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THE EFFECTS OF ALKALI-AGGREGATE REACTION ON REINFORCED CONCRETE STRUCTURES MADE WITH WITWATERSRAND QUARTZITE AGGREGATE

by Prof G E Blight*, Prof JR McIver*, Mr W K Schutte** and Mr R Rimmer***

SYNOPSIS

The paper describes a comprehensive system that detects, assesses and remedies the effects of the alkali-aggregate reaction on reinforced concrete structures.

Both petrographic examination and ultra-sonic pulse testing are used to detect and assess the extent of deterioration. The effects of deterioration on the strength of a structure are assessed by strength and deformation tests on cores taken from affected portions of the structure and (if necessary) remedial measures are designed in the light of the test results. Experience of the effects of alkali-aggregate reaction on the strength, elastic and creep moduli of concrete will also be described.

SAMEVATTING

Die referaat beskryf 'n omvattende stelsel wat die uitwerking van die alkali-aggregaatreaksie op gewapende betonstrukture opspoor, na waarde bepaal en herstel.

Petrografiese ondersoeke sowel as ultrasoniese pulstoetse word gebruik om die omvang van verswakking op te spoor en vas te stel. Die uitwerking van verswakking op die sterkte van 'n struktuur word bepaal volgens sterkte- en vervormingstoetse op kerne wat uit aangetaste gedeeltes van die struktuur geneem is en herstelmaatreëls word (indien nodig) in die lig van die toetsresultate beplan. Ondervinding in verband met die uitwerking van alkali-aggregaatreaksie op die sterkte, elastisiteit en kruipmoduli van beton sal ook beskryf word.

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1. INTRODUCTION

Until relatively recently the problem of alkali-aggregate reaction was thought not to occur in Southern Africa. Fulton[6], as late as 1976, wrote: 'in South Africa the only known cases have arisen when glass has been used as an aggregate, or in the form of insets in decorative concrete'. It has recently become evident that alkali-aggregate reaction has been responsible for damaging a number of bridge and retaining structures in the south western Cape[12]. Even more recently, a number of instances of severe structural damage resulting from alkali-aggregate reaction have come to light in the Witwatersrand area.

Although the phenomenon of cement-aggregate reaction has been known and researched since it was first identified by Stanton[13] more than forty years ago, most of the research has been directed at understanding the mechanism of the reaction (eg Diamond[4,5] and Hansen[7]) and of devising ways of inhibiting or preventing the reaction from occurring (eg McCoy and Caldwell[9]). Little, if any work appears to have been done on the physical effects of the deterioration process on the strength and deformation properties of concrete.

On the Witwatersrand, damage to structures that appears to be associated with alkali-aggregate reaction, has so far been identified only in concrete made with Witwatersrand quartzite aggregate - both coarse aggregate and crusher sand. After describing the features associated with the deterioration which identify alkali-aggregate reaction as the cause, this paper will consider:

(a) means of assessing the extent of internal disruption of the concrete in reinforced concrete structures;
(b) the strength characteristics of concrete that has deteriorated as a result of alkali-aggregate reaction;
(c) the elastic and creep behaviour of deteriorated concrete; and
(d) a systematic procedure for investigating the condition of reinforced concrete structures that have possibly been affected by alkali-aggregate reaction.

2. VISUAL CHARACTERISTICS OF DETERIORATION OF CONCRETE MADE WITH WITWATERSRAND QUARTZITE AGGREGATE

Figure 1(a), page 2, illustrates the extensive surface cracking that has affected a number of bridge structures on the Witwatersrand. In the example shown, an attempt has been made to seal the cracks using an epoxy resin, but continuing expansion of the underlying concrete has reopened the cracks. The scale of the figure is indicated by two pairs of Demec targets on 100 mm centres stuck to the concrete and visible at the top centre and lower left of the photograph.

Figure 1(b), page 2, shows the deterioration of a bridge pier which has resulted in surface cracking, staining of the concrete with a white deposit and deep structural cracking which is aligned with the direction of the major principal stress in the concrete. Note also the presence of moisture seeping from a blocked drainage hole.

Figures 2(a), (b), page 3, and (c), page 4, show some of the features of the deterioration visible on a polished section of the concrete:

Figure 2(a) shows the dark reaction rims that develop around the edges of the larger aggregate particles, while Figure 2(b) shows cracking that has occurred at the interface between an aggregate particle and its surrounding mortar. Figure 2(c) shows (to a larger magnification) a crack between mortar to the left and an aggregate particle to the right, the crack being filled with a white reaction product. All of the above features appear to be typical of the effects of alkali-aggregate reaction.

The features illustrated in Figures 1 and 2 have given rise to two ranking systems for the condition of deteriorated concrete. Table 1, page 4, illustrates the system used for the ranking of concrete condition by visual inspection. It will be noted that the system is not entirely independent of petrographic examination of a polished section, as it has been discovered that signs of slight alkali-aggregate reaction can sometimes be detected on a polished section before there are any visible macroscopic signs of deterioration. The petrographic examinations performed on polished surfaces of concrete identify five features associated with alkali-aggregate reaction. These are the presence of:

(a) dark reaction rims around aggregate particles;
(b) white porcelanous acid-insoluble reaction products;
(c) cracks in aggregate particles;
(d) cracks in mortar; and
(e) loss of bond between mortar and coarse aggregate.

In order to obtain a numerical ranking of the degree of deterioration, a value of 1 has been assigned for the presence of each of the above features in a specimen of concrete. The sum of the points so assigned or the 'petrographic examination score' thus has a value of 5 if all five features are present and 0 if none are.

3. THE RESULTS OF CHEMICAL TESTING FOR POTENTIAL REACTIVITY OF WITWATERSRAND QUARTZITE AGGREGATE

The ASTM 'Standard test method for potential reactivity of aggregate (chemical method)', C289[1], measures the amount of reaction during twenty-four hours at 80 °C between 1N sodium hydroxide solution and crushed aggregate. The results of the test are expressed in terms of two parameters: R_c, the reduction in alkalinity, and S_c, the dissolved silica.

Figure 3, page 6, shows the results of a number of tests on Witwatersrand quartzite aggregate, and it will be seen that all but two of the results fall into the 'potentially deleterious' or 'deleterious' area of the diagram. The twenty four results represented by circles are for specimens of aggregate taken from concrete that had deteriorated by alkali-aggregate reaction, thus supporting the validity of the test.

