CASE STUDIES OF THE PRACTICAL AND ECONOMICAL IMPACT OF ALKALI-SILICA REACTION IN SOUTH AFRICA

R E Oberholster
PPC Technical Services
P O Box 40073
Cleveland 2022
South Africa

ABSTRACT

Alkali-silica reaction has affected the serviceability and durability of a significant number of concrete structures in South Africa, especially in the South Western Cape where the current worth of structures affected is estimated to be more than R1 600 million (R1 = US$0,2743). Several million rand has already been spent on rehabilitating some of the affected structures in South Africa and some of those in the South Western Cape are discussed as case studies in this paper. Indications are that the cost of rehabilitating and partially replacing the structures that are discussed, could be between R50- and R100 million.
Keywords: Alkali-silica reaction, repair and rehabilitation, economical impact.

INTRODUCTION

In South Africa, ASR was first identified in the 1970s (Oberholster & Brandt, 1974) as the reason for the expansion and cracking of concrete structures in the Cape Peninsula. Currently, enough is known of the phenomenon to minimise the risk of ASR in a new structure. The purpose of this paper is to review the impact of ASR on the management and rehabilitation of affected structures and some of the consequences of measures taken to minimise the risk of its occurrence.

DETERMINATION OF THE WORTH OF AFFECTED STRUCTURES

The incidence of ASR is highest in the South Western Cape. Although widespread, it occurs on a smaller scale in the Eastern Cape and Gauteng. During the investigation of the problem in bridges in the South Western Cape, it was found to be totally impossible to assess the cost of damage in terms of the replacement cost of the structures or parts thereof. The only approximation that could be made was the *present worth* of the structures which was calculated as follows:

$$\text{Present Worth Factor} = \frac{\text{Construction Price Index for 1977}}{\text{Construction Price Index for Particular Year}}$$

$$\text{Construction Price Index} = 0.66(\text{Building Materials Index}) + 0.33(\text{Consumer Price Index})$$

Taking into account only those bridges and the 27 km concrete pavement that were affected by the reaction, the *present worth* estimate of the investment was R37,3 million in 1977 (Semmelink, 1981). Taking an average annual escalation of 15 %, this would amount to about R530 million in 1996. If other structures which have subsequently been identified as being affected by ASR are included, the figure could conservatively exceed R2 billion. However, the real significance of ASR can only be assessed if one analyses the situation for specific structures.

REHABILITATION AND MANAGEMENT OF AFFECTED STRUCTURES

In a number of instances, structures affected by ASR had to be demolished or parts replaced, because they had become unserviceable or dangerous. For example, part of the upper beam of a reinforced double-storey portal frame of the M1 motorway in Johannesburg, constructed in 1966 (Alexander et al. 1992) was demolished and reconstructed in 1991 at a cost of R1 million. However, where problems with the serviceability and durability of concrete structures are encountered, the aim should first be to carry out a proper condition survey to establish the extent and cause of the problem as outlined in a number of publications (ACI, 1990; British Cement Association, 1992). Once ASR has been identified in a structure, a future course of action should be established as described, for example, in the publication of The Institution of Structural Engineers (1992) or in Fulton's Concrete Technology (Oberholster, 1994). Depending on the extent and severity of ASR, this could involve monitoring only of the structure at intervals, repair, or repair and strengthening, in each case combined with regular monitoring. In extreme cases, demolition of the structure may be necessary.

The following examples describe the procedures that have been followed and the economical and practical implications, for a number of different structures in the South Western Cape.

The N2/1 concrete freeway between Cape Town and Somerset West

In 1970, approximately 27 km of National Route N2/1, between Cape Town and Somerset West were realigned and reconstructed as a dual-carriage freeway. The carriageway consists of a continuous unreinforced concrete slab 7,3 m wide and 200 mm thick placed with a slipform paver in a single lift. It has a longitudinal joint along the centre of the slab formed by a polythene tape insert (50 mm x 0,1 mm) and the two halves are tied together with high tensile dowel bars 600 mm long and 16 mm in diameter at 750 mm intervals. Skewed transverse joints at 80° to the longitudinal axis were formed by sawing to a depth of 50 mm with a diamond saw. Joints were sealed with a preformed neoprene seal. Malmesbury Group metasediment was used as coarse aggregate.

