Minimising the Risk of Damage to Concrete

Guidance Notes and Recommended Practice

(Second Edition incorporating Amendment 1:2012)
Acknowledgements

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TR 3

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Preface

New Zealand has been fortunate that despite the use of reactive aggregates in concrete, there have been few cases where damage due to alkali aggregate reaction (AAR) has necessitated extensive remedial repairs. This has largely been due to the early recognition of the potential for reaction and careful use of low alkali cements in areas where there was the greatest risk.

The first edition of this publication drew on findings of extensive studies by DSIR Chemistry Division and the Ministry of Works and Development during the 1950’s and 1960’s. These were reviewed by a working party, chaired by David Barnard of CCANZ, to prepare guidelines that could be used to minimise the damage caused by AAR in future construction. While all members of the Working Party contributed to information gathering and decisions required to produce this document, D.A. St John, DSIR Chemistry Division, undertook the principal editing and writing role. This reflected his considerable experience of New Zealand alkali aggregate reactions and knowledge of international research.

Since then, ongoing work by D.A. St John and R.L. Goguel of Industrial Research Limited (formerly DSIR Chemistry Division) and S.A. Freitag of Opus International Consultants Limited (formerly Works Consultancy) has clarified the extent and severity of the reaction in New Zealand, and explained some unusual cases of AAR observed in concrete structures. Information gained from these studies includes the contribution of alalis from aggregates (often in amounts outweighing the contribution from other sources), the applicability of tests for assessing aggregate reactivity, greater understanding about the behaviour of greywacke aggregates, and the fact that many precautions only alleviate the reaction, rather than prevent it. There have also been significant developments in the management of AAR internationally since the 1991 edition was published.

There still remain unanswered questions, but it is believed that this edition will provide sufficient information for users and manufacturers of concrete to use materials available in New Zealand with minimal risk to damage from AAR. It is hoped that the electronic format will permit appropriate revisions to be made with minimal effort in future to accommodate local and international developments.

S.A. Freitag
R. Goguel
N.B. Milestone

Amendment 1 of Second Edition (2012)

This amendment of TR3 includes:

(a) Recommendations made in a report (Freitag, Bruce & Shayan, 2011) on ASR observed in some precast concrete piles using South Island aggregates containing microcrystalline quartz or strained quartz.

(b) Precautions to minimise the risk of ASR in concrete containing recycled concrete aggregate, based on CCANZ Technical Report TR 14.

Derek Chisholm
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1.0 Introduction

1.1 Scope and Purpose of TR3

This document was written to provide a comprehensive review of New Zealand and international experience and research that would enable concrete specifiers and suppliers to minimise the risk of damage caused by alkali silica reaction (ASR) in New Zealand concrete.

It applies to concrete to be used in new construction. It does not apply to the repair or control of damage in existing structures.

The practice recommended herein applies to concrete containing:

- potentially reactive aggregates (i.e. aggregates that test as potentially reactive or are known to have reacted in site concrete, or aggregates of a type shown by laboratory tests or field performance to be potentially reactive); or

- aggregates of unknown reactivity.

It does not apply to concrete containing aggregates known to be non-reactive.

The New Zealand construction industry often refers to “alkali aggregate reaction” (AAR) instead of “alkali silica reaction” (ASR). ASR is the specific type of alkali aggregate reaction that is of concern in New Zealand. This document does not apply to alkali carbonate reactions (ACR), which to date have not been an issue in New Zealand because limestone has not been widely used as a concrete aggregate here.

1.2 Using this Information

Section 2 presents the practice recommended for minimising the risk of ASR in New Zealand concrete.

Two approaches are provided, one for Normal Concrete and one for Special Concrete (as defined by NZS 3104:2003). The approach for Normal Concrete assumes a 50-year specified intended service life and accepts the risk that minor ASR may occur. The specifier should consider designating the concrete Special Concrete if one or both of these assumptions are inappropriate for an individual structure.

1.3 Scope of 2003 Revision

This edition incorporates a revision of the text of the 1991 edition plus updated or additional information on:

- standard specifications;
- test methods for potential aggregate reactivity and concrete expansion;
- release of alkalis by New Zealand aggregates;
- alkali reactivity of New Zealand aggregates;
- effects associated with supplementary cementitious materials (SCMs), added either specifically to control ASR or to achieve other benefits;
- chemical admixtures;
- use of chemicals to reduce ASR;
• use of recycled wash water in concrete;
• features of specialty concretes that might affect the risk of ASR damage;
• aspects of design and environmental exposure that might affect the potential risk posed by ASR to a particular structure;
• effects of ASR on concrete and structures.

Significant changes are:

(a) **categorisation of concrete as Normal Concrete or Special** Concrete to bring it into line with the categories for concrete specification introduced in NZS 3104:2003. Slightly different approaches for minimising the risk of ASR damage are presented for Normal Concrete and Special Concrete.

(b) **shifting of responsibility for selecting the method of ASR** mitigation from the specifier to the concrete supplier for the majority of concrete construction. Suppliers are now responsible for minimising the risk of ASR expansion in Normal Concrete produced to NZS 3104:2003. For Special Concrete, the specifier identifies the risks associated with ASR on an individual structure, the supplier is responsible for proposing a solution appropriate for the level of risk, and the specifier is responsible for accepting or rejecting the proposed solution.

(c) **removal of detailed model specification clauses.** Although this demands that specifiers and suppliers understand the principles of ASR mitigation, it was felt that this approach would better allow specifications to incorporate future local and international developments, and would allow this document to focus on technical rather than contractual issues.

(d) **a risk-based approach to minimising damage on specific structures.** The provisions for Special Concrete allow precautions to be designed for individual structures so that they are appropriate for the reactivity of particular aggregates, the risk of ASR posed to the structure by its service conditions, its required service life, structural consequences of ASR, and aesthetic requirements.

(e) **the adoption of minimum levels of SCM additions without the need for their reactive alkali contents to be assessed.** Instead, the total alkali content or a proportion of it is considered when assessing concrete alkali contents, and there is an increased emphasis on the need for testing individual materials. This follows international developments in analytical techniques and the understanding of the effects of SCM’s on ASR.

(f) **updating of test methods.** A range of options is provided for testing aggregates and combinations of mix components and proportions.

The 1991 edition offered the following options for minimising the risk of ASR expansion:

• limiting the reactive alkali content of the concrete to a maximum of 2.5kg/m³ alkali equivalent. This was the usual approach where potentially reactive aggregate was available;
• using SCMs;
• using non-reactive aggregates;
• allowing potentially reactive aggregates in concrete containing more than 2.5kg/m³ alkali equivalent in conditions known not to result in ASR expansion.

The practice recommended in this revision follows the same principles as the 1991 edition but provides different methods to achieve them, reflecting findings from New Zealand and international research in the 1990’s and increasing interest in risk-based durability design to optimise cost and performance.
Lack of public research or widespread experience with SCMs and blended cements in New Zealand means that most of the updated recommendations are based on information from Canadian Standards Association (CSA, 2000), RILEM AAR-7 (2003), the Cement and Concrete Association of Australia (1996), the Concrete Society (of the UK) (1999), the Building Research Establishment (BRE, 1999 and 2002). A revised edition of the CSA standard is due to be published in 2005, and at the end of 2004 RILEM plans to publish the first draft of a guide to specification for minimising ASR damage (AAR-7). Suppliers and specifiers are encouraged to refer to the most recent editions of these publications.

1.4 Scope of 2012 Amendment

(a) Recommendations made in a report (Freitag, Bruce, Shayan, 2011) on ASR/DEF observed in the precast concrete piles of two bridges using South Island aggregates containing microcrystalline quartz or strained quartz, are outlined. These aggregates have been found to be potentially alkali-reactive, but act more slowly than fresh volcanic materials, and concrete containing such aggregates may need temperatures exceeding 70°C to produce deleterious ASR expansion, even if it contains sufficient alkali.

(b) Precautions to minimise the risk of ASR in concrete containing recycled concrete aggregate, based on CCANZ Technical Report TR 14 ‘Best Practice Guide for the use of Recycled Aggregates in New Concrete’ are outlined. The potential reactivity of recycled concrete aggregate shall be based on the potential reactivity of the parent concrete, the original concrete from which the exposed aggregate came. Recommendations on the alkali contribution from the recycled aggregates which contributes to the total alkali for assessing the risk of ASR are also given.
2.0 Recommended Practice for Minimising the Risk of ASR

The general principles behind the recommended practice are outlined in section 2.1. The practice is detailed in section 2.2. Normal Concrete and Special Concrete are as defined in NZS 3104:2003.

2.1 General Principles

The following steps outline the precautions necessary to avoid ASR damage for a particular concrete structure to be built in New Zealand.

(a) Determine the mineral composition of the aggregate by petrographic examination. *If the aggregate contains no potentially reactive components, these guidance notes are not required.* If it does contain potentially reactive components, even as contaminants, either assume the aggregate to be reactive and proceed to items (d) to (j), or carry out further investigation as outlined in items (b) and (c).

(b) If the aggregate contains potentially reactive material, then assess the aggregate’s reactivity from existing test data or field experience. If neither is available, assess its reactivity by ASTM C289. However ASTM C 289 may not be reliable in predicting reactivity of slow reacting aggregates (e.g. quartzite). Aggregates found to be innocuous by this method are highly unlikely to be reactive in concrete and can be treated as non-reactive. If ASTM C289 tests show the aggregate to be innocuous, these guidance notes are not required. Aggregates found to be potentially deleterious or deleterious are likely to react if sufficient moisture and alkalis are available.

(c) If ASTM C289 shows the aggregate to be deleterious or potentially deleterious, either assume that it is reactive and proceed to items (d) onward, or carry out mortar/concrete tests to establish its likely reactivity in concrete before taking appropriate action. If mortar or concrete tests show the aggregate to be non-reactive these guidance notes are not required.

(d) For Normal Concrete containing potentially reactive aggregate, apply a maximum concrete alkali limit of 2.5 kg/m3.

(e) For Special Concrete containing potentially reactive aggregate, identify the level of damage that will be acceptable on the structure from asset management and aesthetic considerations. (See Table 2).

(f) For Special Concrete, select the degree of precaution required against ASR from an evaluation of the above data. (See Table 3).

(g) For Special Concrete containing potentially reactive aggregate, identify whether the structure will be exposed to moisture conditions or other environmental factors that could increase the risk of ASR. (See Table 1).

(h) For Normal and Special Concretes, if an SCM is to be used to minimise ASR damage, check that it complies with an appropriate standard. The effectiveness of the SCM at one or more dosages can be demonstrated by the mortar and/or concrete tests used to measure aggregate reactivity.