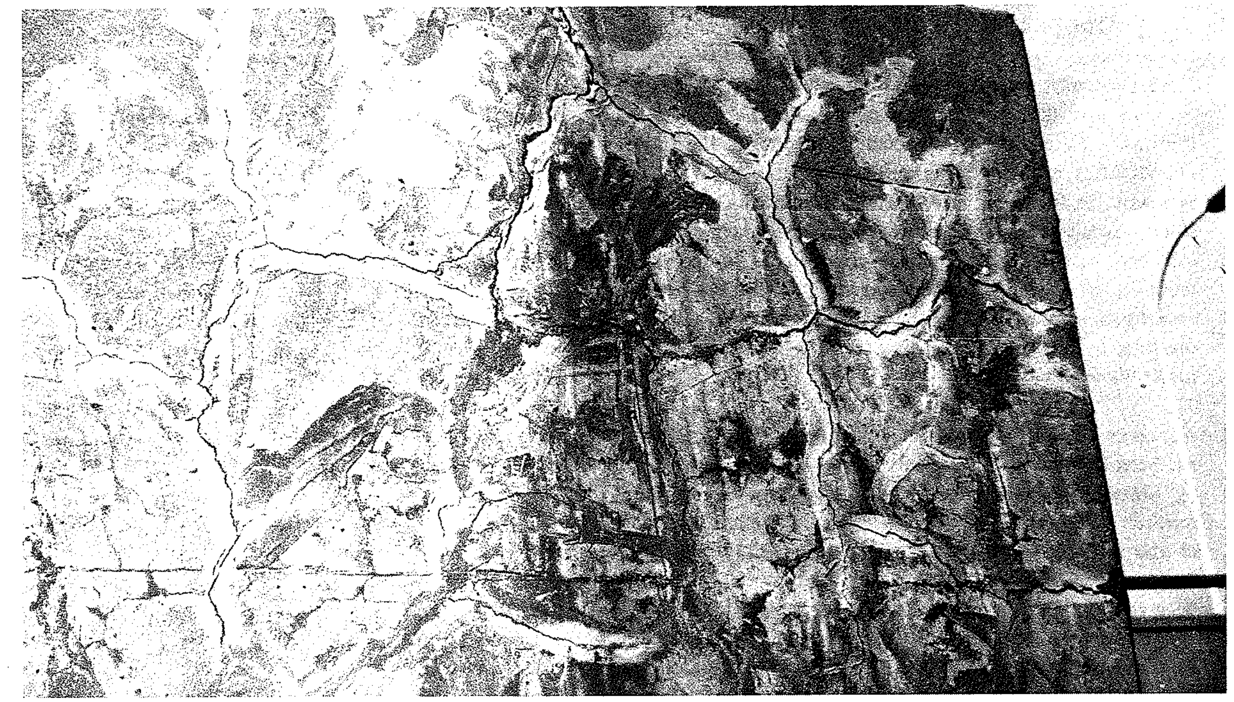

Figure 1 (a) Extensive cracking of exposed bridge portal. Attempts to seal the cracks by means of epoxy resin have failed.

FIGURE 1 : Examples of damage to structures constructed with Witwatersrand quartzite aggregate

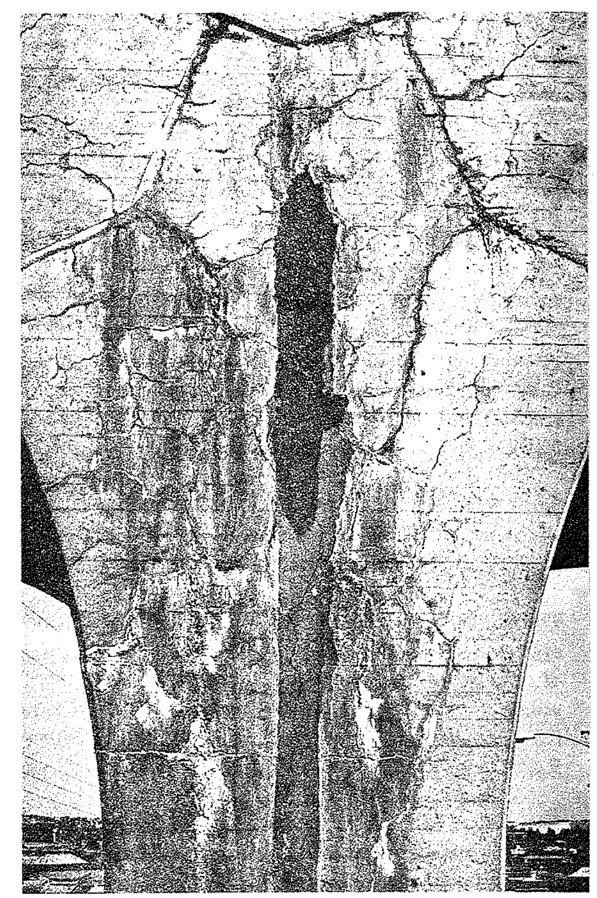

Figure 1 (b) : Damage to central pier. Note:

(i) blocked weephole

(ii) white staining of concrete surface

(iii) major cracking parallel to direction of stress.

The remaining test results, marked by letters, are for samples of aggregate taken from widely-spread points in the Transvaal and Orange Free State gold mining area (see the key map in Figure 3). The net effect of the results shown in Figure 3 is to give the impression that almost all Witwatersrand quartzites are potentially subject to alkali-aggregate reaction.

4. ALKALI CONTENT AND CORRELATION WITH RESULTS OF PETROGRAPHIC EXAMINATION AND CONCRETE CONDITION CLASSIFICATION

Table 2, page 5, summarizes the results of a number of alkali-content determinations and correlates these results with the corresponding petrographic examination score, the moisture condition of the concrete and the ranking by visual inspection. The first thing to be noted from Table 2 are the startlingly high alkali contents when expressed as a percentage of the mortar. A limited number of analyses of Witwatersrand quartzite have shown that the equivalent Na_2O content of the quartzite may be between 0,1 per cent and 0,7 per cent. The figures for the equivalent Na_2O content of the cement have accordingly been adjusted on the assumption that the equivalent Na_2O content of the sand in all of the mortar samples was 0,7 per cent. The figures shown in Table 2 therefore probably represent a lower limit to the equivalent Na_2O content of the cement. The cement content of the concrete was estimated by digesting the concrete in concentrated hydrochloric acid thus establishing the grading of the total aggregate. The cement content was taken to be represented by the acid-soluble portion of the digested concrete. Interestingly enough, the lowest values of alkali content recorded in Table 2 correspond to an analysis of efflorescent