By 1975, an unusually high incidence of hairline cracking was observed in the pavement. It was established that this was due to ASR. The action of traffic aggravated the cracking so that by 1979 a number of failures at transverse joints had occurred. These failures took the form of spalling at the joints and required patching. It was, however, apparent that the pavement might fail progressively under traffic. A programme was commenced to: (i) evaluate the combined effects of ASR and traffic; (ii) determine the potential life of the pavement (Freeme & Shackel, 1981); (iii) evaluate all the experimental data on the pavement; and (iv) based on the findings, propose maintenance and remedial procedures and draw up proper contract documents (Van der Walt et al. 1981).

The first phase of the investigation involved (i) heavy vehicle simulator (HVS) testing on two selected sites; (ii) recording and quantifying the development of surface cracking; (iii) measuring the structural behaviour in terms of deflection and deformation; and (iv) rehabilitation analysis. It was concluded that all of the pavement was potentially liable to fail under traffic regardless of whether ASR progressed or abated and that the estimated remaining life of the pavement was between two and five years for the most heavily trafficked sections. Also, the construction of an experimental section, employing a number of various rehabilitation alternatives, would be essential to arrive at the ultimate solution.

In the second phase, therefore, an experimental section of overlays was constructed using three different types of materials, namely asphalt, crushed stone with an asphalt overlay and OPC concrete. The asphalt overlays were built at three different thicknesses using a semi-gapgraded asphalt with precoated chips at the surface. Two types of rigid overlays were constructed, namely, a jointed and a continuously reinforced concrete pavement. Performance evaluation of the different overlays was expedited by using an HVS.

Based on the results of the evaluations, rehabilitation of 22 km of the freeway was commenced in 1986 and was done in the following stages:

- Repair of the joints by breaking out the concrete and replacing with a 40 MPa Portland fly ash cement concrete.
- Installation of 4 300 cylindrical no-fines concrete shoulder drains 150 mm in diameter in the asphalt shoulder to drain water collecting on top of the clay working platform under the concrete pavement, thus retarding further ASR.
- Cleaning cracks and joints and sealing with a bitumen-rubber sealant.
- Applying a 40-mm bitumen-rubber overlay with a single seal bitumen-rubber stress absorbing interlayer between the concrete and the overlay.

The rehabilitation of an additional 3 km was carried out separately as follows:

- 1981, cracks, transverse joints where neoprene sealers had been pressed below the surface and longitudinal joints between the concrete and the premix, cleaned with compressed air and sealed with pli-astic. Spalls patched with premix. Surface sealed with a 13-mm and 7-mm double seal.
- 1991, repairing the joints by breaking out the concrete and replacing with a 40 MPa concrete and applying a 13-mm bitumen-rubber single seal.
- 1994, applying a 40-mm bitumen-rubber asphalt overlay, continuously graded with 19-mm maximum aggregate size.

The costs of the different stages of the freeway project are as follows:

• 1970: Construction cost of pavement and ancillaries	R 2 x 10^6
• 1979: HVS investigations and rehabilitation analysis	R 1.8 x 10^6
• 1983: Experimental overlays and HVS investigations, about	R 2.3 x 10^6
• 1985: Repairs to 22 km of existing road and bitumen single seal	R 2.3 x 10^6
• 1986: Bitumen-rubber asphalt overlay	R 6 x 10^6
• 1981: Repairs to 3 km (costs not available)	-
• 1991: Joint repairs and 13 mm bitumen-rubber single seal to 3 km	R 2 x 10^6
• 1994: 40-mm bitumen-rubber asphalt overlay to 3 km	R 2.4 x 10^6
Total estimated cost	R16.8 x 10^6

Concrete railway sleepers in the Sishen-Saldanha line

The 861-km long Sishen-Saldanha railway line was constructed between 1973 and 1976 to connect the iron ore deposits being mined at Sishen in the Northern Cape with the nearest suitable export harbour at Saldanha on the West Coast. Approximately 1.7 million heavy duty FY prestressed concrete sleepers were manufactured for this purpose, of which approximately 1.4 million were used in the construction of the line. The remainder was stockpiled at Saldanha for maintenance purposes.

