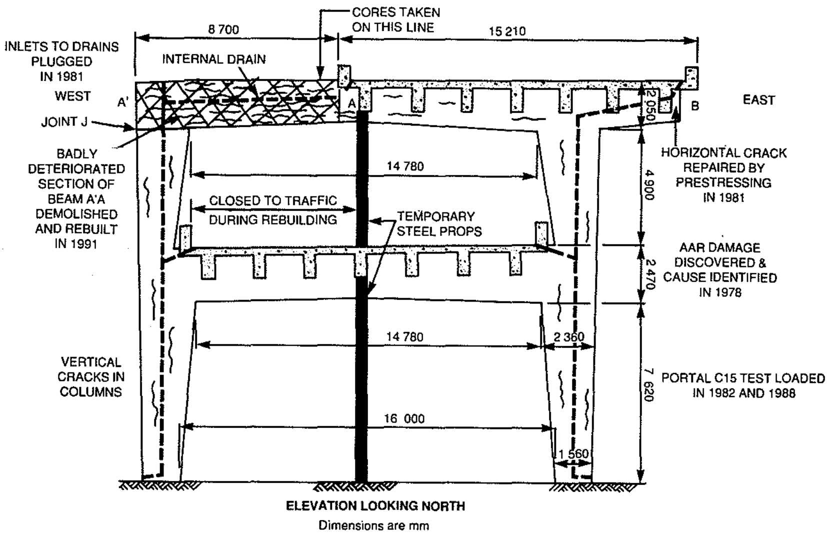

FIGURE 1 Portal C15 - principal dimensions, test history and method of rehabilitation

was determined at 18 GPa. It was concluded from the test that the appearance of the deterioration was more alarming than serious. The structure was at that time fully capable of carrying its design load and behaved predictably and elastically under load.

Continuing measurements of crack widths showed that the AAR expansion was proceeding in the portal with ever-widening cracks, as well as the appearance of new cracks. A second full-scale loadtest was carried out in 1988 (3). The results were very much the same as those of the test carried out 6 years earlier. Figure 2 shows the predicted and observed relationships between midspan deflection of the upper beam of the portal and load. The load is expressed as a percentage of South African NA loading (125 percent of NA load corresponds to about 100 percent of British BS153 HA loading). Predictions were made on the two assumptions that (a) full moment continuity had been retained at the upper left hand (or west) joint of the portal (J in Figure 1) and (b) that continuity had been lost. The figure shows that the portal performed better than the prediction assuming full continuity at J. It also shows how well the results of the 1982 and 1988 load tests agreed, and how close to elastic was the behaviour of the portal under load. The actual modulus of elasticity of the portal was about 24 GPa, as compared with 18 GPa assessed from tests on cores.

Moisture Measurements

A series of measurements was started in April of 1989 to find if it would be possible to dry out the beam of the portal during the dry season and then coat it with a waterproof or water repellent preparation. In this way it was hoped to slow down, or even stop the AAR. Psychrometer moisture sensors were embedded in the concrete and a series of measurements was made over the next 19 months (4,5). The measurements showed that it is very difficult to control moisture conditions in concrete structures exposed to the elements and that short of jacketing the portal in a ventilated metal tube, it would be almost impossible to dry it out permanently.