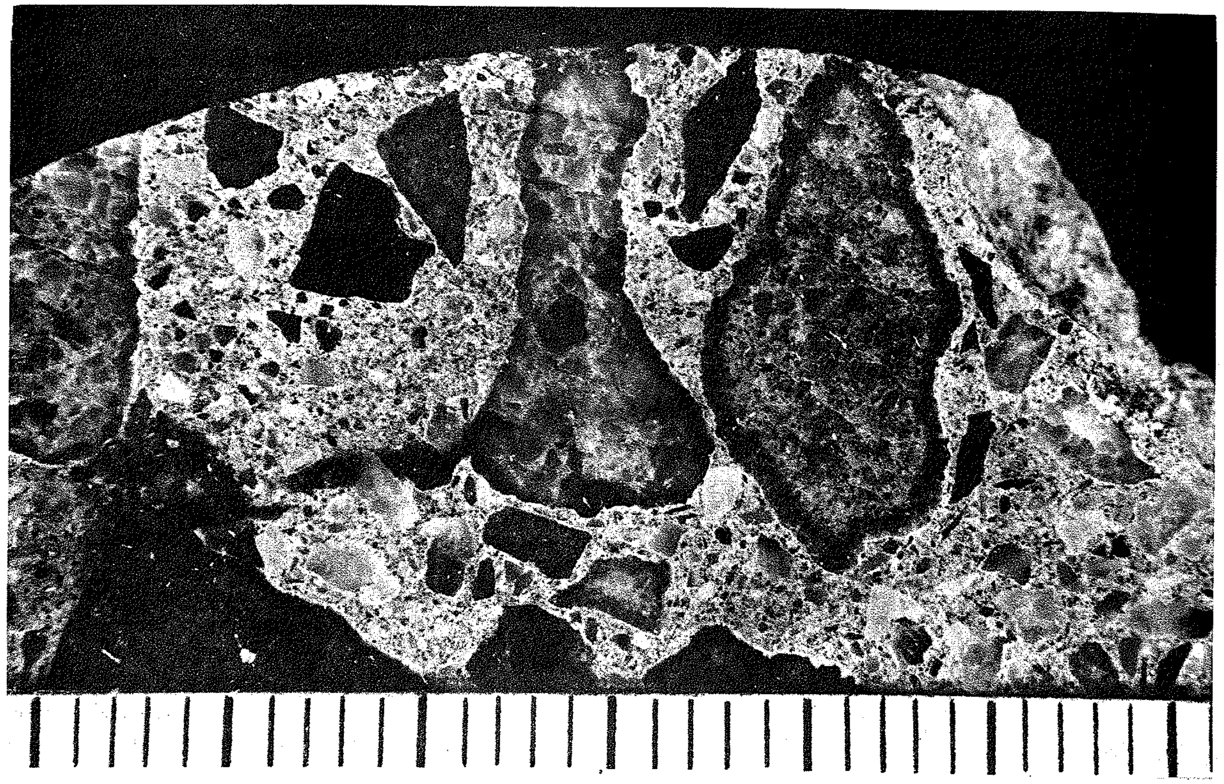

FIGURE 2(a) : Dark reaction rims around aggregate particles (scale in mm).

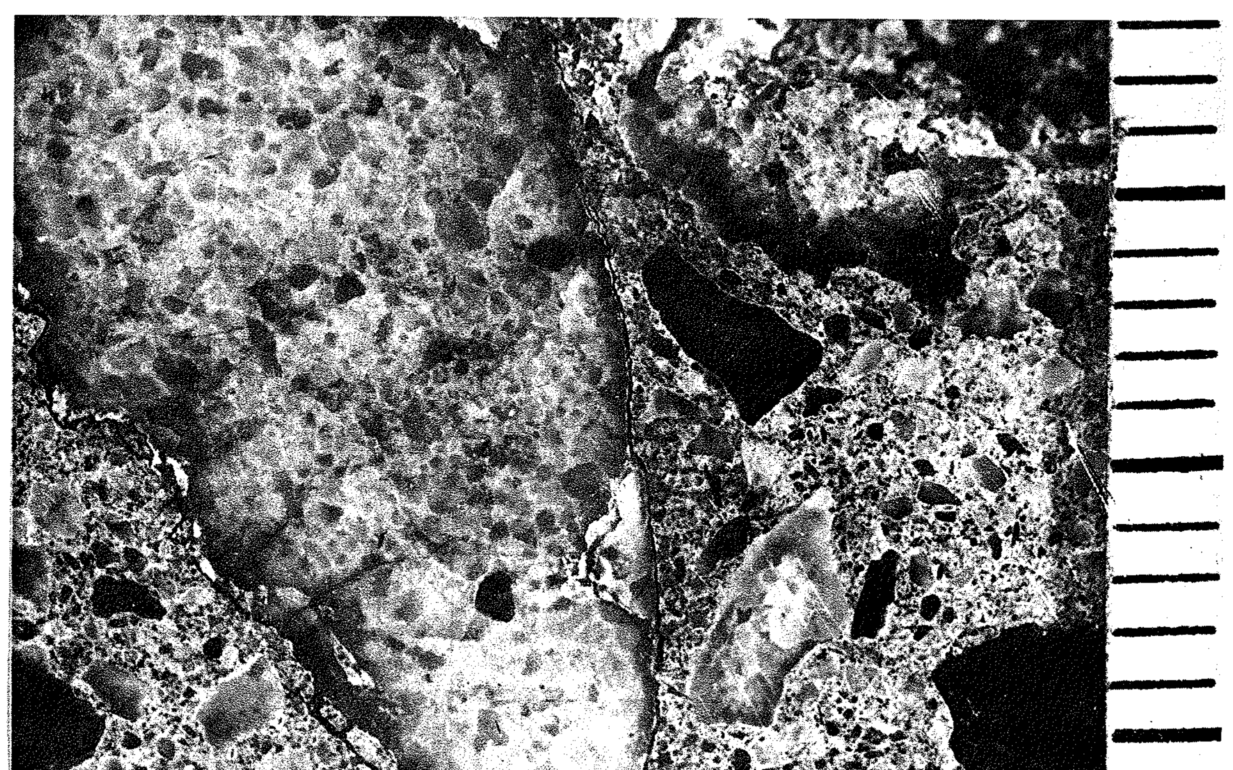

FIGURE 2(b) : Open crack between mortar and aggregate (scale in mm).

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FIGURE 2(c) : Crack between mortar (to left) and aggregate (to right) filled with reaction product. (field of view measures 0,5 mm x 0,5 mm)

salts scraped from the surface of a deteriorated structure. However, other efflorescent salts have shown high K_2O contents.

The reason for these abnormally high alkali contents is not known at present. One possible explanation is that the alkali had been concentrated by moisture migration through the concrete and evaporation near the surface, as suggested by Nixon et al[11]. This possibility is still being investigated. However, it appears significant that efflorescent salts in which the alkali content would be expected to be consistently high if concentration by percolation and evaporation were occurring, sometimes turn out to have a relatively low alkali content.