The sleepers were manufactured at Saldanha by the longline process, using local granite aggregate and RHPC from the Riebeeck Plant of PPC. Maintenance staff reported serious longitudinal cracking of the sleepers in 1985 in the line (Oberholster et al. 1992). Inspection revealed that exposed sleepers in the stockpiles were also cracked. Investigations following on the discovery of the cracking of the sleepers included:

- Laboratory investigation to establish the cause of the cracking.
- Tests on cores to establish the potential for continued expansion of the sleepers.
- Long-term tests on two halves of a cracked sleeper from the line, to compare the expansion of one untreated half to that of the other half treated with a silane.
- Spot checks of 100 sleepers per km over the entire length of the line between 1988 and 1989 to determine the extent of the problem.
- Measurements on specific sleepers every three months to determine the rate of increase in crack width.
- Dynamic load tests on uncracked sleepers and sleepers from the line with different degrees of cracking, to develop an evaluation model in terms of the degree of ASR. This model was used to predict the service life and draw up a programme for repair and replacement of the sleepers.
- A detailed survey over the total distance of the line to determine the percentage of sleepers in each category.

The detailed survey revealed that approximately 70 % of the sleepers in the line show symptoms of ASR varying from slight to severe. The rehabilitation consists of treating reparable sleepers with silane. Since about 30 trains per week with an average length of 210 wagons (2.5 km) use the line and there are crossing loops only every 85 km, sleepers cannot be removed from the line for treatment, but individual sleepers have to be opened up without disturbing the geometry, classified and treated.

The relevant costs for this project are as follows:

• Original cost of 1.4 million sleepers in line	R 8.4 x 10^6
• Investigations, replacement and rehabilitation of sleepers since 1987	R 0.5 x 10^6
• Estimated rehabilitation costs for next 10 years	R17 x 10^6
• Estimated replacement costs for next 10 years	R17 x 10^6
Total estimated cost	R34.5 x 10^6

The estimates are in 1994 Rand value. A sleeper currently costs about R115.

The Good Hope Centre, Cape Town

The Good Hope Centre was constructed between 1976 and 1977. It is the largest cross-vaulted concrete structure in the world. The roof structure is 80 m by 80 m on plan and 20 m high. It is made up of 900 triangular, precast, gunited forms supported by in-situ ribs which in turn are supported by two diagonal, reinforced concrete arches each spanning 113 m and four external, terminal, reinforced concrete arches each spanning 80 m. The arch beams come together at each corner of the structure and are connected by means of exposed in-situ reinforced concrete buttresses. The horizontal forces generated by the arch beams are resisted by four prestressed concrete perimeter beams, each tie beam able to resist a force of 2 000 tons by means of high tensile prestressing cable strands. A 75-mm thick layer of sand completely separates the structure from the foundations to allow for initial shortening of the tie beams due to prestressing, temperature variations and creep of the concrete. The four pile caps at the corners of the structure each support a vertical load of 5 000 tons and each pile cap consists of five ton large diameter piles, founded and keyed into hard Malmesbury Group rock at depths varying from 6 m to 20 m. For the concrete of the four buttresses a granite aggregate and a cement content of 395 kg/m^3 were used. For the concrete of the arches a Malmesbury metasediment coarse aggregate was used. The eastern terminal arch was cast with a cement content of 428 kg/m^3 and the three remaining terminal arches and the diagonal arches of concrete with a cement content of 371 kg/m^3. The cement was from the Riebeeck Plant of PPC and, according to statistics, the average Na_2O equivalent of the cement from this factory over the period of construction varied between 0.8 % and 0.9 % (Semmelink, 1981).

Slight cracking of all four buttresses and terminal arches was reported in 1981. The cracking of the buttresses containing granite coarse aggregate, was regarded as unusual because at that stage ASR had only been identified in structures containing Malmesbury metasediment as aggregate. In 1993, an assessment of the extent and seriousness of ASR in the structure was commenced to decide on remedial measures and also to institute a management programme for monitoring and controlling the effects of ASR in the future.

An extensive condition survey of the buttresses and arches was carried out and the crack widths and distribution mapped in detail on a plan of the structure. Cores from the concrete were examined petrographically for the presence of ASR. The estimated expansion of the concrete was calculated and a structural severity rating assigned to the different elements. Measuring points were installed and the dimensional changes monitored with a Demec strain gauge.

