# ALKALI AGGREGATE REACTION IN EXISTING STRUCTURES - WHAT CAN IT TELL US?

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## ABSTRACT

Concrete in New Zealand structures was used to study three aspects of AAR. Firstly, the incidence of AAR was examined to identify the conditions which lead to the reaction so that they can be avoided in future constructions. Secondly, in situ and accelerated expansions of concrete from five structures affected by AAR were compared to find out whether accelerated core expansions can be used to predict the likelihood of future in situ expansion. Thirdly, the ability of a proprietary surface treatment to reduce further expansion was evaluated by accelerated core expansion tests.
*Keywords:* alkali aggregate reaction, expansion monitoring, surface treatments

## INTRODUCTION

Potentially reactive aggregates are the cheapest and most readily available in parts of New Zealand and there is a need to determine how they can be used safely. In addition, once a structure has been found to be affected by AAR its owners want to know what action, if any, is needed to maintain its performance.

We have used the performance of field concretes to look at three aspects of managing AAR. These are: to identify the reactive combinations of mix designs and materials so that they can be avoided in future constructions; to find out whether AAR expansion of in situ concrete can be predicted from the expansion of core samples; and to see whether a surface treatment could prevent or reduce further expansion of concrete damaged by AAR. This approach has allowed us to study more concretes than would have been possible with laboratory mixes and avoids the difficulty of predicting in-situ performance from laboratory mixed concretes.

## PREVENTION OF AAR IN NEW CONCRETES

### Selection of "problem" aggregates to study

The incidence and severity of AAR in New Zealand has been limited by the widespread use of low alkali cements. Surveys of several hundred bridges in areas where potentially reactive aggregates are available found that at most 12% of the structures in each area showed signs of AAR (Freitag 1994). Most damage observed was minor. Only five bridges with extensive networks of wide cracks were seen, all in the Tongariro area.

ASTM C289 and C227 tests had earlier identified that andesites and rhyolites from Taranaki, Tongariro and Waikato River deposits were reactive or potentially reactive (Kennerley & St John. 1988). Taranaki andesites and Waikato River deposits are widely used in concrete because alternatives are expensive. Tongariro volcanics are not used as often but have produced severe AAR damage. These three types of aggregate were thus considered most worthy of closer study.

The early investigations of these aggregates included assessing the effect of alkali content on the incidence of reaction, and the current code of practice for minimising AAR in New Zealand (CCANZ 1991) is based on this work, limiting concrete alkali contents to 2.5 $kg/m^3$ and/or using mineral admixtures or slag. The present investigations aim to specify "safe" alkali limits for the three selected aggregates so that unnecessary costs are not incurred in complying with inappropriate restrictions.

## Techniques

Core samples were taken from concretes showing characteristics such as triple point cracking, pattern cracking, open cracks, or wet stained cracks, which were considered the most likely indicators of AAR.

Petrographic techniques were used to identify the aggregates present and whether AAR had actually occurred. Information on the materials and mix designs of concretes believed to be affected by AAR was obtained from construction records wherever possible. To confirm the materials used, cement source and alkali contents were determined by chemical analysis. Cement contents were estimated from point counts in thin section and from determination of water to cement ratio by fluorescence in thin section. These techniques are discussed in an accompanying paper (St John & Freitag. 1996).

## Results

For the Taranaki concretes there was no consistent relationship between the incidence of AAR and concrete alkali contents. The reaction occurred in some concretes which contained 1.2-1.7 $kg/m^3$ alkalis, yet other concretes with the same or higher alkali contents were not cracked and therefore not sampled. Cement alkali contents determined from chemical analyses were within the range indicated by the original records, and chloride ion contents were very low despite the surfaces sampled being exposed to sea spray. These observations suggest that the concrete surfaces were too impermeable for alkalis to be leached by runoff or augmented by external sources. The apparently variable reactivity is therefore probably due to the difference between andesites from different volcanic events. Exposure to moisture could also contribute, since the cracking was most often seen on bridge abutments. The findings suggest that an alkali limit of 1.5 $kg/m^3$ might be necessary to completely prevent cracking due to AAR where Taranaki andesites are used, compared to earlier laboratory results which suggested that concrete alkali contents less than 3.5 $kg/m^3$ would be "safe" (Smith 1988).