Figure 4 represents an attempt to correlate the equivalent Na_2O content of the mortar with the petrographic examination score, and shows that there is in fact no correlation between these two variables. The data appear to indicate that a combination of a high alkali content in the cement and an alkali susceptible aggregate will not necessarily result in damage by alkali-aggregate reaction. Environmental factors are evidently very important as well, and it will be seen from Table 2 that the most detrimental environmental condition is one of repeated wetting and drying

TABLE 1 : Ranking of concrete condition

Class or ranking	Likely petrographic examination score	Visible deterioration of concrete	Reduction in initial margin of safety	Remedial measures required	Protection against ingress of moisture required
A1	0	None	None	None	No
A2	0				Yes
A3	1 to 3	Signs of reaction visible on polished section			
B1	3 to 5	Hair cracks $\ngtr$ 0,3 mm wide	Negligible	Seal or grout cracks/strengthen/repair/reconstruct depending on outcome of serviceability investigation	Yes
B2	3 to 5	Cracks > 0,3 mm, but capable of being sealed	Requires investigation		
B3	4 to 5	Severe cracking through depth of member	Requires investigation		

TABLE 2 : Alkali contents of concrete and corresponding petrographic examination scores

Alkali content $Na_2O + K_2O$ (% of mortar)	Equivalent Na_2O content* (% of mortar)	Equivalent Na_2O content* (% of cement)	Petrographic examination score	Wet/dry condition	Visual inspection ranking
1,403	1,094	1,7	1	Permanently damp	A3
1,010	0,775	0,9	0		A3
0,642	0,510	0,3	1	Only occasionally wet/dry	A1
1,809	1,467	2,6	5	Cyclic wet/dry from internal drainage system	B3
1,518	1,308	2,2	0		B2
1,573	1,288	2,2	2		B2
1,510	1,201	1,9	5		B3
1,058	0,807	1,0	2		B2
1,021	0,778	0,9	2		B2
0,961	0,738	0,8	3		B2
0,939	0,692	0,7	4		B1
0,861	0,659	0,6	0		B2
0,732	0,551	0,4	1		B2
0,295	0,250	-	Efflorescent salts only		-
0,253	0,203	-			-

* $Na_2O + 0,66\ K_2O$

cycles. Concrete that remains dry or permanently damp appears to be less susceptible to alkali attack.

5. DETERMINATION OF THE EXTENT OF INTERNAL CRACKING IN A STRUCTURE

If a structure displays serious surface cracking, such as the vertical crack down the centre of the bridge pier, illustrated in Figure 1, it is clearly necessary to establish how deeply the crack penetrates the structure and whether other internal cracks exist which may not be evident as serious cracks on the surface of the structure. The ultrasonic pulse technique has been found most useful and effective for this purpose. Figure 5, page 7, demonstrates some of the capabilities of this technique. A transmitter and a receiver are placed in contact with opposite sides of the concrete member. An ultrasonic pulse is transmitted through the member and the average velocity of the pulse is measured. Any obstruction to the travel of the pulse in the form of an internal void or crack reduces the average pulse velocity. Figure 5(a) shows a set of measurements taken in two directions at right angles through a rectangular bridge pier. The ultrasonic pulse velocities measured in the two directions are virtually identical, and it may be concluded that the pier contains no serious internal defects. The measurements shown in Figure 5(b) indicate some form of discontinuity on the centre line of the column, which is otherwise sound. The discontinuity in this case was known to be an internal drainage duct of 200 mm diameter, thus showing that the ultrasonic pulse technique is capable of picking up discontinuities of relatively small lateral extent. The measurements shown in Figures 5(c) and (d) indicate the existence of a severe discontinuity that in each case passes transversely through the rectangular pier. The discontinuity was, in each case, visible on both surfaces of the column as a crack and the ultrasonic pulse measurement shows that the crack penetrates completely through the pier.

The ultrasonic pulse technique has also been used to assess the strength of concrete, eg Jones and Gatfield[8]. However, attempts to use the technique for this purpose have proved fruitless because, although the ultrasonic pulse velocity is highly sensitive to any form of disruption in the concrete, it is relatively insensitive to strength. This statement is demonstrated by the results shown in Figure 6, page 7. At the top of the diagram, the trend given by Jones and Gatfield has been reproduced, and shows that doubling the strength of a concrete from 20 MPa to 40 MPa results in only a 3 per cent increase in ultrasonic pulse velocity. Each

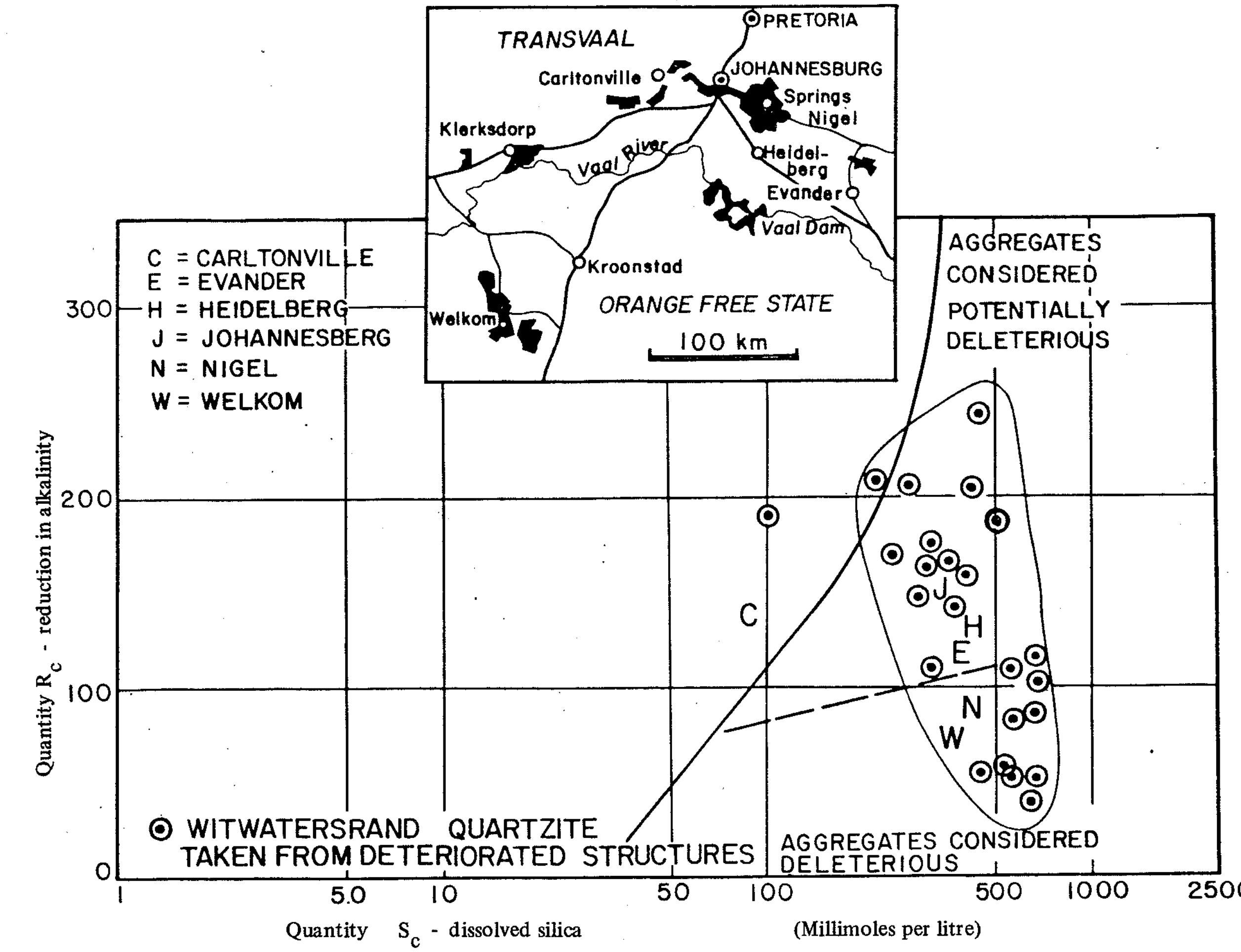


FIGURE 3 : Deleteriousness of Witwatersrand quartzite as indicated by ASTM C289.