(i) For Normal and Special Concretes, if an SCM is to be used for other purposes, and the aggregate is reactive, either carry out mortar or concrete tests to ascertain whether the proposed quantity of SCM will mitigate ASR expansion, or if the proposed quantity is lower than the recommended minimum for mitigating ASR expansion, estimate the amount of reactive alkali in the SCM and include it in the concrete alkali content.
(j) For Normal and Special Concretes, if the concrete will be exposed to moisture or high humidity, avoid aggregate blends that include aggregates that release alkalis (e.g. some Auckland basalts) and acid volcanics (e.g. Waikato River sand) in the sand fraction unless the basalt has been shown not to release significant amounts of alkali.

(k) For Normal or Special Concretes exposed to elevated temperatures over 70°C, either through accelerated curing or heat of hydration of mass concrete, DEF (Delayed Ettringite Formation) may occur in conjunction with ASR. DEF requires a source of moisture in addition to high temperature and causes damage to concrete similar to ASR.

Care should be taken with quality control measures in concretes containing reactive aggregates, and in particular the control of curing practices to ensure that concrete temperatures remain below 70°C. Such precautions alone may be sufficient to prevent ASR, even if alkali levels are above the threshold for ASR.

2.2 Recommended Practice

It is recommended that the approach described in this section be followed for New Zealand concrete construction. This practice follows principles used by the Canadian Standards Association (CSA, 2000), and RILEM (AAR-7). It stresses the need for testing to establish the reactivity of individual mixes containing SCMs. Although UK practices (BRE (1999), Concrete Society (UK) (1999)) are comprehensive and soundly based, they are designed for aggregates that are less reactive than New Zealand volcanic materials and therefore they should not be used in New Zealand without first demonstrating by mortar and concrete testing that they are appropriate.

2.2.1 Specification

The heading and clauses (a) to (c) below are provided as model specification clauses. Dialogue between specifier and supplier is implicit in the nature of Special Concrete, and therefore this procedure assumes that where necessary such discussions will include the measures to be taken to minimise the risk of ASR damage.

*Precautions for minimising the risk of ASR.*

(a) The reactivity of the fine and coarse aggregates proposed for use in a particular concrete shall be determined by petrographic examination, accelerated laboratory testing or field experience as described in section 6.1 of TR3 (2003).

(b) If the aggregate supplier and/or concrete producer can confirm that the proposed aggregates are non-reactive as defined in clause 6.1 of TR3 (2003) then no further precautions need be applied.

(c) If the aggregate supplier and/or concrete producer cannot confirm that the proposed aggregates are non-reactive as defined in clause 6.1 of TR3 (2003) then the following precautions shall be taken:

(i) For Normal Concrete as defined by NZS 3104:2003 the concrete producer shall certify that the total alkali content in the concrete shall not exceed 2.5 kg/m³ from all sources.

(ii) Where the concrete producer cannot certify that the total alkali content is less than 2.5 kg/m³ then the concrete shall be designated as Special Concrete. The specifier must be informed of the change in designation, and must take appropriate action.

(iii) For Special Concrete, the specifier shall evaluate and specify the risks associated with ASR according to clauses 2.2.3 and 2.2.4 of TR3 (2003), and shall specify the
appropriate level of precaution required to minimise the risk of ASR damage according to clause 2.2.5 of TR3 (2003). The supplier shall then select appropriate preventive measures according to clause 2.2.6 of TR3 (2003), and shall provide supporting evidence such as calculations and/or test results to demonstrate to the specifier that the selected preventive measures will be effective.”

2.2.2 Normal Concrete

The restriction of total alkalis to 2.5kg/m³ or less is based on a level of prevention suitable for a structure designed for a specified intended service life of 50 years according to the New Zealand Building Code, or 100 years according to the Transit New Zealand Bridge Manual (2003), and on which cosmetic damage is acceptable but damage that affects structural performance is not. If the structure is intended to have a service life shorter than 50 years or longer than 100 years, and if cosmetic damage would not be acceptable, it is recommended that the specifier designates the concrete as Special Concrete.

Aggregates that release alkalis should not be used with reactive aggregates in Normal Concrete but should be evaluated as a Special Concrete. However see section 7.6.1 for use of recycled concrete aggregate.

The process of assessing the risk of ASR in Normal Concrete is described in Figure 1.
Figure 1: Procedure for Assessment of ASR Risk – Normal Concrete

START

Assess Risk from Aggregates

Assess Risk from Alkalis

Petrographic Examination (6.2)

Potentially Reactive? (6.2.1-6.2.2)

Optional Testing:
- Chemical Test (6.4.1, Appendix D), and/or
- Mortar Tests (6.4.2), and/or
- Concrete Tests (6.4.3), and/or
- Field Experience (6.5)

No

Yes

Potentially Reactive? (6.3, 6.4, 6.5)

Do Not Use Potentially Reactive and Alkali Releasing Aggregates Together (except RCA)

Reject Potentially Reactive Aggregate

No

Potentially Alkali Releasing? (7.6)

Yes

No

Is Concrete Alkali > 2.5 kg/m³?

No

Yes

Select Concrete Material and Mix Design to Meet Specified Strength

Determine Alkali Contents of Materials as % Na₂O eq:
- Cement (7.2)
- SCM (7.3)
- Water (7.4)
- Admixtures and Pigments (7.5)

Calculate Alkali Content Of Concrete as kg/m³ (Appendix C)

No

Yes

Is Aggregate Potentially Reactive?

No

Yes

Is Alkali Released by Aggregate?

No

Yes

Select Appropriate Action

Or

Reduce Concrete Alkali Content

No Action Required

Or

Use Non-reactive Aggregate

Refer to Specifier as Special Concrete (2.2.3-2.2.6)

No Action Required

Is Concrete Alkali > 2.5 kg/m³?

No

Yes

Refer to Specifier as Special Concrete (2.2.3-2.2.6)
2.2.3 Special Concrete: Identification of the Risk of ASR Damage Associated with Environmental Effects

The process for assessing the risk and selecting preventive measures for Special concrete is described in figure 2.

The specifier identifies the risk of ASR damage related to environmental effects in accordance with Table 1. These categories take into account the size of the concrete element, the availability of moisture and external sources of alkali, and the effects of temperature. The environmental risk category is stated in the concrete specification. Section 4.2 gives further information on environmental effects.

Table 1: Environmental Risk of ASR Damage

<table>
<thead>
<tr>
<th>Environmental Risk Category</th>
<th>Size and Environmental Exposure of Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1</td>
<td>Non-massive(^1) and dry(^2), e.g. a damp-proofed floor in dry service conditions</td>
</tr>
<tr>
<td>E2</td>
<td>Massive and dry(^3,4), all concrete exposed to humid air, condensation, rain, run-off, groundwater, sea water(^4), or other sources of moisture.</td>
</tr>
<tr>
<td></td>
<td>e.g. building facades, foundations, concrete elements in a building enclosing a swimming pool or laundry, water-retaining structures.</td>
</tr>
<tr>
<td>E3</td>
<td>Concrete exposed to external moisture and to aggravating factors such as freezing and thawing, wetting and drying in a marine environment or prolonged elevated temperatures.</td>
</tr>
<tr>
<td></td>
<td>e.g. concrete in the splash zone of a marine structure; concrete exposed to moisture and elevated temperatures such as a cooling tower or chimney</td>
</tr>
</tbody>
</table>

\(^1\) A massive element has a least dimension of 0.5 m or more.

\(^2\) A dry environment corresponds to an ambient average relative humidity condition lower than 60% (normally only found inside buildings) and no exposure to external moisture sources.

\(^3\) A risk of alkali-silica reaction exists for massive concrete elements in a dry environment because the internal concrete may still have a high relative humidity.

\(^4\) A non-massive concrete element constantly immersed in sea water does not present a higher risk of ASR than a similar element exposed to humid air, buried in the ground, or immersed in pure water, because the alkali concentration of sea water (30 g/l NaCl, i.e. 0.57 M NaCl or Na) is lower than the alkali concentration of the pore solution of most concretes, and the penetration of Cl-ions is usually limited to a few centimetres.
2.2.4 Special Concrete: Identification of the Risk of ASR Damage Associated with the Structure

In accordance with Table 2, the specifier identifies the risk of ASR damage associated with the engineering, safety, aesthetic and economic effects of ASR damage and the service life of the concrete structure or element. The acceptable level of risk associated with these features is determined by the owner or authority responsible for the structure, in cooperation with the designer/specifier and sometimes also the contractor. The structure risk category is stated in the concrete specification.
The acceptable level of damage is determined by the structural performance required of the element, the ease with deterioration can be detected and managed, the significance of the structure’s appearance, and real and perceived public perceptions of safety. **The S2 risk level will be appropriate for most structures.** The S3 level was introduced for structures where not even cosmetic damage is acceptable. Situations where this might apply include:

- architectural concrete;
- prominent structures where cracking might arouse public concerns about safety;
- high value structures where extra costs associated with preventing ASR are acceptable;
- structures designed to remain in service for 100 years or more without significant deterioration;
- structures on which damage would not be detected during normal use and inspection of the structure and remediation of the affected element or structure would not be practical.

Reinforcement and prestressing will restrain expansion, but only in the direction of the reinforcement/prestressing. “Structure risk” considers the risk associated with the consequences rather than the likelihood of expansion because reinforcement/prestressing will not necessarily provide adequate restraint against expansion in all directions. Section 4.3 describes some of the possible effects of ASR expansion on structures.

### Table 2: Risk of ASR Damage Associated with Structure.

<table>
<thead>
<tr>
<th>Structure Risk Category</th>
<th>Features of Structure</th>
</tr>
</thead>
</table>
| **S1**                  | Some deterioration from ASR is acceptable  
                          e.g. non-load bearing elements inside buildings, temporary structures. Includes elements on which deterioration would be detected during normal use and inspection of the structure and that are easy and cheap to replace. These elements are likely to be designed for service life less than 50 years\(^1\). |
| **S2**                  | Minor ASR and resulting cosmetic cracking acceptable  
                          e.g. most buildings and civil structures (e.g. bridges). Includes elements on which deterioration might or might not be detected during normal inspection but where remediation of the element or structure would be possible if necessary. These structures would normally be designed for service life 50 or 100 years\(^2\). |
| **S3**                  | No ASR damage is acceptable, even if only cosmetic  
                          e.g. dams, tunnels and other major or prominent civil structures, structures retaining hazardous materials, nuclear installations, architectural finishes such as F6. Includes critical elements on which deterioration would not be detected in normal inspection and where remediation of the element structure would not be practical. These structures would normally be designed for service life 100 years or longer\(^3\). |

\(^1\) Service life in this context may be the specified intended life according to the New Zealand Building Code, or it may be the technical service life defined by NZS 3101 as the time at which a defined level of deterioration is likely to have developed.
2.2.5 Special Concrete: Identification of Level of Precaution

The specifier identifies the level of precaution required from Table 3. This represents the overall level of risk. The level of precaution is stated in the concrete specification.