Among the Tongariro concretes the most severe damage occurred when a high alkali cement was combined with a fine aggregate containing rhyolite at close to its pessimum proportion. Incidences of AAR detected in other concretes from the area were minor and very localised. They may have been caused by alkalis released by andesite particles, particularly since one such concrete contained pozzolan. Overall, a concrete alkali content of 2.0 $kg/m^3$ or less appears to be required to prevent AAR with the rhyolitic material, but no clear critical alkali content was evident for the andesites.

The Waikato/South Auckland investigation showed that AAR was restricted to structures which contained sand from the Waikato River and were built during a period when a high alkali cement was available in the area. Most also contained basalt coarse aggregate and some contained basalt crusher dust. The incidence of AAR in the samples analysed suggests that even if the cement does not provide enough alkali, the alkali from the basalt could provide the balance required for AAR to proceed, as indicated by laboratory investigations described elsewhere in these proceedings (Goguel 1996). Work to establish critical alkali levels for Waikato River sand is continuing.

## PREDICTING FUTURE EXPANSION IN AFFECTED CONCRETES

In New Zealand few cases of AAR have warranted the expense of structural analysis and detailed site monitoring. A more simple method of predicting future expansion is needed so that maintenance decisions can be based on physical evidence rather than speculation.

In 1989 when the investigation was conceived, the most widely used method of predicting the likelihood of further expansion was to measure the accelerated expansion of core samples as described by the British Cement Association (1988). This investigation sought to find out whether such expansion tests would give results which represented qualitatively what was happening to the original structure. The nature of accelerated tests on core samples means it is reasonable to assume that if a core does not expand in the laboratory then the concrete is unlikely to expand in situ, but precludes an accurate quantitative relationship. Should the tests produce significant expansion then in situ monitoring would be recommended.

### Selection of test concretes

Six structures were selected for monitoring. Three, in the Tongariro region, had extensively cracked abutments and thus represented the degree of damage most likely to prompt a monitoring programme. The other three were in Taranaki and had minor cracking on the abutments, which were more heavily reinforced than those in Tongariro. Monitoring these concretes would thus test the method on small amounts of in situ expansion.

### Laboratory testing

Core samples were taken from the six structures in February 1991. Demec studs were attached along one gauge length representing the interior of the concrete on each side of the core. After being instrumented, the cores were wrapped in polythene bags to prevent further drying and stored for several months to let them equilibrate after being removed from the restraints of the surrounding concrete. They were air dry when put into the test conditions.

The storage conditions specified by the BCA's (1988) method were modified to avoid leaching alkalis from the specimens. Instead, the cores were stored in plastic bags containing about 20 ml of water, with the original outer surface of the core in contact with the water. There was enough space around the core for air (and water vapour) to circulate. Initially each core was wrapped in damp cotton cloth as subsequently recommended by the BCA's modified method (1992) but the cloths soon started to rot in the alkaline conditions and they were discarded after three weeks. This meant that some alkali was removed from the system. The plastic bags containing the cores were stored in waterproof containers immersed in water baths operating at 40 ±2°C throughout the test period.

Gauge lengths were measured approximately every three months. Before each length measurement the cores were cooled to 21 ±2°C overnight. They were also weighed at every measurement. Water was added to the bag if needed. Many of the plastic bags leaked, resulting in further loss of alkali from the test environment.

At 14 months an equipment failure resulted in the water baths overheating to 95°C during a weekend. The cores were measured and weighed after being allowed to cool, then were left at 21 ±2°C for four weeks while the damaged equipment was repaired.

### In situ monitoring

Demec studs were mounted over several cracks on abutments of five of the bridges. At each demec site three or four studs straddled two or more intersecting cracks with each gauge length approximately perpendicular to the crack. This arrangement was designed to exaggerate movement, making it easier to detect. Gauge lengths were measured with a vernier calliper fitted with demec points to allow greater flexibility in gauge length. The calliper was fitted with a low precision dial gauge so that it would not detect the small movements caused by moisture movement or thermal effects. Measurements were taken in the same month each year from 1991.

### Results

The in situ expansion measurements and core expansions are given in Table 1.

All cores expanded rapidly in the first eight days due to moisture uptake. Records of weight change showed that the specimens were generally taking up moisture consistently throughout the test period, but occasional surface drying on the upper part of the core suggested that moisture distribution in the plastic bag was not even.