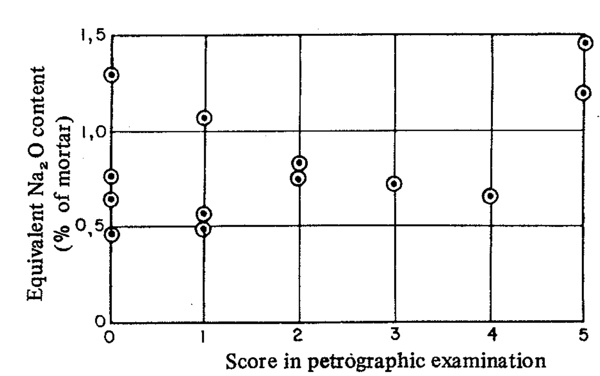


FIGURE 4 : Attempt to correlate condition of concrete as evidenced in petrographic examination with equivalent Na_2O content

set of experimental results in Figure 6 corresponds to the following test procedure:
The ultrasonic pulse velocity was measured across the diameter of a cylindrical concrete core at mid-height. The cylinder was then loaded in compression to just past the peak point of the stress/strain curve. The specimen was then unloaded and the ultrasonic pulse velocity re-determined. After this, the specimen was re-loaded to the maximum stress it could sustain and then unloaded once more. The process was repeated a number of times.

The results show in each case that as the concrete became progressively disrupted by the loading process, the ultrasonic pulse velocity decreased rapidly. The stress that the partly-disrupted concrete could sustain in each loading cycle did not however decrease by very much. Thus the ultrasonic pulse velocity gave an excellent indication of the state of disruption of the material, but a very insensitive

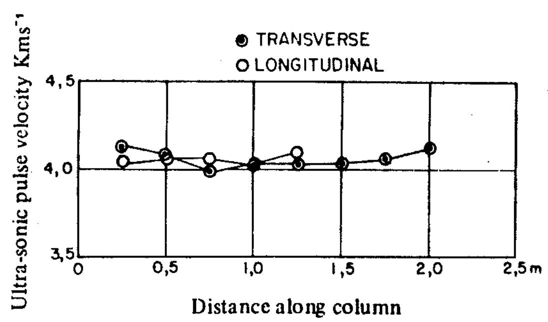


Figure 5(a) : Pulse velocity is the same in two orthogonal directions in an uncracked pier

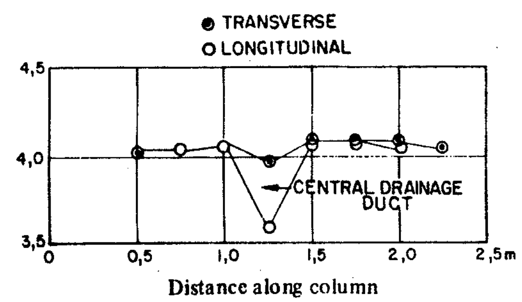
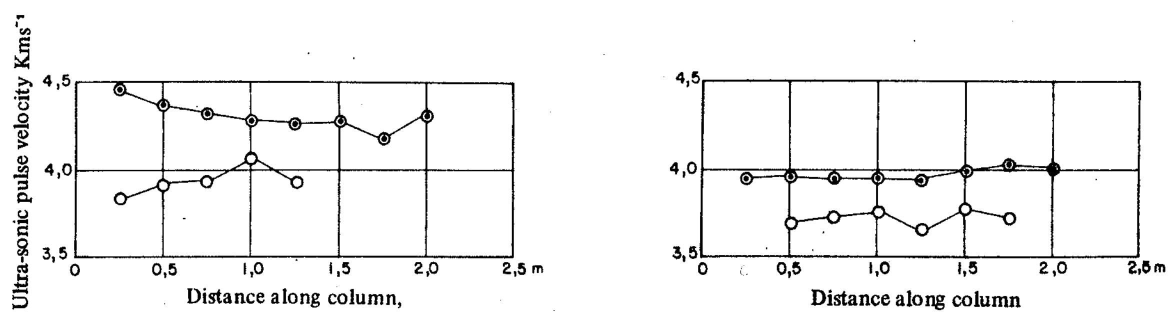


Figure 5(b) : The central built-in drainage duct in an uncracked pier is detected by a local decrease in the pulse velocity.



Figures 5(c) and (d) : Cracks penetrating the full width of two piers are indicated by a distinct difference in the pulse velocity parallel to and across the crack

FIGURE 5 : Use of ultrasonic pulse velocity to indicate extent of internal cracking in structures

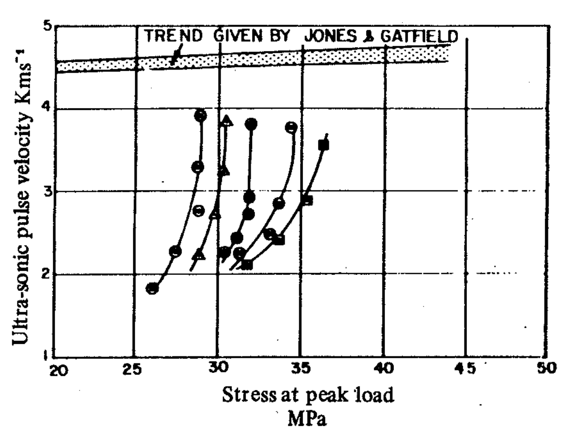


FIGURE 6 : Illustration of inability of ultrasonic pulse velocity to indicate strength of concrete combined with excellent ability to indicate extent of disruption.

indication of its change of strength. Similar results to those shown in Figure 6 have been obtained for indirect tension (cylinder splitting) tests.

6. THE STRENGTH OF DETERIORATED CONCRETE

The results shown in Figure 6 indicate that physical disruption of a concrete has a surprisingly small effect on the stress it can sustain. An extensive programme of strength tests on concrete that had been subject to alkali-aggregate reaction has come to a rather similar conclusion. The tests were performed on 85 mm diameter cores taken from structures showing various extents of deterioration. Unconfined compression tests were used to evaluate the crushing strength of the concrete, while indirect tension tests were used to evaluate the tensile strength of the material. The testing was carried out in accordance with the recommendations of the British Concrete Society[14].