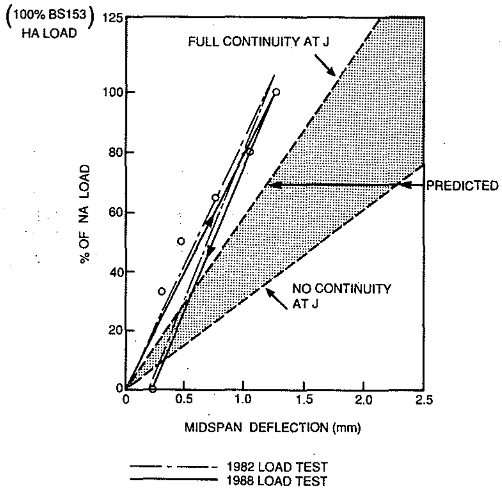

FIGURE 2 Typical results recorded in two full scale load tests on Portal C15

Continued Swelling of the Concrete

Although crack widths had been measured on the structure since cracking was first discovered, no-one thought to measure swelling strains until April 1990, when the concrete was 25 years old. The measurements were carried out with a 400 mm Demec gauge and were corrected for temperature by means of thermocouples embedded in the portal. The results, shown in Figure 3, indicated that the concrete was still expanding (5). The expansion had probably been re-activated when the owners of the structure sealed its surface while the concrete was wet (4) thus sealing in the moisture and preventing the concrete from drying out.

The cause of the continuing expansion was queried by Hobbs (6) who suggested that delayed ettringite formation (7) might be responsible. However a careful examination of concrete from the portal was unable to find ettringite or the structures and crystal growth patterns associated with delayed ettringite formation.

Other Remedial Measures

While the moisture condition of the portal was being studied, other more drastic measures were also under consideration. These included

(a) Demolishing length A'A of the upper beam (see Figure 1) and reconstructing it in reinforced concrete exactly as before, except that low alkali cement and non-reactive aggregate would be used.

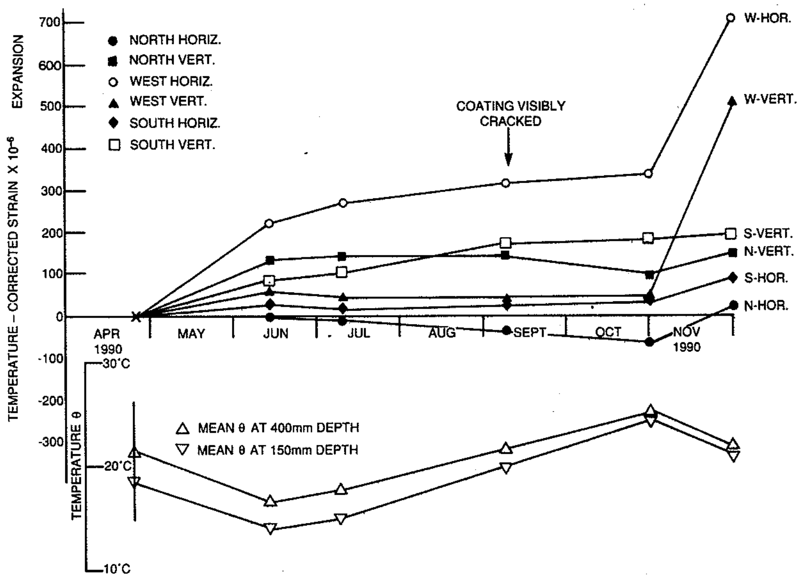

FIGURE 3 **Measured expansion on beam of Portal C15 recorded shortly before demolition**

(b) Augmenting the strength of A'A by means of bolt-on steel splints; and

(c) Replacing A'A with a bolt-on steel beam.

Eventually it was decided to adopt course (a) and to demolish and rebuild A'A in reinforced concrete. The decision was not taken for purely technical reasons -it being accepted that the structure was adequately safe and would continue to be so for many years to come. It was basically a management decision based on a reluctance to appear indecisive about a solution to the problem, as well as a reluctance to continue indefinitely with a programme of monitoring.

PRE-DEMOLITION OBSERVATIONS

Prior to defining the extent of demolition of beam A'A, tests were made to establish the properties of the existing concrete at the proposed junction of the new and old sections. Three horizontal cores were taken on the vertical line indicated in Figure 1. The cores penetrated the full 1240mm thickness of the beam and were drilled at roughly 500mm vertical intervals.

Compressive Strength

The cores were divided into suitable lengths for testing and one piece from each core was tested for compressive strength. Considering the results of earlier tests, it was not surprising to find a mean compressive strength of 31 MPa, whereas the design strength had been 4500 psi or 31 MPa. Although the concrete showed signs of deterioration, the cores appeared to be in better condition than some of the concrete in A'A.