None of the monitoring sites on the Taranaki bridges exhibited consistent expansion during the first four years so the expansions of the cores from these bridges can be taken to represent concrete which is not expanding in situ.

Core samples were taken near six of the monitoring sites on the Tongariro bridges. Two of these, T2 and TW2, had expanded more than 1000 microstrain after two years and both corresponded to sites where in situ expansions were detected on more than one gauge length over four years. The two cores from each bridge represent the concrete from the same part of the structure. Therefore it is reasonable to assume that

*Table 1 In situ and accelerated core expansions*

| Bridge | | In Situ Crack Widths | | | | | | | | Accelerated Net Core Expansions (Inner Gauge Lengths) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | In Situ Site | Increase in Gauge Length after Four Years (cm*) | | | | | | | Nearest Core Site | Net Expansion (microstrain)** | | |
| | | | AB | AC | AD | BC | BD | CD | | | 711 Days | 1053 Days | 1333 Days |
| TARANAKI | Oakura | 1 | 0 | 0 | 0 | 0 | 0 | 0 | | ) Oa 2b | 235 | 228 | 228 |
| | | | | | | | | | | ) Oa 3 | 162 | 177 | 207 |
| | Waiongana | 1 | 0 | 0 | | 0 | | | | ) | | | |
| | | 2 | 0 | 0 | | 0 | | | | ) Wg 2 | 70 | 84 | 21 |
| | | 3 | 0 | 0 | | 0 | | | | ) | | | |
| | Waitara | No in situ measurements | | | | | | | | Wr 3 | 182 | 196 | 168 |
| TONGARIRO | Tokaanu | 1 | 0 | 0.020 | 0 | 0 | 0 | 0 | | - | - | - | - |
| | | 2 | 0.015 | 0.025 | | 0.030 | | | | T 2 | 1561 | 2247 | 2506 |
| | | 3 | 0 | 0 | | 0 | | | | T 4 | 441 | 861 | 1134 |
| | | 4 | 0 | 0 | | 0 | | | | - | - | - | - |
| | Te Whaiau | 1 | 0.010 | 0 | 0 | 0 | 0 | 0 | | - | - | - | - |
| | | 2 | 0.015 | 0 | 0 | 0 | 0 | 0 | | TW 1 | 706*** | 842*** | 888*** |
| | | 3 | 0.020 | 0.010 | 0 | 0.010 | 0 | 0 | | TW 2 | 1372 | 1939 | 2268 |
| | | 4 | 0 | 0 | 0.010 | 0 | 0.010 | 0 | | - | - | - | - |
| | | 5 | 0.020 | 0.015 | 0 | 0 | 0 | 0 | | - | - | - | - |
| | Wanganui | 1 | 0 | 0 | | 0 | | | | - | - | - | - |
| | | 2 | 0.020 | 0.010 | | 0 | | | | W 2 | 658 | 721 | 812 |
| | | 3 | 0 | 0 | | 0 | | | | W 3 | 644 | 882 | 1071 |
| | | 4 | 0 | 0 | | 0 | | | | - | - | - | - |

* Smallest division on dial gauge was 0.01 cm. Gauge lengths ranged from 8 to 24 cm, with most between 10 and 20 cm.

** Excludes initial 8 day expansion, which was attributed to moisture uptake.

*** This core rapidly expanded some 2000 microstrain shortly after the equipment failure. The expansion quoted excludes this rapid expansion.

The reactive aggregates are andesite in the Taranaki bridges, and rhyolitic material in the Tongariro region.

the expansions of all the cores from the Tokaanu and Te Whaiau Bridges represent concrete which is expanding in situ. The expansions of the cores from the Wanganui bridge are of similar magnitude to TW1 and T4 so were assumed to indicate in situ expansion even though only one site recorded in situ expansion.

These results suggest that core expansions greater than 700 microstrain at three years could indicate an in situ expansion. The likelihood of the correlation increases with the magnitude of the core expansion: the larger expansions were in cores from bridges where expansion was recorded at more than one location. Thus core expansions greater than 1000 microstrain after two years can probably also be taken to indicate in situ expansion. However, the slow expansion of core Tokaanu 4 suggests that three years is a minimum test period for the conditions used here.

The variation between cores from the same concrete demonstrates how important it is to take more than one core sample from a concrete for accelerated expansion testing.