The results of two suites of tests are summarized in Figure 7. Crushing strengths plot along the dashed line at a slope of 45° to the right of the origin, while indirect tensile values plot along the mirror image line at 45° to the left of the origin. In both cases, the nominal strength of the concrete was 30 MPa, and it will be seen that the lowest compressive strength recorded was 12 MPa. The values of tensile strength recorded in Figure 7 are, in a sense, fictitious as the strength measured across a fissure cannot differ very much from zero. Hence the assessment of the strength margin of a deteriorated structure or the design of strengthening measures for such a structure, must, of necessity, assume that the concrete has zero tensile strength.

7. ELASTIC AND TIME-DEPENDENT DEFORMATION OF DETERIORATED CONCRETE

Remedial and strengthening measures for structures weakened by alkali-aggregate reaction, will often involve prestressing procedures. Hence it is necessary to have a knowledge both of the elastic and time-dependent properties of the material. Figure 8 summarizes the elastic and time-dependent deformation properties of three typical concrete specimens. Specimen A was a concrete, classed A1 by visual inspection and had a petrographic examination score of 1. B and C were classed B3 by visual inspection, and had a petrographic examination score of 5. It will be noted that the elastic or instantaneous deformation of the badly deteriorated concrete, was approximately three times that of the relatively sound concrete, while the creep strain over a period of nine months was two and a half to four times as large. In making this comparison, however, it should be remembered that all three specimens were subjected to the same stress of 15 MPa. Whereas this stress represented approximately 50 per cent of the strength of specimen A, it probably represented 80 or 90 per cent of the strength of specimens B and C.

Table 3, page 9, summarizes the results of a number of Young's modulus and creep tests on concrete with different visual inspection ratings. It will be seen that deteriorated concrete, especially that in categories B2 and B3, is characterised by a low modulus of elasticity, a low structural viscosity, as well as an unusually high creep coefficient. With regard to the creep coefficient (defined as the ratio of creep strain to elastic strain at a particular time) the Comité Européen du Beton (CEB) in their international recommendations for the design and construction of concrete structures, give an expression that enables creep coefficients to be calculated for normal concretes (Neville[10]). According to this expression, the creep coefficient for a normal concrete would be expected to be of the order of 0,5 which is much less than most of the values

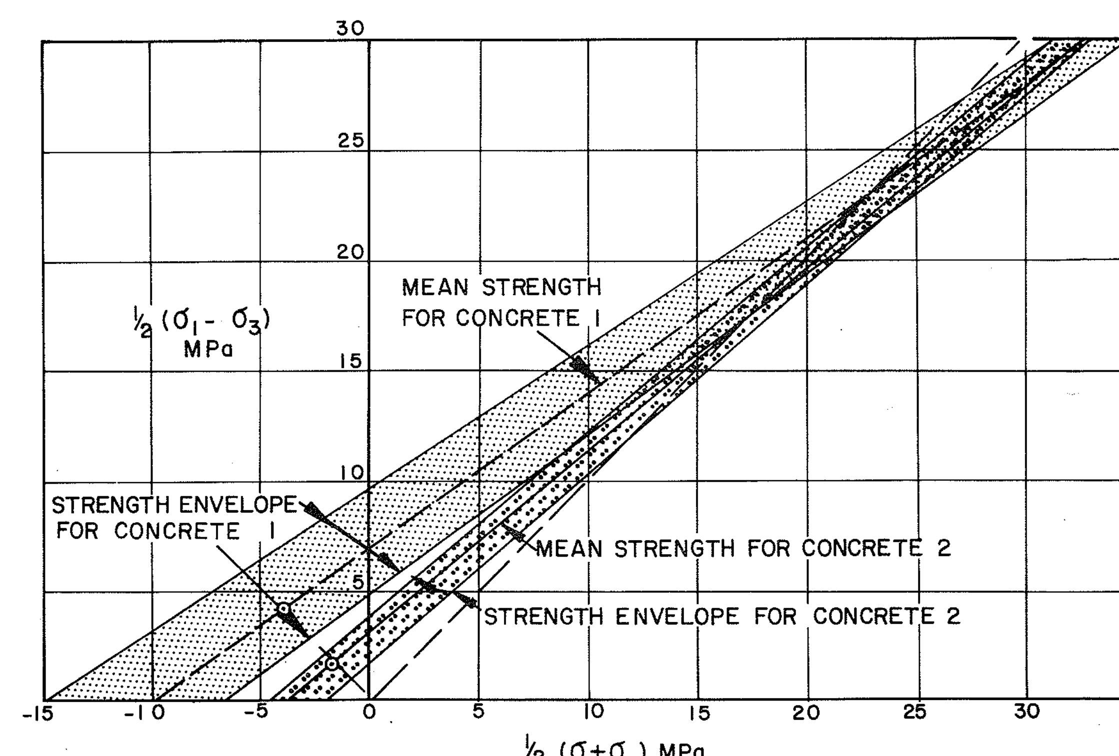


FIGURE 7 : Strength envelopes established for two deteriorated concretes by means of compression and indirect tension test.