Table 3: Level of Precaution Required to Minimise the Risk of ASR Damage.

<table>
<thead>
<tr>
<th>Structure Risk</th>
<th>Environmental Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Nil</td>
</tr>
<tr>
<td>S2</td>
<td>Low</td>
</tr>
<tr>
<td>S3</td>
<td>Standard</td>
</tr>
</tbody>
</table>

The level of precaution defined as “Standard” provides protection equivalent to the 2.5kg/m³ maximum alkali limit prescribed by the 1991 edition of TR3, and will be appropriate for most structures.

“Extraordinary” levels of precaution should only be specified in exceptional circumstances, and the reasons for this level of precaution in terms of environment risk and structure risk must be explained in the concrete specification.

2.2.6 Special Concrete: Identification of Preventive Measures

The supplier identifies appropriate preventive measures from Table 4, and provides supporting calculations and/or test results or other evidence to demonstrate to the specifier that the selected preventive measures will be effective.

Sometimes it will not be possible for concrete with alkali content less than 2.5kg/m³ to meet other specified requirements. Alkali contents of less than 1.8kg/m³ may be even more difficult to supply. Consequently it is anticipated that SCMs will often be used as a preventive measure for Special Concrete. Many Special Concretes will contain SCMs for other reasons and if they are already present in sufficient quantity no extra measures will be needed to minimize the risk of ASR damage. If they are present in smaller quantity than these guidelines indicate as necessary for ASR control, then their effectiveness in minimizing ASR damage will need to be assessed.

Guidance to the use of SCMs for managing ASR is given in sections 5.4 and 7.3.

Table 4 does not include lithium treatments as preventive measures because at the time of writing they are likely to be more expensive than the other options. However they may be used in accordance with international guidelines (section 5.6) if appropriate.
Table 4: Preventive Measures.

<table>
<thead>
<tr>
<th>Precaution Level (From Table 3)</th>
<th>Measures to Prevent Deleterious Alkali-silica Reaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nil</td>
<td>No special precautions against ASR damage are necessary. Ensure that the concrete is specified, supplied, placed and cured according to NZS 3101, NZS 3104, and NZS 3109.</td>
</tr>
</tbody>
</table>
| Low                            | Mild preventive action is required; use one of the following techniques:  
  L1: Reject the proposed aggregate or use option L2 or L3.  
  L2: Limit the alkali content of the concrete to <3.0 kg/m³ Na₂O eq¹.  
  L3: Use a sufficient amount of effective SCM or combination of effective SCMs to minimise ASR expansion. |
| Standard                       | Standard preventive action is required; use one of the following techniques:  
  N1: Reject the proposed aggregate or use option N2 or N3.  
  N2: Limit the alkali content of the concrete to < 2.5 kg/m³ Na₂O eq¹.  
  N3: Use a sufficient amount of effective SCM or combination of effective SCMs to minimise ASR expansion. |
| Extraordinary                  | Exceptional preventive action is required; use one of the following techniques:  
  X1: Reject the proposed aggregate or use option X2 and/or X3.  
  X2: Limit the alkali content of the concrete to < 1.8 kg/m³ Na₂O eq¹.  
  X3: Use a sufficient amount of effective SCM or combination of effective SCMs to minimise ASR expansion. |

¹ If the concrete contains SCM in sufficient quantity to effectively reduce ASR expansion then alkali from the SCM does not need to be included. If SCM is added in smaller quantity than required to effectively reduce ASR expansion, or if its effect on ASR is unknown then the following proportions of SCM alkali equivalents shall be included: (Similar information is also given in 5.4, page 26. These should be consistent with section 7.3).

<table>
<thead>
<tr>
<th>Material</th>
<th>Proportion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blastfurnace slag</td>
<td>50%</td>
</tr>
<tr>
<td>Fly ash</td>
<td>20%</td>
</tr>
<tr>
<td>Metakaolin</td>
<td>0%</td>
</tr>
<tr>
<td>Silica fume</td>
<td>100%</td>
</tr>
<tr>
<td>Geothermal silica</td>
<td>100%</td>
</tr>
<tr>
<td>Natural pozzolan</td>
<td>100%</td>
</tr>
</tbody>
</table>

Allowance shall be made for likely variations that will occur in the alkali content of the cement and for variations that will occur in the cement content of the concrete.

Blended cements are to be treated as Portland cement plus SCM.
3.0 ASR in New Zealand

This section deals specifically with ASR in New Zealand. For more general information on ASR, refer to Section 4.

3.1 History of Research into ASR in New Zealand

In 1939, Thomas Stanton (1940) and co-workers at the California Department of Transport discovered that unexplained cracking in concrete that had been occurring in the USA for over two decades was caused by certain aggregates reacting with cement alkalis. They named this phenomenon alkali-aggregate reaction (AAR). The initial work of Stanton was quickly expanded by other workers (Mielenz & Witte, 1948) at major US agencies such as the Bureau of Reclamation and the US Army Corps of Engineers.

In 1943, engineers from the New Zealand Public Works Department (PWD) were seconded to the USA to gain experience for the construction of hydroelectric power stations in the Waikato River valley. US experience with ASR was relevant to New Zealand (Hutton, 1945), as some of the reactive volcanic rocks present in the USA are similar to those found in the Waikato River valley. In 1947, PWD engineers were advised to avoid using reactive aggregates with Portland cements containing more than 0.6% alkalis, and to seek petrographic examinations of aggregates they were uncertain about (Langbein, 1947).

Between 1945 and 1960, the Public Works Department assisted by the Dominion Laboratory of DSIR, investigated aggregates for concrete for the Waikato River hydroelectric projects. Low-alkali cement was used in many of the structures and this is believed to have minimised the incidence of damage from ASR (Dekker & Scott, 1969). Pozzolans were introduced to improve concrete workability and reduce drying shrinkage in the latter stages of the project, and this further reduced the risk of ASR damage.

Over the next two decades, the Dominion Laboratory investigated the reactivity of many New Zealand aggregates (Kennerley & St John, 1969). Petrographic techniques were developed to allow the analysis of hardened concrete undergoing ASR (Abbott and St John, 1983).

In the 1980's and 1990's, the New Zealand Ministry of Works and Development [followed by its successors Works and Development Services Corporation (NZ) Ltd (WORKS), now Opus International Consultants Ltd (Opus)], assisted by DSIR Chemistry [now Industrial Research Ltd (IRL)], investigated the incidence of ASR in structures in areas of the North Island where reactive aggregates were available (Freitag & Rowe, 1987; Rowe, Smith & Freitag, 1989; Freitag, 1994). This work identified structures with minor ASR that would not have been reported during routine maintenance surveys, and provided important field data on alkali reactivity of aggregates (Doyle, 1988; Freitag, 1990; St John, 1988, 1988). The 1991 edition of these guidance notes incorporated much of this information.

Research by WORKS/Opus and IRL in the 1990's concentrated on the reactivity of New Zealand greywackes, the potential use of rapid mortar bar tests for aggregate reactivity, and the release of alkalis by aggregate (Freitag, St John & Goguel, 2000; Freitag, 1998, 2002; Freitag & St John, 1996; Freitag, St John & Goguel, 2000; Goguel, 1995; Goguel & Milestone, 1997, 2000). It also involved developing analytical techniques for investigating the chemistry of hardened concrete (Goguel & St John, 1993; Goguel, 1995; Goguel & Milestone, 1997, 2000).

3.2 Damage to New Zealand Structures

A combination of factors (see section 3.3) has meant that damage in New Zealand structures due to ASR is usually minor compared to that observed overseas. However, evidence from investigations (Freitag, 1994, 2002) indicates that minor ASR can occur even where low-alkali cement and pozzolan have been used and concrete alkali contents are less than 2.0kg/m³. The extent of ASR observed indicates that given the highly reactive nature of some New Zealand volcanic rocks, the reaction has the potential to cause significant damage in concrete structures unless adequate controls are maintained.
In the 1980's and 1990's, over 500 structures were inspected in several North Island areas where potentially reactive aggregates are available. Signs of ASR were seen in over 10% of the structures inspected in the Taranaki and Tongariro regions, in about 10% of the structures in the Waikato and North Auckland regions, and in fewer than 5% of the structures in the Coromandel and Bay of Plenty. Most of the damage observed was minor, and in some cases the cracking that prompted analysis of the concrete might have been caused by other mechanisms. Most of the affected structures, including the more severely damaged ones, were built when high alkali cement was available. However, these inspections only included readily accessible concrete. ASR damage could be more common and/or severe in buried concrete that is in contact with moisture in the ground (Freitag & Rowe, 1987; Rowe, Smith & Freitag, 1989; St John 1988, 1988; St John, 1989).

Damage is normally confined to concrete where one or more sides are exposed to moisture. Visible damage usually takes the form of areas of pattern cracking or isolated cracks not obviously due to structural causes or normal concrete behaviour. Only in the more advanced cases is extensive pattern cracking observed and apart from the darkening of crack margins, it is rare to observe alkali-silica gel on the surface of the concrete. SHRP C315 gives guidance on identifying ASR in highway structures. Section 4.3 describes potential mechanical effects of ASR on concrete and structures. Most cases of ASR in New Zealand have shown some external symptoms within ten years, and signs of the reaction often develop within five years.

As a consequence of precautions taken to date, few structures in New Zealand have been seriously damaged. In one bridge, the prestressed deck beams were replaced because of possible delamination. In a water storage reservoir, the floor had cracked and expanded allowing leakage and so required replacement. In several other structures, expansion and cracking of foundation concrete reduced the anchoring capacity. While the other seriously affected structures have not been replaced, maintenance has been required. A portion of a large concrete pavement had been expanding for twenty-five years, necessitating renewal of drains and repairs to abutting structures, and other affected structures have required minor repairs. In general, the expansive movement of ASR in New Zealand structures is potentially more serious than the cracking because of closure of joints and damage to abutting structures.

ASR expansion is restrained by reinforcement and prestressing. Because most significant structures in New Zealand are heavily reinforced for seismic reasons, the risk of structural damage is relatively low.

In 2006, routine inspections on two Southland bridges with a pile bent substructure revealed significant cracking, spalling and surface erosion of precast prestressed piles below high tide level. An investigation was undertaken by Opus International Consultants which showed that ASR and DEF (Delayed Ettringite Formation) had taken place and the damage was caused by the interaction of two chemical reactions in the concrete that caused it to expand. This type of deterioration, referred to herein as ASR/DEF, is not easily detected or identified until it is well advanced which may be significant for buried or immersed parts of piles, which are not often closely inspected. In general ASR precedes DEF except in rare cases.