## EFFECTIVENESS OF SURFACE TREATMENTS FOR REDUCING EXPANSION

### Selection of test concretes

Prestressed beams were removed from a bridge in 1988 because of cracking caused by AAR. The beams were recovered intact and stored near the site to simulate the original exposure conditions and enable the beams to be used for research.

Initial attempts to examine the effectiveness of coatings to reduce expansion involved treating the beams and monitoring changes in dimension with surveying equipment. The results of this work were inconclusive because neither treated nor control beams expanded significantly (Freitag & Rowe. 1992).

It was then decided to store core samples from the beams in the accelerated conditions described above to determine whether the concrete had any potential for further expansion and, if so, whether the expansion could be reduced by a proprietary surface treatment (Freitag 1995). Core samples were thus taken from two of the beams in 1992.

### Test parameters

The surface treatment tested was a proprietary system consisting of a silane-siloxane primer overcoated with an acrylic. This was applied to cores with diameters of 70 mm and 95 mm and lengths of 100-130 mm. Cores were coated in two moisture conditions: air dry and slightly drier than saturated surface dry (SSD). Twenty four specimens were tested, including 16 uncoated specimens used to distinguish the effects of core diameter, parent beam and moisture condition.

### Results

Table 2 gives the results of weight change and expansion monitoring to three years.

Weight changes of air dried cores suggest that the treatment may provide a small degree of protection against the ingress of moisture from a humid environment (SSD coated cores gained more weight than the SSD controls because they were drier at the start of the exposure period). All cores expanded, but only five expanded more than 700 microstrain after three years. This supports the lack of overall in situ expansion observed in the earlier work. There was no significant difference between expansions of treated and untreated cores, but no treated cores had net expansions greater than 600 microstrain after three years.

The silane acrylic treatment therefore needs to be tested on an actively expanding concrete to determine whether its effect on deleterious expansion is significant, and whether the moisture condition of the concrete at the time the treatment is applied affects the ability of the treatment to reduce expansion.

*Table 2 Weight increase and expansion of treated and untreated cores*

| Treatment | Core | Initial Moisture Condition* | Diameter (mm) | Weight Increase 966 Days (g) | Net Expansion 966 Days** (microstrain) |
|---|---|---|---|---|---|
| | 33 | SSD | 75 | 5 | 240 |
| | 34 | SSD | 75 | 3 | 120 |
| | 3 | SSD | 95 | 5 | 192 |
| | 7 | SSD | 95 | 6 | 302 |
| | 18 | SSD | 75 | 4 | 181 |
| | 20 | SSD | 75 | 2 | 874 |
| | 42 | SSD | 95 | 2 | 439 |
| | 43 | SSD | 95 | 3 | 755 |
| Untreated | 35 | Air dry | 75 | 18 | 407 |
| | 36 | Air dry | 75 | 20 | 257 |
| | 4 | Air dry | 95 | 32 | 389 |
| | 9 | Air dry | 95 | 29 | 229 |
| | 21 | Air dry | 75 | 17 | 3198 |
| | 22 | Air dry | 75 | 15 | 706 |
| | 44 | Air dry | 95 | 28 | 856 |
| | 45 | Air dry | 95 | 26 | 460 |
| | 13 | SSD | 95 | 14 | 195 |
| | 5 | SSD | 95 | 16 | 330 |
| | 28 | SSD | 75 | 9 | 350 |
| Silane-Acrylic | 29 | SSD | 75 | 9 | 579 |
| | 14 | Air dry | 95 | 21 | 381 |
| | 17 | Air dry | 95 | 25 | 417 |
| | 30 | Air dry | 75 | 13 | 456 |
| | 31 | Air dry | 75 | 16 | 371 |

* SSD = saturated surface dry (slightly drier than SSD in the case of the treated cores).
**Expansion at 966 days less expansion at 7 days, attributed to moisture uptake.

## CONCLUSIONS

Examination of concretes from existing structures has identified reactive combinations of materials and mix designs, and use of this information will help to minimise the risk of AAR expansion in future construction. The investigation has also established a qualitative relationship between in situ and accelerated core expansions, which will enable the likelihood of future expansion of a damaged structure to be predicted. Attempts to evaluate the effectiveness of a surface treatment in reducing expansion were not completely successful because the subject concrete was found to be no longer actively expanding, but the results suggest that the material is worth examining further.

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