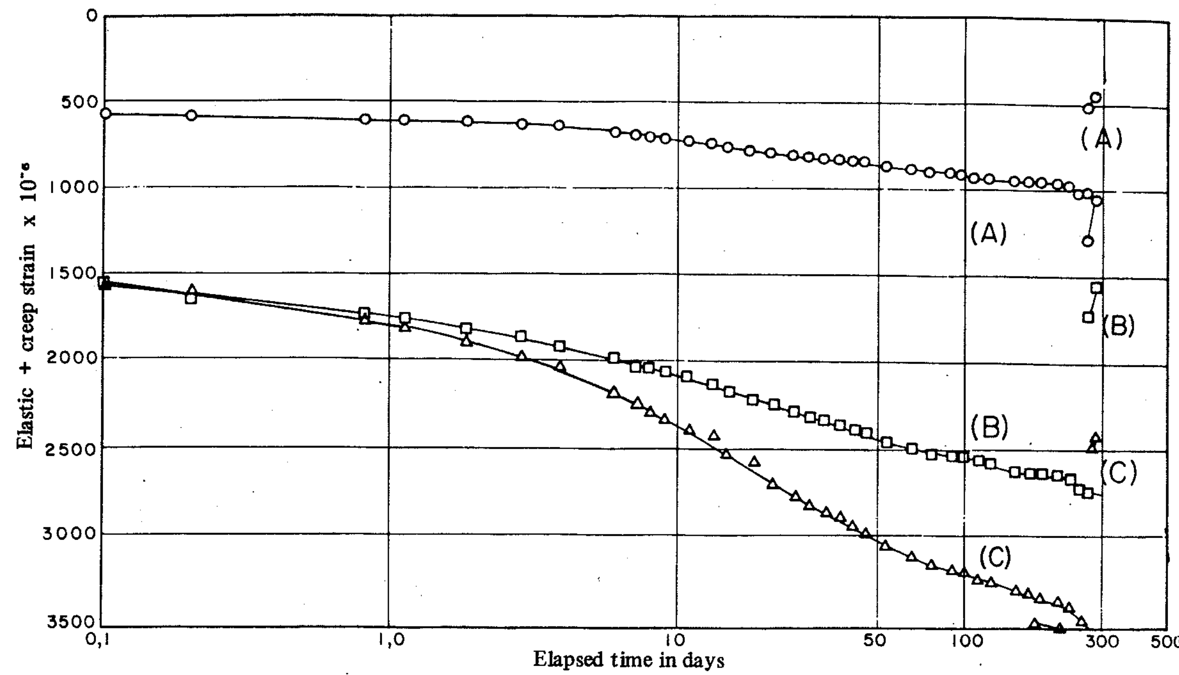


FIGURE 8 : Typical creep curves under a stress of 15MPa at a relative humidity of 45% for:
A: Undeteriorated concrete (class A1)
B: Deteriorated concrete (class B2)
C: Deteriorated concrete (class B3)

TABLE 3 : Creep behaviour of deteriorated concrete under a stress of 15 MPa and at 45 per cent relative humidity

Condition of concrete from visual inspection	Deformation parameters		Creep Coefficient at 9 months ϕ_9
	Elastic modulus E GPa	Structural viscosity* η GPa years	
A1 (A in Figure 8)	31,5	24,7	0,93
B3	11,9	17,0	0,52
B3 (B in Figure 8)	11,7	9,2	0,96
B3 (C in Figure 8)	12,6	5,6	1,69
B2	13,3	9,0	1,11
B2	27,3	11,1	1,88

* The time-dependent deformation of the concrete is here described by the Maxwell equation (eg Blight et al[2])

$$\varepsilon = \sigma \left(\frac{1}{E} + \frac{t}{\eta}\right)$$

in which ε = time-dependent strain
σ = sustained stress
t = time under stress

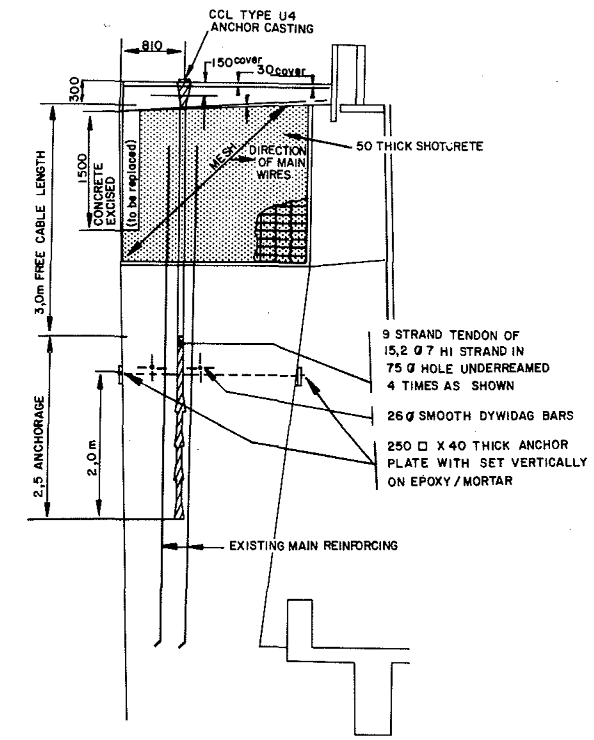


FIGURE 9 : Drawing of a double-deck portal structure which was seriously damaged by the alkali-aggregate reaction, showing the remedial measures which were taken

recorded in Table 3*. However, the CEB expression is meant to apply only to concretes stressed up to 35 per cent of their strength, whereas, as pointed out earlier, the results in Table 3 correspond to stresses of between 50 per cent and possibly 90 per cent of the specimen's strength. The important point, however, is that both the elastic and time-dependent deformation properties of the concrete must be known when designing remedial measures for structures that have deteriorated.

8. EXAMPLE OF REMEDIAL TREATMENT OF A DETERIORATED STRUCTURE

Alkali-aggregate reaction had resulted in severe deterioration of the concrete at the junction between the upper beam and the columns of the double-deck portal structure illustrated in Figure 9. Wide cracks extending through the entire depth of the structure had developed as a result of the loss of tensile strength of the concrete. This had been brought about by a faulty upvc rain water pipe embedded in the portal (which permitted intermittent ingress of water into the interior of the concrete). The deterioration had progressed to such an extent that the moment continuity between the beam and the columns could no longer be relied on. Some of the deteriorated concrete was broken away, new reinforcing was spliced in and the concrete that had been removed was replaced with new.

Moment continuity was restored by installing vertical pre-stressing tendons through the ends of the beam and anchoring them into the columns by means of epoxy mortar grout. After unloading the portal frame by jacking up both its beams, the beam ends were stressed against the tops of the columns. Provision was made for re-stressing the tendons at suitable time intervals to make up for any loss of prestress that may result from creep in the beam ends, the columns and the epoxy mortar grouted anchorages.

The maximum design stress in the concrete at the junction of the column and the upper beam of the structure is 7,6 MPa on the inside of the column. A maximum stress of 2,1 MPa results at the outside of the column from the remedial prestressing. The maximum design stress is, however, virtually unaffected by the prestress. The maximum design stress is well within the strength of the deteriorated concrete as illustrated in Figure 7.

9. CONCLUSION

The features present in concrete made with Witwatersrand quartzite aggregate, and which is showing severe deterioration, are consistent with those accepted as being characteristic of alkali-aggregate reaction. In addition, chemical testing in accordance with ASTM C289[1] indicates that Witwatersrand quartzite is susceptible to reaction with alkali.

A number of alkali determinations on concrete showing signs of deterioration, have indicated abnormally high alkali contents. Deterioration, however, only appears to occur under conditions of cyclic wetting and drying. A state of continual dampness does not appear to induce a severe reaction although some signs of deterioration are visible by petrographic examination, while a state of permanent dryness appears to preclude the development of the alkali-aggregate reaction.

Measurements of ultra-sonic pulse velocity have been shown to give a sensitive indication of the state of internal disruption of concrete in structural members. The ultrasonic pulse velocity, however, is not a reliable indicator of the strength of concrete. Although the strength of concrete attacked by alkali-aggregate reaction is reduced, it is not reduced by nearly as much as visual inspection would suggest. In the worst cases investigated to date, the compressive strength has been approximately halved, but the tensile strength has probably been reduced to zero.