Ultrasonic Pulse Velocities

A measurement of ultrasonic pulse velocity (UPV) was made on each piece of core prior to further testing. The ultrasound pulse was transmitted along the axis of the core in each case, thus simulating horizontal transmission through the thickness of the beam.

The results of the measurements are shown in Figure 4(a) where they are compared with the mean UPV established for the better quality concrete in beam A'A during the 1988 tests (3). As the figure shows, the mean UPV of 4.09 km/s measured on the cores was very close to the mean UPV of 4.07 km/s measured in situ in 1988, prior to the second full-scale loading test.

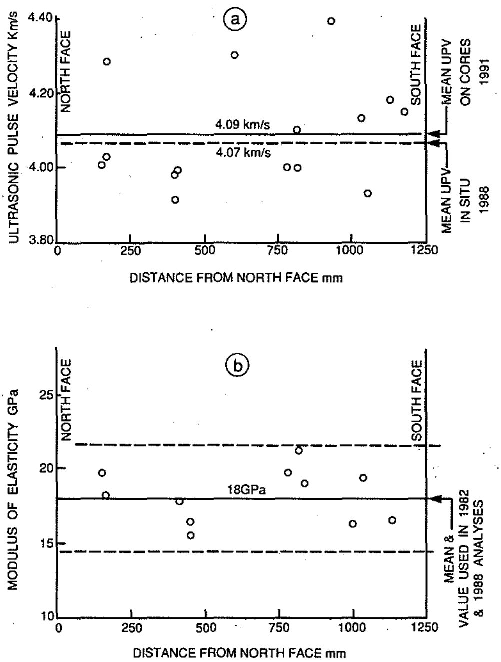

FIGURE 4 (a) Ultrasonic pulse velocity and (b) modulus of elasticity measurement on cores from Portal C15

Figure 4(a) shows the measured UPVs plotted against the position of the cores in the thickness of the beam. There appears to be no particular pattern in the measurements, no indication, for example, that concrete from the heart of the member had deteriorated less than concrete from close to the surface.

Modulus of Elasticity

The static modulus of elasticity was also measured on the cores. These results are plotted in Figure 4. In considering Figure 4, it must be remembered that the measurements represent moduli measured in what would have been a horizontal transverse direction in situ.

Individual measurements ranged from 15.5 GPa to 21.1 GPa with a mean value of 18.1 GPa. This is close to the value of elastic modulus of 18 GPa chosen in 1982 and again in 1988 for the finite element analyses and shown by the loading tests to be reasonably representative of the structure's behaviour (although the elastic modulus for the structure exceeded 18 GPa). Here again, there is no indication that the heart concrete was of any better quality than concrete from closer to the surface of the member.

Swell-Under-Load

Some of the cores were subjected to swell-under-load tests to assess if the concrete was still expansive, and if so, to estimate the residual swelling pressure.

The cores were mounted in modified soil testing oedometers which enabled a constant axial stress to be applied via dead weights and a lever system. The axial strain of the specimens could be measured by means of dial gauges reading to 0.001mm, enabling a strain of 5 x 10^{-6} or 5 $\mu\varepsilon$ to be resolved on a specimen 200mm long. The specimens were enclosed in a jacket over distilled water, and the tests were carried out at a constant temperature of 23°C.