The cost statistics are as follows:

• Original cost of construction	R 12 x 10^6
• Condition survey, consultation, laboratory investigations, monitoring	R 38 x 10^3
• Silane/siloxane treatment of buttresses, epoxy injection of cracks, sealing of surface; silane/siloxane treatment of terminal arches, epoxy injection where required; monitoring and consultation, 1994/95	R250 x 10^3
• Future monitoring at 6-month intervals, per annum	R 10 x 10^3
Total estimated cost	R298 x 10^3

The Steenbras Pumped-Storage Power Station

The Steenbras Power Station was commissioned in 1979. In 1984 it was established that the cracking of the concrete of the power station buildings, machine foundations and ancillary works was due to ASR. Monitoring of the concrete for dimensional, humidity and temperature changes was carried out between 1985 and 1990. In 1987 the Cape Town City Electrical Engineer's Department set up a monitoring programme aimed at determining the movement of two machines in Shaft B by means of a precision distometer with invar wires and an electronic clinometer. By September 1992 the measuring grid for the distometer clinometers was redesigned and implemented for Shaft A. This gave more detailed information with regard to the effects of ASR causing movement between the turbine top cover and generator lower bracket of the machines in this shaft. The B-shaft system was upgraded to the same configuration in the following year.

The results of the readings taken by the different authorities are briefly as follows:

- The concrete of the machine foundations, cooling water basement and anchor block is expanding and a total expansion of 0.550% and an expansion rate of 0.129% per annum have been recorded for the turbine floor in shaft B. The relative humidity in the concrete ranged between 71% and 100% at the different points.
- The measurements of the concrete movement over a period of 4.7 years have shown that: (i) Expansion of the concrete is causing the entire machine support structure to rise. This rise is restrained by the rock in contact with the shaft lining and is, therefore, greatest towards the centre of the shafts. The differential effect results in a tilting of the various floors and the machines themselves. (ii) Above the turbine floor (which is in the drier part of the support structure) a total vertical expansion of 1.5 mm has been recorded between levels 53.65 m and 66.12 m, which represents 27 microstrain per annum. (iii) There appears to be a local upward bulging of the turbine floor around the machines. (iv) Distometer measurements indicate that the entire shaft B is expanding and compressing against the surrounding rock. The rate of expansion seems to be decreasing, probably as a result of increasing resistance from the rock. (v) Expansion of the concrete is heterogeneous, i.e. at some points little or no movement is taking place, at others expansion is continuing at a steady rate and at others shrinkage is occurring.
- Measurement of machine movements carried out over a period of only 340 days since the new measuring grid was installed showed that the average machine movements correspond with the apparent movement of the concrete. There is a significant seasonal thermal movement of up to three times the apparent permanent effect.

The practical impact of the ASR in the concrete is misalignment of the four machines, each consisting of a reversible pump/turbine and motor/generator set rated at 45 MW. Consequently, these machines have to be realigned at intervals of approximately 5 years. An outage expressly for realignment purposes would take up to 8 weeks. The potential economic impact could be up to R2 million per machine, but in reality is more likely to be in the region of R1 million due to alternative loading strategies. Loss of machine availability over the sixteen year life of the power station to date would be approximately five to six machine years out of a possible sixty. This represents about 10 % unavailability due to direct and indirect consequences of ASR. During these outage periods where extensive repairs and refurbishment take place (which is not related to ASR) shaft realignment is checked and corrected as required. These outage periods could range from six to eight months, of which less than a month would be due to realignment as a consequence of ASR. In this way the economic impact of machine outages due to ASR is reduced to less than R1 million per machine per five year cycle, or R3- to R4 million for the station over a five year cycle. This represents less than 20 % of lost revenue due to machine down time for planned maintenance. The extra cost incurred due to extra labour and material is less than 10% of the total repair costs.

The approximate relevant cost statistics are summarised below:

• Original cost of Power Station	R 75 x 10^6
• Diagnosis of ASR, monitoring	R 10 x 10^3
• Consultation, installation of monitoring equipment, measurements	R130 x 10^3
• Cost of measurement systems	R200 x 10^3
• Loss of revenue due to ASR over 15-year period	R 12 x 10^6
• Cost of realignment/refurbishment due to ASR over 15-year period	R 12 x 10^6
Total estimated cost over 15-year period	R 24,3x 10^6

Dams

Reliable information of the cost implications of ASR in dams is not available. However, a brief overview of ASR and its effects in eight affected dams in South Africa has been given by Seddon and van den Berg, 1990. Subsequently ASR has also been diagnosed in the Ceres Dam.