Specialist inspections were then carried out of 51 South Island bridges with pile bent substructures and two wharves. This revealed very few cases of damage resembling ASR/DEF and cracking was revealed on only six structures. Cores samples taken from Southland/Otago and Nelson/Marlborough showed ASR present in both cracked and uncracked piles, but associated damage was generally minor. The findings suggested that ASR/DEF may only be associated with the use of curing temperatures higher than 70°C. Concrete in damp or wet conditions increases the risk of ASR/DEF.

Up until 2006, no ASR has been observed in the South Island even though it was recognized that potentially reactive aggregates were present in the South Island. The presence of elevated concrete temperatures in the construction of the two Southland Bridges would therefore appear to be a mitigating factor in the deterioration observed.

### 3.3 Differences Between ASR in New Zealand and Overseas

Several aspects of ASR in New Zealand differ from the effects of the reaction observed overseas. These are:
(a) **Freeze-thaw damage:** Apart from some elevated and inland areas, freeze-thaw attack on concrete is rare in New Zealand and those few cases observed are usually minor. The severe damage due to the expansive freezing of water that has percolated into the cracks of concrete undergoing ASR has not been observed in this country. Similarly, de-icing salts are rarely used, so ASR is not exacerbated by this external source of alkali.

(b) **Salt-spray:** A large portion of New Zealand is subject to deposition of salt spray (Balance and Duncan, 1985) carried in the prevailing winds. However, most areas receive sufficient rain throughout the year to wash salt off exposed surfaces. Thus augmentation of alkalis from deposited salt-spray is restricted to sheltered parts of structures and does not contribute significantly to ASR (see (c) below, and also section 4.2.3).

(c) **Humidity and condensation:** Atmospheric relative humidity is commonly between 70-80% in coastal areas and approaches 60% inland. These conditions, combined with a high incidence of wind, effectively prevent serious condensation on the surfaces of structures that could otherwise increase the risk of ASR.

(d) **Low-alkali cement:** Since the 1950’s, low alkali cements have been used in all major public construction, with replacement of up to 15% of the cement by pozzolan. Although there has been no mandatory limit on cement alkalis, almost all cements manufactured in New Zealand since 1970 (St John, 1988, April) have had alkali contents less than 0.6% Na₂O equivalent. NZS 3122: 1995 allows for an alkali content of less than 0.60% Na₂O equivalent to be specified if the cement is to be used with potentially reactive aggregate. Low-alkali cement has thus been the norm for use with reactive aggregates, unlike overseas where low-alkali cement is often not readily available. This is the principal reason for the low incidence of ASR damage in New Zealand structures. The use of low-alkali cement in New Zealand has minimised the damage due to ASR, and provides a ready means of minimising the problem in the future. However, higher-alkali cements (produced locally or imported) are likely to become available in New Zealand in the future because their production is cheaper and/or less energy intensive. If such cements are used, the recommendations in this document must be followed.

### 3.4 Significance and Future Management of ASR in New Zealand

Although ASR damage in New Zealand to date has not been structurally significant, the cracking it causes can be unsightly. The cracking can also increase the permeability of the concrete and thus increase the risk of other types of deterioration such as freeze-thaw damage and reinforcement corrosion. When permeability-based models are used in durability design, cracking caused by ASR expansion could compromise the designed service life.

The practices adopted to date to minimise ASR damage in New Zealand have generally been successful. There has been little publicity about the few significant cases, and other cases have not required maintenance. As a result, there is a perception in the concrete construction industry that ASR is not a problem in New Zealand. Although we know how to avoid major ASR damage, successful management of the reaction in this country in the future will require the following issues to be recognised:

- Perceptions within the industry that “ASR is not a problem” ignore the fact that this is because precautions were taken in the past to prevent it becoming a problem. It will become a problem if industry neglects to take appropriate measures to control it in the future;

- Information required to minimise ASR damage is not always widely disseminated throughout the industry;

- Research has found evidence of ASR in concrete containing low alkali cement and pozzolan, indicating that care must be taken to establish appropriate dosage rates of supplementary cementitious materials (SCM) to achieve an acceptable level of ASR control;
New Zealand’s acid volcanic aggregates are reactive and will continue to be used as in some areas they are the most economic materials available;

Our knowledge about ASR in New Zealand is largely based on laboratory testing carried out before the introduction of the SCMs in widespread use today, and on limited site investigations of easily accessible concrete.

The simplest means of addressing these issues is to restrict the use of cements with alkali contents above 0.60% when potentially reactive aggregates are used, but this will not always be economic and does not always work. The most appropriate precautions for minimising the risk of damage for any given structure will be determined by the cost and availability of alternative aggregates and cementitious binders, the amount of damage considered acceptable, and the risk of this damage occurring. This requires greater understanding of the mechanisms involved in the reaction and its control.

These guidance notes attempt to redress some of the problems noted above. They are based on the current experience with New Zealand materials and will require modification as new information on ASR becomes available. Overseas publications such as guidance notes by Cement and Concrete Association of Australia/Standards Australia, the UK Concrete Society, the Building Research Establishment (BRE), and the Canadian Standards Association (CSA) provide a good understanding of the issues surrounding the management of ASR but the details specified may not be entirely appropriate for New Zealand aggregates and conditions. The European organisation International Union of Testing and Research Laboratories for Materials and Structures [RILEM] has convened a committee to develop universal guidelines to minimise the risk of ASR damage based on the contributions of contributing members’ countries. The guidelines presented in this edition of TR3 are based on an early draft of these RILEM recommendations (RILEM, 2003).
4.0 Alkali-Aggregate Reactions

4.1 General Considerations

Alkali-aggregate reaction (AAR) is the expansive reaction between the alkaline pore solution of concrete and certain minerals in the aggregates. The principal source of alkalinity in the pore solution is from the cement itself, but any source of sodium or potassium can contribute to the reaction provided that the alkali can move from its source into the pore solution of the concrete.

AAR has been divided into two types of reaction depending on the types of minerals involved:

(a) **Alkali-silica reaction (ASR):** the reaction between the alkaline pore solution and silica minerals such as opal, chalcedony, micro and cryptocrystalline quartz, cristobalite and tridymite and volcanic glasses. The reaction produces alkali-silica gels that take up water and expand in moist conditions (see Appendix B).

Sands wholly or partly derived from acid/intermediate rocks are the principal cause of ASR in New Zealand. Opal and chalcedony are rare in New Zealand concrete aggregates but volcanic glasses and to a lesser extent, cristobalite and tridymite are widespread in the groundmass of our acid/intermediate volcanic rocks.

Overseas, ASR has been observed with aggregates such as quartzite, greywacke, argillite, hornfelsed shale, phyllite, granite, and gneiss. Reaction with these minerals was once distinguished as “alkali silicate reaction”, but is now believed to be ASR involving micro- and cryptocrystalline quartz.

(b) **Alkali-carbonate reaction (ACR):** the reaction between the alkaline pore solution and argillaceous dolomitic limestones containing clay impurities. Two mechanisms contribute to expansion of the concrete: crystallisation of brucite and calcite during the “dedolomitisation” reaction between dolomite and the pore solution, and the adsorption of alkalis by the clay. No expansive alkali-silica gel is formed in this reaction.

Dolomitic aggregates are not used in New Zealand and currently there are no test data on New Zealand dolomitic rocks.

*This document applies only to ASR because the risk of ACR in New Zealand is very low.*

The damage due to ASR is caused by expansion of alkali-silica gel that forms during the reaction. This gel absorbs water and swells, exerting pressure that may be sufficient to expand and crack the concrete.

A SR requires three factors to proceed:

(a) Sufficient moisture in the pore structure of the concrete. ASR damage is unlikely to occur when the equilibrium internal relative humidity in the concrete is less than 75%.

(b) Sufficient alkali in the pore solution. Alkali can be supplied by cement and other binder constituents, chemical admixtures and/or the aggregate. Alkali contents can exhibit a pessimum proportion in some concretes.

(c) **Reactive mineral(s)** in the aggregate. Some reactive aggregates will only cause significant expansion if they are present in a critical amount known as a pessimum proportion (see section 6.3). This is the proportion of reactive aggregate at which greatest expansion occurs.

If any one of these three factors is absent, then ASR will not proceed. Once all available water or alkali is used up the reaction will stop, but it may recommence if the conditions (a) and/or (b) are again satisfied.
Once ASR has started in a structure, there is no way of stopping it if sufficient moisture remains in the concrete. It will continue until the alkalinity of the pore solution falls below the required concentration for reaction, or until the reactive component of the aggregate is exhausted.

Repairs to affected structures are generally limited to relieving stresses due to expansion of concrete and strengthening by tying or propping. Overseas literature indicates that impregnation with lithium salt solutions can help alleviate future expansion (Federal Highway Administration [FHWA], 2003). However the treatment requires that the concrete be fully impregnated with the lithium solution, which would be difficult to achieve with the high quality concretes that are often affected. Experience in New Zealand is too limited to make any recommendation on its use. Treatments based on restricting the supply of water to the concrete will not work unless they prevent water ingress from all sources and allow water already in the concrete to evaporate.

The recommendations in this document designed to minimise the risk of damage from alkali-silica reaction in new concrete construction are based on eliminating at least one of the factors (a), (b) or (c) above.

4.2 Environmental Effects on ASR

4.2.1 Temperature

In moist conditions, ASR expansion is faster at high temperature but can stabilise earlier (Olafsson, 1987). At lower temperatures, the expansion is slower but proceeds for longer. Temperatures of 38°C and above are used to accelerate expansion in laboratory tests. Although temperature effects could cause different amounts of expansion on different parts of a structure, in the New Zealand climate they are usually less significant than the availability of moisture (4.2.2). The main exception to this is industrial structures such as chimneys that are exposed to elevated temperatures.

The presence of ASR/DEF found in precast prestressed concrete piles in the two Southland bridges was investigated with accelerated mortar bar and concrete prism testing. This showed that aggregates from Oreti Beach and nearby alluvial deposits are potentially alkali-reactive, but may react slower than fresh volcanic materials and concrete temperatures may need to exceed 70°C to produce deleterious expansion, even if the concrete contains sufficient alkali.

4.2.2 Moisture

As explained in section 4.1, ASR requires a minimum relative humidity in the concrete. Internal relative humidity will vary with the environment of the structure and will influence the amount of protection required against ASR. Testing has indicated that where a concrete undergoing ASR is kept moist, the reaction will proceed continuously.

Moisture may be derived from high atmospheric humidity, proximity to waterways, retained water, or exposure to groundwater or runoff that drains over or ponds on the concrete surface.