Alkali-aggregate deterioration appears to have a somewhat more serious effect on the instantaneous and time-dependent deformation of concrete. The instantaneous

* The creep coefficient is not an entirely satisfactory description of creep characteristics. A concrete having a low Young's modulus and a large creep strain, eg the second entry in Table 3, can nevertheless have an apparently acceptable creep coefficient.

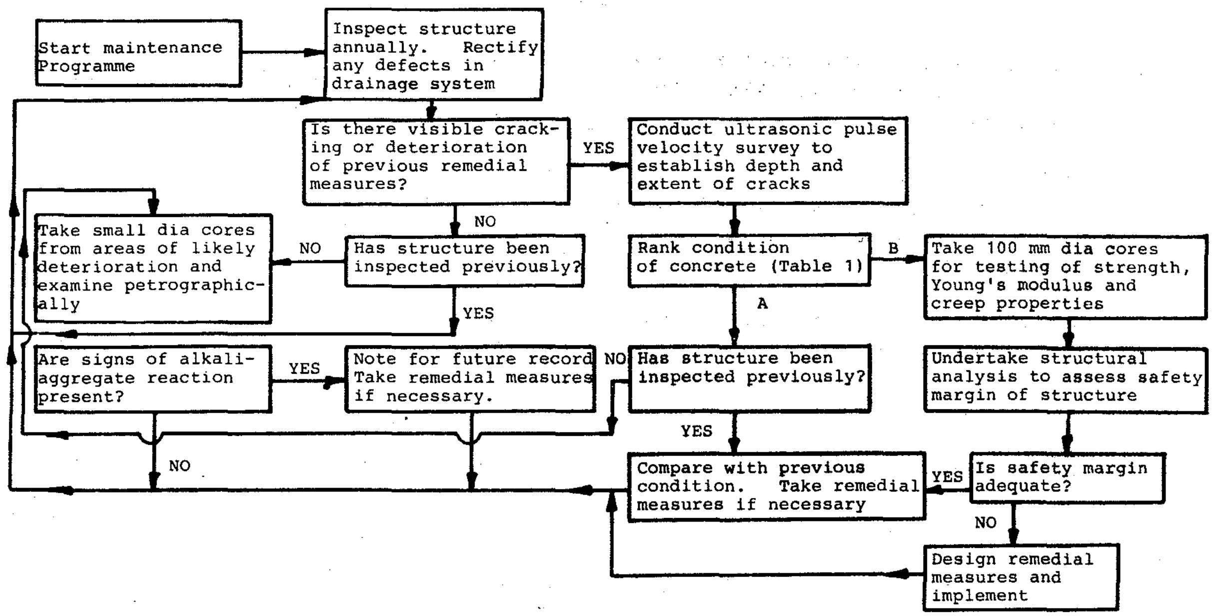


TABLE 4 : Proposed procedure for investigating condition of reinforced concrete structures that have possibly been affected by alkali-aggregate reaction

Young's modulus of a deteriorated concrete may be only about one third of that of a sound concrete, while the creep strain may be two and a half to four times as large.

In most of the cases of deterioration inspected by the authors, deterioration has only become noticeable on visual inspection from ten to fifteen years after construction. It is considered important that the owners of any concrete structure that is severely exposed to the weather and subjected to cycles of wetting and drying, adopt a systematic procedure for periodically investigating the condition of these structures. Table 4 (above) proposes such a system of inspection and testing, which incorporates many of the techniques described in this paper. It is considered necessary to inspect each structure annually, and to rectify any defects in the drainage system. Whether or not there is any visible deterioration, small diameter cores should be taken from areas of likely deterioration and these should be subjected to petrographic examination for signs of incipient deterioration. If visible deterioration has occurred, the extent of any cracking in the interior of the structure should be established by means of an ultrasonic pulse velocity survey, and depending on the results of this survey, measures should be taken to ascertain the safety margin of the structure. These measures will involve the taking of large diameter cores for the testing of strength, Young's modulus and creep properties. The results of these tests will enable the strength margin of the structure to be investigated. If the safety margin proves inadequate, remedial measures should be designed and implemented.

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DISCUSSION

Dr A C Liebenberg (Consultant, Cape Town) referred to reports that the reaction in practice seemed to have been more common where there had been cyclic wet and dry conditions. However, all the standard tests were non-cyclic, and he asked for an explanation of this. He suggested that the cyclic phenonemon in practice might perhaps be a purely mechanical magnification of crack size, but the tests were done non-cyclically. He also referred to the statement by Mr Vivian that the cracking of dams had occurred on the downstream face, and asked why no mention had been made of the upstream face which was continuously submerged.

Prof Blight said that he was not sure what the explanation was. He merely offered what had been observed 'for what it was worth' and as some enlightenment for the audience. Apart from looking at structures which were buried and therefore kept continually damp, they had looked at the walls of water storage reservoirs which were thin in relation to the head of water they retained and were therefore subject to a continuous seepage of water. In the case of the more massive wall of a gravity dam, it was quite possible that the exterior surface was virtually unaffected by seepage from the dam, and they had found absolutely no evidence of damage to, or even any signs of alkali-aggregate reaction in these water-retaining structures. They had looked specifically at water-retaining structures that had been constructed at about the same time as some of the bridge structures which were now showing signs of damage. They observed faces exposed to the weather and also faces in internal galleries which were obviously not exposed to the weather but were subject to seepage. Absolutely no sign of damage had been found.

Prof S Diamond (Purdue University, Lafayette, USA) referring to the question of cyclic wetting and drying, suggested that the possibility at least existed of repeated concentration of the alkali hydroxide pore solution as the concrete dried out and water was lost; with a resultant rise in the concentration of alkali. If there was a droplet or a fume of alkali hydroxide next to a reactive grain as it became progressively more concentrated, the attack would speed up, and then when the concrete was re-wet, presumably the alkali hydroxide would be diluted and the attack might slow down. Because of this process, one might have a far more severe attack than with a constant medium concentration of alkali hydroxide. He knew of no evidence to support this view, but it seemed a reasonable possibility.

Dr D Hobbs (C & CA, London, England) disagreed with Prof Diamond's explanation; and felt that Mr Vivian had given the explanation. He had said that the heart of the concrete had been virtually undamaged, while the outer drying surface of the concrete had been damaged. Dr Hobbs suggested that the inner core had expanded more than the outer layers of the concrete, and as a result the outer layers had gone into tension and the inner core had gone into compression. Thus more expansion had occurred in the core of the concrete. He did not suggest that alkali migration did not occur, but the fracture or the way in which the cracks appeared in the concrete, suggested that more expansion had occurred in the inner core.

Prof Blight referred to the Lansdowne Bridge they had studied the previous day. He had noticed that the cracks were in fact empty. If they were empty the question was what was keeping them open, and the answer could only be expansion further back so in fact what they were looking at was perhaps a skin which had failed in tension which was adhering to a core which was in an expanded state. He agreed with Dr Hobbs' theory.

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