Where cracking in dams has occurred, detailed inspections have been carried out, cores drilled for laboratory investigation to determine whether ASR is present and the stability re-evaluated. In several dams, cores have been extracted from the crest down into the foundations and also through the wall. At most of the dams where ASR has been identified, monitoring of total vertical strain has been initiated by precise levelling of points on the crest and/or sliding micrometers in boreholes.

The Churchill Dam is the only South African dam where major remedial work has been carried out. This multiple arch dam was built in 1944 and extensive, mainly horizontal cracking of the arches and buttresses was first observed in 1957. Some of the horizontal cracks penetrated right through the buttress at the upstream side. Initial investigations concluded that the cracking was caused by shrinkage of aggregates and thermal effects and that the cracking impaired the safety of the structure (Stutterheim et al. 1967). Consequently, as an immediate measure, the reservoir water level was restricted. Remedial action commenced and the buttresses were strengthened by the addition of reinforced concrete (colcrete) extensions on the downstream side. It was established in 1987 that the deterioration of the dam was in fact caused by ASR. Upward expansion of the arches at a greater rate than the buttresses caused the partial lifting and cracking of the upstream end of the buttresses.

MEASURES TO MINIMISE OR PREVENT THE RISK OF ASR

It is now well established that the risk of ASR in a new structure can be minimised or eliminated by taking appropriate measures such as using a non-reactive aggregate. Where an alkali-reactive aggregate has to be used, the alkali content of the concrete must be limited or the use of an approved cement extender should be considered (Oberholster, 1994). However, each of these might have some technical or economic implications.

Technical and economic implications of producing a low-alkali cement

All ordinary and rapid-hardening portland cements (OPC and RHPC) currently produced in South Africa are low-alkali cements as defined by SABS Standard Specification 471, although control of the alkali content is necessary for only a relatively small proportion of portland cement applications. For example, it is estimated that in the South Western Cape, only 10 % to 15 % of all cement produced in this region is used in concrete which could be susceptible to ASR. Controlling the alkali content of a cement at a specific limit, has certain technical and cost implications for a cement manufacturer, some of which are discussed below, using the PPC De Hoek plant as an example (Waterson & Harding, 1990). The measures applied at this factory to control the alkali levels, and the cost implications, might be substantially different from other cement plants in South Africa.

Screening levels and raw mix design

To produce a cement with an Na_2O equivalent of $\leq 0.55\%$, about 20% of the limestone (the fine, high-alkali portion) has to be screened out and discarded. Should a cement with an Na_2O equivalent of, for example, 0.8% be allowed, the screened rejects would be reduced to about 7.5% and this results in:

- Extension of limestone reserves by at least 14%.
- An annual saving in production costs of R1 million.
- Greater flexibility in raw mix design and savings on burning and milling costs.

Cement strengths

Dropping the alkali content of a cement results in a decrease in the early strengths but an increase in the 28-day strength. This effect of lower early strengths might become more pronounced with fly ash (FA) and Ground Granulated Blastfurnace Slag (GGBS) blends.

Cost of cement and concrete

Currently OPC and RHPC with an Na_2O equivalent of about 0.55% and a low-alkali sulphate-resisting portland cement (LASRC) with a guaranteed low Na_2O equivalent of 0.3% are being sold in the South Western Cape. For large, exposed concrete structures in which aggregate of the Malmesbury Group is used, it is usually required that a cement with an alkali content of less than 0.50% Na_2O equivalent, i.e. the LASRC, be used. It is calculated that the price differential between using the low-alkali OPC and the LASRC, would be about 9 %/m^3, in favour of the former for a 40 MPa concrete. The price difference between using a low-alkali OPC and an OPC with an Na_2O equivalent of 0.8 % in a 40 MPa concrete, could be of the same order in favour of the high-alkali cement. Currently, an RHPC with an Na_2O equivalent below 0.55 % is not available in the South Western Cape. This is a matter for concern in the precast concrete industry (where a high cement content is used as a rule) if Malmesbury Group metasediment aggregate is used.

An alternative measure for minimising the risk of ASR with an alkali-reactive aggregate, is to substitute a portion of a high-alkali cement with an extender, for example a minimum of 20 % FA or 40 % GGBS. Unfortunately, both of these industrial by-products are generated in areas where transport would add substantially to the delivered price in for example, the Western Cape. For example, a 40 MPa concrete in Gauteng costs about 4 %/m^3 less when a 50 % GGBS blend is used compared with 100 % OPC, while in Cape Town, the GGBS blend would cost about 4 %/m^3 more.