ASR cracking is caused by tension generated by expansion of the internal concrete rather than by reaction of the 75-100mm surface layer of concrete. Thus it is the moisture condition of the internal concrete that must be considered. The moisture content of internal concrete in elements with large cross section is less sensitive to wetting and drying of the surface than is the internal concrete in smaller elements.

In most parts of New Zealand, temperatures are mild and rainfall is distributed throughout the year. This climate allows the outer concrete to dry but leaves sufficient moisture inside the concrete for ASR to proceed.

In a low humidity environment, the internal portions of concrete can dry out. Reports from overseas (Stark, 1985) indicate that even under desert conditions, internal moisture loss is slow and concrete members with large cross sections may retain high levels of internal moisture for more than twenty years. This indicates that although concrete not exposed to external moisture sources is at low risk from ASR, it
cannot be assumed that such concrete will dry sufficiently to prevent the reaction. Poorer quality concretes that are less dense and more porous will dry more rapidly and will be less prone to ASR than higher quality concretes in a dry environment.

One of the most aggressive environments in which ASR can proceed is hot, dry conditions during the day and cold conditions at night where the temperature drops below the dew point and water condenses on the exposed surfaces (Foakes, 1980). These conditions accelerate ASR, and if salts are also deposited on these surfaces, rapid disintegration of the concrete may occur due to combined expansion from ASR and penetration of chloride causing corrosion of reinforcement. These types of climatic conditions are rare in New Zealand.

Alkalis are likely to be leached from concrete immersed in flowing water, reducing the risk of ASR.

Buried concrete such as piles or foundations can be exposed to continuously damp environments with little or no flowing water, which presents a high risk. The risk associated with such concrete is further increased because inspection is usually difficult or impossible.

### 4.2.3 External Sources of Alkalis

As explained in section 4.1, for ASR to proceed there must be both enough moisture and sufficient alkali in the pore solution. It is possible for a concrete to be mixed with insufficient alkali for any significant ASR to proceed, and subsequently to have the environment provide enough extra alkali for the reaction to develop.

In many countries with severe winters, sodium chloride has been used as a de-icing salt. Ingress of this salt into the concrete may increase alkali concentrations (Rogers, 1987). Where wind deposits salt under dry conditions and water condenses on the concrete surface at night, salt is rapidly absorbed into the concrete, especially if the concrete is permeable (Foakes, 1980). Neither of these conditions occurs in New Zealand. However, the ingress of salt from spray-laden wind should always be considered as a potential source of external alkali.

Sulphate rich ground waters can also contribute alkalis to the concrete. In New Zealand this is only a problem in some geothermal areas where sulphate levels in the ground water are high. In such cases the concrete needs protection from both ASR and sulphate attack (Kennerley, 1980).

The alkali metal ions sodium and potassium are mobile in solution, so any movement of moisture through concrete will move alkalis, depleting them in some parts and concentrating them in others (Nixon, Collins & Rayment, 1979). Evaporation of water from a drying surface when a second surface is in contact with a supply of moisture (foundation walls for example) can thus concentrate alkalis near the drying surface. Similarly, salts deposited on a dry, exposed surface will move into the concrete once the surface is wetted, and alkalis may be leached from concrete surfaces that are frequently wetted.

Salt spray does not appear to contribute significant alkali to concrete in New Zealand conditions. In 1991, analyses of chloride contents in coastal Taranaki bridges built in the 1970’s recorded up to 0.04% chloride by weight of concrete at the surface, diminishing to 0.01-0.02% at 40mm depth. Similar figures were obtained from a concrete pavement at Whenuapai air base, Auckland. In another 1970’s coastal Taranaki structure, up to 0.2% chlorides were recorded at the surface and 0.03-0.04% chlorides were recorded at about 50mm depth. Bridges in the Tongariro region contained 0.00-0.01% chloride at the surface and at approximately 40mm depth. The contributions from the chlorides in the aggregates are not known, although geochronological knowledge suggests that such contributions from New Zealand greywackes and volcanic rocks are probably insignificant. A chloride content of 0.02% is equivalent to around 0.2kg/m³ of sodium if it is assumed that all chloride is associated with sodium. An increase in alkali content of this order over 20 years is unlikely to be significant.

There are few reports on the effect of immersion in seawater. Oberholster (1994) observed that units of a particular concrete had cracked in the tidal zone and above, but were not cracked below the tidal zone.
The concentration of alkalis in seawater is less than in the pore solutions of most concretes, so exposure to seawater will only increase the concrete alkali content if the concrete is exposed to wetting and drying.

Recent Canadian work found that exposure to sodium chloride did not change the chemistry of the pore solution at depths greater than 60-80mm from the surface, and suggested that exposure to seawater or de-icing salt would not cause expansion in concrete with low alkali content (Berubé, Dorion, Duchesne, Fournier & Vezina, 2003). It was suggested that the formation of chloroaluminates could increase the overall expansion of high alkali concrete in the long term under these circumstances, although the mechanism is not fully understood.

The lower the water to cement ratio of the concrete, the greater its resistance to the ingress of alkalis from external sources.

4.3 Effects of ASR on Concrete and Structures

Although cracking is usually considered to be the main effect of ASR, it can affect structural behaviour. The Cement and Concrete Association of Australia (1996), and BRE (1993) describe the potential effects of ASR on structural performance, and the Institute of Structural Engineers (1992) provides methods of assessing the effects.

Concrete compressive strength may be reduced, but the principal effects on concrete properties are a reduction in tensile/flexural capacity, shear strength and elastic modulus. Concrete failures may become more ductile as a result of the reduced elastic modulus.

The nature and extent of ASR cracking will be determined by restraint provided by adjacent elements, reinforcement and prestressing. Cracks will only develop where expansion is not restrained, with cracks typically parallel to the direction of restraint. The “pattern cracking” considered characteristic of ASR only occurs in unreinforced or lightly reinforced concrete. Cracking may be restricted to the outer (cover) concrete, with the interior of the concrete more typically microcracked in the region of expanding particles.

Well-anchored and confined reinforcement will effectively post-tension the concrete at normal levels of expansion. Excessive concrete expansion can either increase or reduce the bond between concrete and reinforcement depending on the reinforcement detailing. Restraint by adjacent elements will have similar effects. Differential restraint of ASR expansion within an element can concentrate cracks, leading to delamination.

The behaviour of individual elements will largely be determined by restraint of expansion provided by reinforcement. Slabs can delaminate if top and bottom reinforcement is not tied together. In columns, the cover concrete may delaminate and be unable to resist compressive loads. Beam flexural and shear capacities are not generally affected provided the beam is adequately reinforced.

ASR expansion may affect the performance of the entire structure by imposing lateral forces on adjacent members in a framed structure, inducing bending moments in non-symmetrically reinforced elements, closing movement joints and consequently damaging adjacent elements, increasing punching shear on a slab supported by an expanding pier or column, increasing tensile strains in reinforcement, changes in bond strength, changes in the articulation of the structure.

Serviceability may not be affected directly by ASR, but expansive cracking of cover concrete may increase the risk of reinforcement corrosion by providing a route for water, air and chlorides to reach steel reinforcement or prestressing. It will also increase susceptibility to freeze-thaw attack. Microcracking presents less of a risk because it is not continuous. With very wide cracks there is the risk of pieces of surface concrete falling away from edges and corners, although this is only likely where damage is exacerbated by other mechanisms.

The structural and serviceability effects of ASR expansion on an individual element are difficult to predict accurately. Because deterioration may be unsightly and also progressive, users may repeatedly raise
Alkali-Aggregate Reactions

corn about damage. For these reasons it is more practical to minimise the risk of ASR damage at the
time of construction, rather than to accommodate it in the design or to assess the significance of the
effects if and when deterioration becomes evident.
5.0 Principles of Minimising the Risk of ASR

5.1 General Considerations

The guidelines given in this document for Normal Concrete assume that a service life of 50 years (as defined by the New Zealand Building Code or NZS 3101) or 100 years (as specified by Transit New Zealand Bridge Manual, 2003 or NZS 3101) is required unless otherwise specified. Options for shorter or longer service life are also given in section 2.

It must be emphasised that ASR is only one phenomenon that can affect the durability of concrete. Precautions taken to minimise ASR damage must not compromise other aspects of concrete performance or durability, so the effect of materials and mix proportions on other properties of the concrete (e.g. workability, strength development, heat evolution, carbonation resistance, colour, shrinkage and creep) should also be considered.

5.2 Aggregate Selection

The first and most obvious step in minimising the risk of ASR is to identify whether potentially reactive aggregates are to be used in the concrete. Thus knowledge of the mineralogy of the aggregates is essential (Watters, 1969).

Information on the reactivity of New Zealand aggregates is given in section 6. If an aggregate is known to be non-reactive from previous testing or field experience, the precautions outlined in these guidance notes need not be applied. However, aggregate types for which test results and field data are inadequate or lacking pose a problem. These guidance notes assume that unless field or test data indicate an aggregate to be non-reactive, it is the responsibility of the specifier either to require the concrete supplier to show the aggregate to be non-reactive, or to take appropriate steps to minimise the risk of ASR.

Where an absolute guarantee is required that no ASR will occur, it will be necessary to use aggregates with a long history of no reaction in concrete and, as an additional safeguard against the possibility of unknown reactive aggregates, to limit the alkali content of the concrete. As a further safeguard, a supplementary cementitious material (SCM) will also reduce the likelihood of any reaction with the pore solution.

Where the only economic aggregates available contain reactive components, the acceptable level of risk of ASR and the acceptable degree of damage must be decided upon. One option to minimise the risk of damage is to limit the reactive alkali content of the concrete as described in sections 5.3 and 7. Restriction of the reactive alkali content in a concrete by itself may not prevent ASR, but will minimise cracking and expansion. A second option, which provides greater protection, is to use with the Portland cement an SCM known to inhibit ASR, as described in section 5.4.

5.3 Controlling Alkali Contents in the Concrete

The maximum concrete alkali levels recommended in this document as a method of minimising ASR are based on an evaluation of New Zealand and international test and field data. They provide a compromise between practical demands on components of concrete and minimising the probability and magnitude of damage due to ASR. While setting a particular alkali level will not necessarily prevent ASR, it should ensure that damage is minimised. It is the easiest method of control for the construction industry to apply and has effectively been used in a simplified form in New Zealand since 1970.

It has often been thought that the reactive alkali content of the concrete can be controlled by simply specifying low-alkali cement (cement with alkali equivalent less than 0.60%). This approach stems from USA laboratory and field tests in the early 1940's that found a 0.6% cement alkali limit to be an acceptable compromise between economy and risk of ASR damage (Hill, 1996). It assumes that:
- low alkali cement is available and economic to use;
- the maximum cement content of concrete does not exceed 400 kg/m³;
- alkali contribution from other components or environmental sources is negligible; and
- the limit is appropriate for all aggregates.