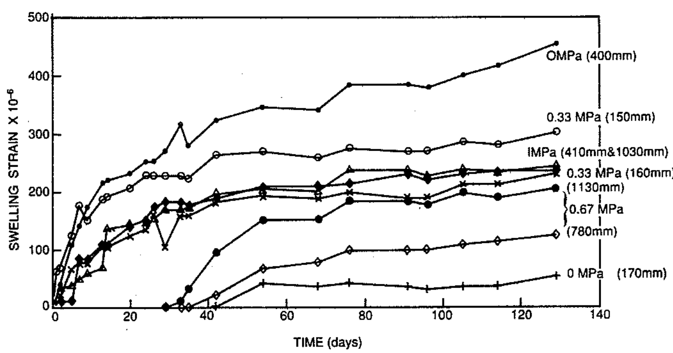

FIGURE 5 Swell under load versus time relationships for cores taken from Portal C15

Figure 5 shows the observed relationships between swell-under-load and time. The constant stress applied to each specimen is indicated next to each curve, and so is the distance from the north face of the beam at which the core was taken. It will be seen that free swells of over 450 $\mu\varepsilon$ were recorded and that strains of close to 250 $\mu\varepsilon$ occurred under a confining stress of 1 MPa. Swelling strains over a period of 4 months were of the same order as those observed previously over a similar period on the surface of the beam (Figure 3).

Figure 6 shows the approximate equilibrium swell strains plotted against the corresponding confining stresses. Extrapolating the trend lines of swell versus stress to zero swell strain gives an indication that the fully restrained swelling pressure of the concrete was in the range of 1 to 2 MPa, even after 25 years of exposure to the elements. Again, there is no indication that the samples from closer to the axis of the beam were more expansive than samples from close to the surface.

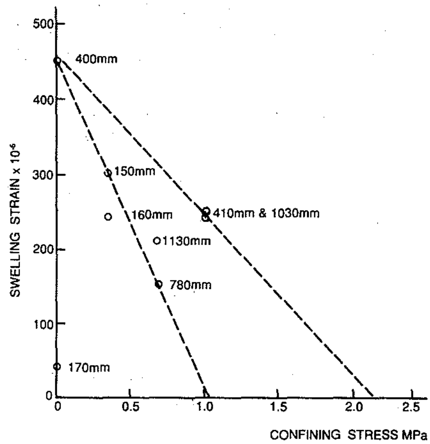

FIGURE 6 Relationships between equilibrium swell strain and confining stress for cores from Portal C15

CONCLUSIONS

The results of the pre-demolition tests have largely confirmed the conclusions reached on the basis of earlier laboratory tests on cores from the structure and the two full-scale load tests:

The strength of the concrete generally remained above the design strength of 31 MPa. This conclusion had been reached in 1981 (1) and was confirmed a number of times in later years. There were, however, zones of concrete where the strength fell below 31 MPa. Even in the pre-1981 tests, the strengths of certain cores were as low as 12 MPa.

Ultrasonic pulse velocities have been used by the authors primarily as an indication of the degree of internal disruption of the concrete. The agreement between UPVs measured on cores and the in situ measurements made in 1988 supports the view that this is a valid and repeatable procedure. However, the in situ measurements made in 1982 and 1988 could not be reconciled, for reasons that remain unknown.

The agreement between elastic moduli measured on cores and the value that could be deduced from the full-scale loading tests shows that the properties of the concrete were reasonably isotropic and non-directional. It also demonstrates that valid predictions of the behaviour of full scale reinforced concrete structures can be made on the basis of engineering properties established by sampling from the structure. This is a most important conclusion, for it shows that the behaviour of AAR-affected structures can be predicted on the basis of properties measured on sample cores.

The measurements shown in Figure 3 indicated that the concrete was still swelling in situ shortly before beam A'A was demolished. Although this swell had probably been re-activated by sealing the surface of the beam at a time when the concrete was wet, it showed that relatively old concrete may retain a swell potential.

This observation has been confirmed by the results of the swelling tests shown in Figures 5 and 6. Given access to moisture, the concrete is not only capable of swelling by up to 500 $\mu\varepsilon$, but can exert fully restrained swelling pressures of between 1 and 2 MPa.

Finally, it appears that the whole volume of concrete comprising beam A'A was more or less equally affected by AAR. There do not seem to be any gradients of properties that would indicate a progression of the reaction from the outside of the concrete towards the axis. However, this may be because during the relatively long period for which the concrete has been subjected to AAR, these gradients have formed and disappeared.

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