CONCLUSIONS

While enough is now known about ASR in South Africa to minimise the risk of its occurrence in a new structure, the implications of taking such precautions are significant in respect of the higher cost of concrete and the reduced life of limestone quarries.

The diagnosis of ASR in concrete, the assessment of the structural severity, the recommendation of remedial measures and the drawing up of a programme of maintenance and monitoring, require specialised and multidisciplinary skills.

In most instances where ASR has been diagnosed in structures, the durability and serviceability have been greatly affected and the service life in many cases significantly reduced. Public authorities and private owners of such structures are hard-pressed for funds for the high costs of reparation and maintenance.

ACKNOWLEDGEMENTS

I wish to acknowledge the help of the following persons whose names are not mentioned in the text, for information supplied:
O Schnitter, B Nothnagel, G Els, E de Villiers, G de Beer, F P Bruwer, P J Coetzee, D R Paul, J B Davidson, C Seddon, A White, A Ackroyd, B Myburgh, J Keating and B Paton.

References

Alexander, M.G., Blight, G.E. & Lampacher, B.J. 1992, 'Pre-demolition tests on structural concrete damaged by AAR', Proc. 9th Int. Conf. on AAR, London, UK, 27-31 July, 1-8.

American Concrete Institute 1990, ACI 201.1R-68, *Guide for making a condition survey of concrete in service*, ACI Manual of Concrete Practice Part 1, 1-22.

British Cement Association 1992, *The diagnosis of alkali-silica reaction*, British Cement Association Publication 45.042, 1-44.

Department of Transport, 1983, Directorate: Land Transport. *Experimental overlays of a distressed rigid pavement*, April, 8pp.

Division of Building Technology, CSIR, 1990, *Monitoring of concrete at Steenbras pumped-storage Power Station*, Contract Report 55034052/42, February, 45pp.

Freeme, C.R. & Shackel, B. 1981, Evaluation of a concrete pavement affected by the alkali-aggregate reaction using a heavy vehicle simulator. Proc.5th Int. Conf. on AAR, Cape

Town, South Africa, S252/20.

Oberholster, R.E. & Brandt, M.P. 1974, An example of cement-aggregate reaction in South Africa. Proc. South African Electron Micr. Soc., Vol 4, 59-60.

Oberholster, R.E. 1994, *Alkali-silica reaction* in Fulton's Concrete Technology, ed B.J.Addis, Portland Cement Institute, Midrand, Seventh (revised) edition, pp181-207.

Oberholster, R.E. 1981, Alkali-aggregate reaction in South Africa. Proc. 5th Int. Conf. on AAR, Cape Town,, S252/8.

Oberholster, R.E., Maree, H. & Brand, J.H.B. 1992, Cracked prestressed concrete railway sleepers: Alkali-silica reaction or delayed ettringite formation. Proc. 9th Int. Conf. on AAR, London, UK, 27-31 July, 739-747.

Seddon, C.V. & van den Berg, C.L. 1990, Deterioration of concrete dams due to alkali-aggregate reaction, SANCOLD Symposium on Dam Safety: Four Years On, Pretoria, September, 19pp.

Semmelink, C.J. 1981, Field survey of the extent of cracking and other details of concrete structures showing deterioration due to alkali-aggregate reaction in the South Western Cape Province. Proc. 5th Int. Conf.on AAR, Cape Town, S252/19.

Stutterheim, N., Laurie, J.A.P. & Shand, N. 1967, Deterioration of a multiple arch dam as a result of shrinkage of aggregate, 9th ICOLD, Q.34-R.14.

The Institution of Structural Engineers, 1992 Structural effects of alkali-silica reaction: Interim technical guidance on the appraisal of existing structures. London: The Institution.

Van der Walt, N., Strauss, P.J. & Schnitter, O. 1981, Rehabilitation analysis of a road pavement cracked by alkali-aggregate reaction. Proc. 5th Int. Conf. AAR, Cape Town, S252/21.

Waterson, C.H. & Harding J. 1990, The implications of raising the alkali levels in ordinary portland cement in the Cape Western Region. PPC Cape Western Region Report, September, 15pp.