Such an approach should not be relied on unless the structure is at low risk from environmental factors, and unless minor cracking or a short service life is acceptable. This document does not advocate such an approach and recommends instead considering alkali content from all sources, not just from the cement.

Concrete alkali content can be controlled by reducing the cement content, using a low-alkali cement and/or replacing part of the cement with a SCM of a lower alkali content (SCMs offer additional advantages as described in section 5.4).

The total binder content of the mix may differ from that specified or determined by trail mixes. Normal fluctuations about the assumed figure will vary according to the conditions under which the concrete is mixed and may be as much as 10%. Variations in binder content must be minimised when reactive aggregates are used. The alkali content of a particular cement will vary from batch to batch, and cement analysis also has an inherent variability. Variations in cement alkali content and methods of accommodating them are discussed in section 7.2.

Several countries have adopted a concrete alkali limit of 3.0 kg/m³, while others have adopted a range of limits to accommodate the reactivity of different aggregates. The UK Concrete Society (1999), and BRE (1999) set limits on concrete alkali content that are adjusted for aggregate reactivity, for the alkali content of the cement and other components, and for the amount of SCM added.

The 2.5 kg/m³ limit on concrete alkali set in the 1991 edition of TR3 was based on the observed behaviour of New Zealand concrete above ground and exposed to cycles of wetting and drying. Most of the concrete on which this limit was based did not contain SCM added to reduce ASR damage; therefore in most circumstances this limit will provide a satisfactory level of protection against ASR damage. In exceptional circumstances it may be appropriate to use other limits that account for the risk of damage and the level of damage that is considered acceptable. A higher limit could be appropriate for concrete not exposed to moisture or where minor cracks would be acceptable, e.g. a temporary structure or an inland structure that is above ground and away from public attention. A lower limit might be more appropriate where:

- no cracking is acceptable, e.g. on a marine structure because of the risk of corrosion due to chloride penetration, or on an architectural feature;
- the aggregate could contribute significant alkalis after the concrete has hardened;
- environmental conditions increase the risk of expansion, e.g. concrete in a hot, damp environment; or
- extended service life is desired (e.g. greater than 100 years).

The practice recommended in section 2 for Special Concrete adopts different maximum levels of concrete alkali content based on the aggregate, the environmental risk, and the service life required.

Section 7 describes how to ascertain the alkali content of individual concrete constituents.
5.4 Supplementary Cementitious Materials (SCMs)

SCMs were referred to as mineral admixtures in the 1991 edition of this document.

SCMs (excluding fillers) can be added to concrete with Portland cement to reduce the cost of concrete, to enhance its fresh and hardened properties, and to reduce the risk of ASR. In this document, only their use for the control of ASR will be considered. If they are already proposed for use in a concrete for other purposes, their effectiveness at the proposed addition rate should be assessed.

Diatomite, pumicite and natural mixtures of pumicite and diatomite were used successfully in concrete in New Zealand in the 1950’s and 1960’s during hydroelectric power station construction (Kennerley & Clelland, 1959; Kennerley, 1959; Smith, 1977). Silica fume and cement containing blastfurnace slag were introduced to the New Zealand market in the 1990’s. Proprietary geothermal silica and metakaolin products were introduced in the late 1990’s. Fly ash from Huntly and from overseas sources has been used as a minor component of Type GP cement since the mid 1990’s, and other imported fly ashes have been used to a limited extent as a cement replacement in concrete since the late 1990’s.

5.4.1 Use of SCMs for Minimising ASR.

Although their predominant use in New Zealand has been to improve other aspects of concrete performance, SCMs can be used specifically to reduce ASR expansion in concrete containing reactive aggregate. Several mechanisms contribute to this effect, their relative importance depending on the concrete composition and the nature and amount of the SCM:

- The total alkali content of the concrete will be reduced if the SCM has a lower alkali content than the cement it replaces. Not all the alkali in SCM is necessarily available to enter the pore solution, so the reactive alkali content of the concrete may be reduced even if the total alkali content is not;

- Reaction of the SCM with the calcium hydroxide produced during cement hydration will reduce the calcium hydroxide available to maintain the very high pH necessary for ASR;

- The product of the reaction between SCM and calcium hydroxide binds alkalis so that they are unavailable to participate in ASR. This retention is considered to be largely due to aluminium in the structure of the silicate reaction product (see comment below on silica fume).

- The concrete will be less permeable, reducing the ingress of moisture and the diffusion of alkalis to reactive minerals;

- The concrete may be stronger and better able to withstand expansive forces without cracking.

Silica fume is a standard component of high alkali (1.50% Na₂O equivalent) Icelandic cements (Asgeirsson & Gudmundsson, 1979; Asgeirsson, 1986; Gudmundsson & Olafsson, 1996) into which it is interground at the level of 7.5% to prevent ASR damage. In countries other than Iceland, the long-term effectiveness of silica fume in avoiding ASR is being questioned. Silica fume is useful mainly because it is effective in reducing permeability of the concrete. Its high reactivity means alkali is bound quickly into the reaction product, reducing the alkalinity of the pore solution and preventing ASR. However, as the alkalinity in the pore solution is reduced, calcium ions become more mobile and in the long term this could lead to the conversion of sodium silicate to calcium silicate. As a result, alkalis could then again become available and attack reactive minerals of the aggregate. This consideration led Bérubé & Duchesne (1993) to see the role of silica fume in terms of postponing rather than avoiding ASR. Another issue is that some silica fumes are difficult to disperse, with the result that the particles of silica fume often remain as agglomerates in the concrete. These agglomerates can act like reactive aggregates, causing local expansion within the concrete if the alkali level is sufficiently high. Some silica fumes can contain significant amounts of alkali (see section 7.3). Boddy, Hooton & Thomas (2003) found that the ability of silica fume to control ASR decreased with decreasing silica content, and that for silica fumes with lower
silica content than that specified by ASTM C1240, typical replacement levels did not adequately control ASR expansion. The same is likely to apply to other amorphous silicas.

**Natural pozzolans** such as diatomite, pumice, and natural silicas derived from geothermal activity have been used in New Zealand. Some were used for purposes other than alleviating ASR but they can be used for this purpose. All are different and their effectiveness in reducing ASR must be assessed individually.

**Geothermal silica** is a highly reactive natural amorphous silica pozzolan unique to New Zealand and becoming accepted for use in concrete. With appropriate processing to control its particle size it can be effective in reducing ASR expansion. It does not need to be densified for safe handling, and is easier to use. Because the particles do not agglomerate it can be more effective than silica fume. It does not contain significant amounts of alkali itself, and it can contain more aluminium than silica fume, which helps bind the alcalis so that they are not available for ASR. As with natural pozzolans, effectiveness in reducing ASR must be assessed for individual products.

**Blast furnace slag** is semi-hydraulic, so acts as a cement as well as an SCM. For it to effectively minimise ASR, it must be well blended with Portland cement, and therefore must be interground or blended with Portland cement before being mixed into concrete. Blending of blast furnace slag by addition at the mixer with Portland cement is not permitted. The reactive alkali in blast furnace slag can be less than half of its total alkali content. This is assumed to be due to aluminium in the framework of the hydrated material requiring a counteraction. The composition of a particular blast furnace slag will be relatively consistent over time because it is used to monitor and control the steel composition.

Class C **fly ash** derived from Huntly coals has historically been shown to reduce ASR expansion when used in sufficient quantity. Different quantities of ashes derived from other coal sources are likely to be needed to achieve the same effect. Many aspects of the performance of fly ash in concrete are now known to be related to its calcium content. To reflect this, the Canadian Standard for fly ash, CSA A23.5, now divides fly ash into low calcium (<8% CaO), high calcium (>20%) and intermediate calcium (8 to 20% CaO) (Thomas, Shehata & Shashiprakash, 1999). This behaviour includes its ability to reduce ASR expansion, with higher levels of high-calcium fly ash (sometimes more than 50% replacement) being required to control expansion to an acceptable level. However, there is no precise relationship between calcium content and efficiency in reducing expansion (Thomas, 1995), therefore the effectiveness of individual fly ashes in reducing ASR expansion must be determined by testing.

**Metakaolin** is a largely amorphous alumino-silicate that acts as a highly reactive pozzolan. It differs from other SCMs because it is manufactured by a controlled process rather than being an industrial by-product or created by natural processes. There is no experience to date of its use to mitigate ASR in New Zealand. Based on limited data, BRE (2002) considered that it could prevent deleterious expansion in concrete when comprising 10-15% of the total binder content by mass. BRE (2002) recommends that only metakaolin with SiO₂ content greater than 45% by mass be used to minimise ASR, and that it be used as a 15% cement replacement.

Where SCM is added to concrete to protect against ASR, the level of cement replacement will depend on the SCM used. Approximate replacement levels needed to minimise ASR damage are:

- at least 8% for silica fume and geothermal silica (Cement and Concrete Association of Australia (1996) recommends that silica fume comprise at least 10% of the total binder content by mass to ensure minimising ASR damage);
- at least 15% for diatomite and metakaolin;
- more than 25% for pumice and fly ash (high calcium fly ashes require an even higher replacement level); and
- more than 50% for blast furnace slag.
CSA (2000) recommends that when two or more SCMs are used together to control ASR, the minimum replacement levels for the individual SCMs may be partially reduced provided that the sum of the parts of each SCM is at least one. For example, when silica fume and blastfurnace slag are combined, the silica fume level may be reduced to one-third of the minimum silica fume level provided that the blastfurnace slag level is at least two-thirds of the minimum slag level.

The level of replacement needed to control ASR expansion will vary with the aggregate, the cement, the presence of other SCMs, and the chemistry, mineralogy and particle size distribution of the particular SCM product(s) used, and must be determined by testing (see section 5.4.2). The level of replacement must not compromise other concrete properties such as workability.

Concrete containing an SCM must be thoroughly cured to ensure that the cement is fully hydrated, the SCM is fully reacted and the desired long-term properties achieved. This may necessitate extended curing periods for SCMs that hydrate more slowly than Portland cement, such as fly ash and blastfurnace slag. Other SCMs such as metakaolin and silica fume react very quickly, causing larger temperature increases than Portland cement, and in some circumstances it may be necessary to take precautions to prevent thermal cracking.

5.4.2 Testing and Acceptance of SCMs for Minimising ASR

Relevant New Zealand, Australian, and ASTM standards covering the use of SCMs with Portland cement are given in Table 5.
<table>
<thead>
<tr>
<th>Material</th>
<th>Specification</th>
<th>Test Method for Determining Effectiveness in Reducing ASR Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland and Blended Cements</td>
<td>NZS 3122 Specification for Portland and blended cements.</td>
<td>N/A</td>
</tr>
<tr>
<td>NZS 3123</td>
<td>Specification for Portland pozzolan cement (includes requirements for pozzolan).</td>
<td>NZS 3111 Methods of test for water and aggregate for concrete. (Based on ASTM C227; NZS 3123 acceptance criteria based on 14 and 56d results).</td>
</tr>
<tr>
<td>AS 3972</td>
<td>Portland and blended cements.</td>
<td>N/A</td>
</tr>
<tr>
<td>ASTM C595</td>
<td>Standard specification for blended hydraulic cements.</td>
<td>ASTM C227 Test method for potential alkali reactivity of cement-aggregate combination (mortar bar method). (ASTM C595 acceptance criteria based on 14d and 56d results for the blended cement, and on 91d result for the pozzolan.).</td>
</tr>
<tr>
<td>Material</td>
<td>Specification</td>
<td>Test Method for Determining Effectiveness in Reducing ASR Expansion</td>
</tr>
<tr>
<td>---------------</td>
<td>--------------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
</tbody>
</table>
5.4.2 Continued

Where SCM is supplied as part of a blended cement, the SCM shall comply with a relevant standard such as those listed in Table 5. In addition, the specifier may require a certificate giving the type and amount used, the reactive alkali content of the cement (section 7.2), and the total or available alkali content of the SCM.

If an SCM is used specifically to reduce ASR expansion, the amount to be added can be determined using the methods and acceptance criteria in the relevant ASTM test methods and specifications listed in Table 5. These require the SCM to reduce the expansion in mortar bars containing crushed Pyrex glass aggregate by a particular amount when compared with a control mortar containing no SCM. There is evidence (Lumley, 1989) that cristobalite gives more reproducible results as an aggregate than Pyrex glass and this document allows the alternative use of cristobalite.

The ASTM methods, which use artificial aggregates and elevated temperatures, may give results that do not correspond to the behaviour of site concrete. The artificial aggregates are also expensive to purchase and prepare. Therefore as an alternative tests designed to evaluate the reactivity of aggregates and the expansion of particular mix designs (see section 6.4) can be used to measure the expansions of mortars and concretes containing the aggregate and cement proposed for a particular application, using varying SCM contents to determine the amount needed to reduce expansion (Thomas & Innis, 1999). An example of such a test is Canadian standard CSA23.2-28A, which is based on a 2-year concrete expansion test but allows for a decision to be made on the basis of a 14-day mortar expansion test. The conditions used in these tests are also designed to accelerate ASR and again the results might not always represent the behaviour of in-situ concrete.

The composition and fineness of SCMs will affect their performance in concrete so uniform quality is essential. Changes in composition that increase or decrease the active components in the SCM could affect its ability to reduce expansion due to ASR, and could also affect other properties such as water demand and strength development of the concrete. For instance, Huntly fly ash has in the past varied in composition due to variability of the source of the coal being burned, although at the time of writing, tighter controls on the coal composition are producing more consistent fly ash. The composition of fly ashes, blastfurnace slags and silica fumes varies significantly between manufacturers. Because local suppliers of these may change their sources from time to time, it should never be assumed that successive purchases are from the same source, so data on composition and fineness must be provided for each purchase. The properties of natural products will vary with time as different parts of a deposit are worked, so must also be monitored. For example, over the years that Whirinaki diatomaceous pumice was used, the amount of the faster reacting diatomite decreased while the slower reacting pumicite increased, and consequently the effectiveness of this material as a pozzolan declined. In addition, the amount of clay present increased to undesirable levels. To manage the variability of the materials, attention should be paid to requirements for uniformity given in standard specifications. If these are lacking or incomplete, it may be necessary to refer to the precision statement given in the relevant test methods.

The effective use of SCMs to minimise ASR damage requires a thorough investigation of the properties of both the SCM and the resulting cement/SCM combination used with the aggregate of interest. If it is proposed to use a particular SCM to mitigate ASR with a particular aggregate for the first time, sufficient time must be allowed for evaluation, which could include testing.

5.4.3 Use of SCMs with Low-alkali Cement and for Purposes Other Than ASR Mitigation

Most overseas research and standards examining the use of SCMs to minimise ASR damage is based on the use of high alkali cement. New Zealand research has found signs of ASR in concrete containing less than 2.0% total alkali and made from low alkali cement, indicating that there is no threshold for concrete alkali content below which ASR does not occur. Instead, it is thought that as alkali content decreases, the reaction becomes more localised, produces less reaction product, and overall expansion and cracking are
reduced accordingly. Some of the concretes examined with low alkali levels were also believed to contain pozzolan. Therefore, where even superficial cracking is unacceptable and reactive aggregate must be used, both SCM and low alkali cement might be necessary to reduce ASR damage.

Structural concrete for use in marine or other aggressive environments will often contain SCM to improve properties such as resistance to chemical attack, water and chloride ion ingress and abrasion. Blastfurnace slag may be used to reduce temperature rise in large sections. This should be taken into account when assessing whether additional precautions need to be taken to minimise ASR damage in such structures. Use of SCM for other reasons might in some circumstances allow restrictions on the use of reactive aggregate to be lifted.

ASTM and other overseas criteria for determining how much blastfurnace slag or silica fume to add might not be appropriate for low alkali cement. Because the risk of ASR damage is already low, in most cases the SCM content of concrete containing low alkali cement should be determined by its effect on other properties, rather than on its effect on ASR.

5.5 Precautions Against External Moisture and Alkalis and Alkali Migration

A well-proportioned and properly consolidated concrete will give some protection against the ingress of moisture and alkalis, particularly if the design of the structure prevents water ponding thus minimising the absorption of moisture and of alkalis from seawater and salt spray. However, this will not be sufficient to prevent ASR damage where highly reactive aggregates such as rhyolite and dacite are used.

Any barrier that is used to protect a concrete surface from the absorption of external water must be a barrier to liquid water and not a vapour barrier. Complete sealing of surfaces prevents natural drying of the concrete and can increase the risk of ASR, especially where an unsealed portion of the concrete is still in contact with a moisture source. The use of vapour permeable treatments based on materials such as acrylic polymers and silanes appears to be successful in restricting the absorption of salt into concrete, but the overall effect of these types of coatings on ASR is uncertain. A major drawback of surface treatments is that they need regular maintenance, and can compromise options for future modification or remedial treatment of the structure. Relying on a moisture barrier to protect against ASR damage is not recommended as a permanent solution because with time surface treatments will degrade and new moisture sources appear.

Migration of alkalis within the concrete is generally related to wetting and/or drying of the concrete, and can be reduced by minimising these effects. Insufficient information is known about the effects of internal alkali migration in the concrete to enable appropriate precautions against it to be recommended.

5.6 Lithium Treatments

Lithium salts added as admixtures to fresh concrete have been advocated for many years as a means of reducing ASR damage but have not been used in New Zealand, largely because of the perception that alternative methods such as SCMs are more reliable, cheaper and offer additional benefits to fresh and hardened concrete properties.

A report by the FHWA (2003) reviews the use of lithium to control ASR. It discusses several mechanisms that have been proposed to explain how lithium controls ASR expansion. They include alteration of the ASR product so that it is less expansive, reducing silica dissolution, decreasing the repolymerisation of silica and silicates and reduction in repulsive forces between colloidal ASR gel particles.

The effectiveness of lithium admixtures varies between aggregates, and is not strictly related to aggregate reactivity as assessed by common test methods (FHWA, 2003).

For a given aggregate, the success of lithium admixtures depends on the particular lithium compound used and appropriate ratios of lithium to sodium equivalent being achieved. Lithium hydroxide and
lithium nitrate are preferred because they are highly soluble and therefore easily dispersed in the concrete mixture. Lithium nitrate is more efficient at reducing expansion and is safer to handle than lithium hydroxide, but can increase the risk of stress corrosion cracking in high tensile strand used in pre-tensioned concrete. Insufficient addition rates of some lithium compounds (including lithium hydroxide) can produce greater expansion than in concrete without lithium, and some combinations of lithium salts with blastfurnace slag can produce greater expansion when less than the recommended dosages of lithium are used. These effects are similar to the “pessimum proportions” observed with some reactive aggregates (see section 6.3).

BRE (2002) gives recommendations for the use of lithium hydroxide and lithium nitrate as concrete admixtures based on UK laboratory trials. It recommends dosage rates based on aggregate reactivity, lithium compound and the quantity of fly ash in the concrete. RILEM draft specification AAR-7 includes the use of lithium nitrate, with dosage rates based on aggregate reactivity. FHWA (2003) recommends a particular molar proportion of lithium to other alkali as adequate for most aggregates, but notes that some aggregates may need more and some may need less. It also recommends the combined use of lithium and SCM to reduce costs and concrete permeability. No specific guidance to the use of lithium treatments in New Zealand is given herein because of lack of experience with New Zealand materials, although recommendations for “reactive” or “highly reactive” aggregates in other documents may be applicable.
6.0 Assessment of Aggregates

Recent guidelines for assessing the alkali reactivity of aggregates have been published by the Cement and Concrete Association of Australia (1996), the UK Concrete Society (1999) and the Canadian Standards Association (CSA, 2000). RILEM technical committee TC ARP has developed a protocol and accompanying tests for assessing aggregate reactivity (Nixon 2000a, 2000b; Sims & Nixon, 2001, 2003a, 2003b), and has further tests and protocols under development. New Zealand is represented on this committee.

Aggregates of the same general type from different parts of the world can have quite different alkali reactivity. The information given in this document is based on experience with New Zealand aggregates, and is to take precedence over similar documents from other countries.

Most knowledge about the alkali reactivity of New Zealand aggregates is based on laboratory testing carried out by DSIR in the 1950’s and 1960’s, and on more recent field investigations of North Island structures. The field investigations largely confirmed the findings of the earlier laboratory tests. The ASR damage detected in these field investigations was generally too insignificant to be reported in routine maintenance inspections, so it cannot be assumed that a lack of evidence from such inspections means that ASR has not occurred. Little is known about the in-situ reactivity of South Island aggregates, other than that to date there is no record of ASR damage requiring remedial action in the South Island. This could mean that they are non-reactive, or it could mean that damage is insignificant, has not been observed, has not been recognised as ASR, or has not been investigated and/or repaired.

Reactive aggregates are variously described by different sources as “deleterious”, “potentially deleterious” and “potentially reactive”, depending on the test method used to determine reactivity. Except when discussed in the context of the specific tests, this document considers these classifications together as “reactive”. It does not distinguish different degrees of reactivity.

6.1 Recommended Practice

The initial assessment of an aggregate’s reactivity should be based on petrographic examination to identify the rock type and the likelihood that the aggregate is potentially reactive (section 6.2). If it is not potentially reactive, no further testing is needed. If it is potentially reactive, testing based on chemical methods and/or mortar bar testing or concrete testing may be carried out to confirm its reactivity as described in section 6.4, or its likely reactivity may be assessed from its performance in existing structures (section 6.5). If available, data from concrete testing and field experience shall take precedence over petrographic analysis, chemical testing, or mortar bar testing.

In the practice recommended in section 2, an aggregate is classed as non-reactive if:

- It contains less than 1% reactive or potentially reactive components as determined by petrographic examination; or
- ASTM C289 test results show it to be innocuous; or
- ASTM C1260 (or equivalent) test results at 14 days show it to be non-reactive; or
- ASTM C227 test results at 3 and 6 months show it to be non-reactive; or
- ASTM C1293 (or equivalent) test results at 1 year show it to be non-reactive; or
- Long-term field performance shows that it does not cause deleterious expansion.

An aggregate is classed as reactive if it:
• Contains more than 1% reactive or potentially reactive components and no other evidence about its reactivity is available;
• ASTM C289 test results show it to be deleterious or potentially deleterious and no other evidence from mortar or concrete tests or field evidence is available;
• ASTM C1260 (or equivalent) test results at 14 days show it to be reactive and no other evidence from concrete tests or field evidence is available;
• ASTM C227 test results at 3 and 6 months show it to be show it to be reactive and no other evidence from concrete tests or field evidence is available;
• ASTM C1293 (or equivalent) test results at 1 year show it to be reactive; or
• Long-term field performance shows that it can cause deleterious expansion.

Details of these methods are given in sections 6.2 to 6.5.

### 6.2 Petrographic Examination of Natural Aggregates

Petrographic examination of a concrete aggregate is the initial step by which its potential reactivity in concrete can be assessed. It is carried out by a geologist, preferably one with experience in the assessment of aggregate for use in concrete. The information provided by the examination should indicate whether further testing is required (Watters, 1969). The examination should include inspection of the aggregate source to determine its variability and resulting effect on the composition of the processed aggregate. It must define the groundmasses and proportion of rock types present as reactive and non-reactive as defined in sections 6.2.1 and 6.2.2, using recognised methods of examination that are considered appropriate by the geologist. ASTM C295 provides a guide for the petrographic examination of aggregates for concrete and RILEM method AAR-1 (Sims & Nixon, 2003b) is a petrographic method for identifying rock types and minerals that might be alkali reactive.

#### 6.2.1 Non-reactive Aggregates

On the basis of field experience or, in its absence, laboratory tests with high and low alkali cements, the New Zealand aggregates listed in Table 6 can be considered to be non-reactive, subject to the comments in this section.

Where New Zealand experience with these aggregates is limited and similar aggregates overseas have been shown to be reactive, testing is recommended before these aggregates are used with high alkali cements. Testing is also recommended if one of these aggregates from a particular source is likely to be contaminated with one of the aggregate types described in section 6.2.2.

"Non-reactive" refers to the reaction of aggregate with alkalis. The release of alkalis by aggregate is a different phenomenon and is described in Section 7.6.

### Table 6: New Zealand aggregates believed to be non-expansionary from field experience (with low alkali cement) or laboratory testing.

<table>
<thead>
<tr>
<th>Greywacke</th>
<th>Limestone</th>
<th>Rhyolitic pumice</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basalt &lt;50% SiO₂</td>
<td>Schist</td>
<td>Perlite</td>
</tr>
<tr>
<td>Phonolite</td>
<td>Quartz Sands</td>
<td>Vermiculite</td>
</tr>
<tr>
<td>Granite</td>
<td>Quartz-feldspar sands</td>
<td></td>
</tr>
</tbody>
</table>
Greywacke and Argillite

In New Zealand, aggregates derived from greywacke-suite rocks are used extensively, constituting approximately 60% of the aggregates used in concrete. The suite consists of an interbedded series of rock types known as greywacke and argillite that are derived from poorly sorted sands and muds respectively and are heavily indurated or moderately metamorphosed.

Argillite, being softer and a little weaker than greywacke, is often removed by natural processes in alluvial material and may be reduced in quarried products by selective extraction. The terms greywacke and argillite are used loosely to cover a range of rocks with varying mineralogical composition. These rocks may be considered non-reactive provided that the greywacke has not been hydrothermally altered to form zeolite or amorphous silica. Even where zeolitisation has occurred, it will not necessarily increase reactivity, as the zeolites formed are generally calcium based and will not exchange alkalis with the pore solution. The extensive use of greywacke aggregates in concrete without ASR occurring, even with high alkali cements (Freitag, 1998; Freitag, St John & Goguel, 2000), is unique to New Zealand, as many “greywacke” rocks found overseas are highly reactive. This is one New Zealand aggregate where accelerated tests are known to consistently give false positive tests.

Basalt

Basalt is usually defined by its mineralogical composition, and in normal practice the dividing line between basalt and andesite is a silica content of approximately 52%. However, in this document where basalt contains more than 50% of silica in its bulk composition, it is considered to be an andesite in terms of its alkali reactivity (Katayama, St John & Futagawa, 1989). Basalt with silica levels below 50% has been used extensively in the Auckland area and field evidence indicates it to be non-reactive. Basalts that contain more than 50% silica (e.g. those from Ongaroto near Taupo, and Te Henga, Waitakere) have tested as potentially reactive. Petrographic examination must be supplemented by chemical analysis to determine the silica content of the aggregate, and hence ascertain the need for further investigation of its reactivity. The nepheline basalts from central Auckland, which have been widely used in concrete, are free from expansive constituents but release alkalis and thereby produce very alkaline pore solutions in concretes (section 7.6).

Phonolite

There are no known cases of ASR involving phonolite, which is used in the Dunedin area. However, phonolite contains nepheline and releases alkali so should not be used in conjunction with reactive aggregate.

Granite

Granite is rarely used as a concrete aggregate by itself but is found mixed with other rock types in alluvial aggregates, primarily on the West Coast of the South Island. Test data on New Zealand granites are limited. No cases of ASR involving granite in New Zealand structures have been reported, although some overseas granites are reactive.

Schist

In the New Zealand context, these aggregates are mainly derived from the rocks of the Haast Schist group in the lower half of the South Island. Test data are limited. No cases of reactivity involving these aggregates are known in New Zealand.

Quartz

Quartz sands are used in concrete in some areas of New Zealand. Many of these sands consist of mixture of quartz and iron- and magnesium-rich minerals such as amphiboles, pyroxenes and olivines. Field data indicate that they are not reactive, provided that the sand does not contain volcanic fragments. In some areas of Southland, gravels rich in quartz pebbles derived from schist are used in concrete. There are few test results on these gravels and no cases of reactivity in this area have been reported.


**Quartz – feldspar**

Industry testing of coastal sands from the east coast north of Auckland, the Hauraki Gulf and the Kaipara Harbour has found them to be non-reactive. The East Coast sands have a feldspar content often exceeding 50%, the remainder being mostly quartz. The Kaipara sands are usually up to 50% feldspar.

**Limestone**

Other than dolomitic limestone, which undergoes alkali carbonate reaction, limestone is non-reactive unless it contains siliceous minerals. Overseas, siliceous limestones are often reactive. Limestones are rarely used as concrete aggregate in New Zealand so little is known about their potential reactivity. The presence of amorphous or cryptocrystalline varieties of silica should be ascertained before limestone is used, and the aggregate treated as reactive if it contains significant amounts.

**Lightweight aggregates (Rhyolitic pumice, perlite and vermiculite)**

Rhyolitic pumice, perlite and vermiculite are used in New Zealand for the production of lightweight concretes. Vermiculite, which has to be imported because no commercial deposits are available in New Zealand, is not reactive but both rhyolitic pumice and perlite are reactive to some extent. It is believed these aggregates can accommodate alkali-silica gel in their vesicular structure without expanding. Field experience indicates that both these two materials have been used in concrete without cracking but laboratory tests indicate some minor expansion may occur. Shards of non-vesicular rhyolitic pumice can cause expansion so rhyolitic pumice aggregate should be used with caution.

**6.2.2 Potentially Reactive Natural Aggregates**

The reactive constituents identified in New Zealand aggregates are the glassy to cryptocrystalline matrix of acid and intermediate volcanic rocks, and the minerals cristobalite and tridymite. Where acid and intermediate volcanic rocks are altered and cristobalite and tridymite are not observed in the rock, reactivity may only be determined by testing or from the evaluation of field data.

The aggregates listed in Table 7 are considered to be potentially reactive when used in concrete. This table is restricted to known reactive rock types present in New Zealand together with some important minerals recognised internationally as being reactive. Some potentially reactive aggregates exhibit a behaviour known as pessimum proportion, which is described in section 6.3.

“Potentially reactive” refers to the reaction of aggregate with alkalis. The release of alkalis by aggregate is a different phenomenon and is described in section 7.6.

Table 7: New Zealand aggregates or minerals known to be potentially reactive either from field experience or laboratory testing.

<table>
<thead>
<tr>
<th>Basalt &gt;50% SiO₂</th>
<th>Rhyolite</th>
<th>Cristobalite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Andesite</td>
<td>Volcanic glass</td>
<td>Tridymite</td>
</tr>
<tr>
<td>Dacite</td>
<td>Quartzite</td>
<td></td>
</tr>
</tbody>
</table>

Amorphous and cryptocrystalline silicas including opal and chalcedony

Following deterioration in prestressed piles observed in two Southland Bridges, inspection of 51 South Island bridges found that:

- Dunedin Volcanic Group rocks containing more than 50% silica may be alkali-reactive.
Assessment of Aggregates

- ASR appears to have caused visible damage to precast concrete containing alluvial aggregates from Southland and Nelson that contain quartzite, meta-quartzite, acid metavolcanics, andesite, tuff-like material, gneissic material and siltstones of a particular composition.

- Laboratory tests suggest that Southland alluvial aggregates are non-reactive in concrete cured at temperatures less than 70°C, but may react at curing or service temperatures exceeding 70°C if the concrete contains sufficient alkali.

- To minimise the risk of ASR/DEF-related expansion in concrete containing Southland and Nelson alluvial aggregates, and other rock types with a history of expansive ASR/DEF in New Zealand and overseas, concrete temperatures during curing should be monitored and should not exceed 70°C. This applies particularly to heat-cured concrete and to mass concrete.

**Basalt**

Where the silica content of the bulk composition exceeds 50% (Katayama, St John & Futagawa, 1989), basalt may be potentially reactive and should be treated as if it is an andesite in terms of reactivity.

**Andesite**

Andesite (St John, 1988) is used extensively in the Taranaki, Taupo and Bay of Plenty regions. Taranaki andesite is fresh and glassy and has no pessimum proportion. Tongariro andesite exhibits a broad pessimum proportion around 50%. Test data on other andesites are limited. Any andesite comprising more than 1% of the aggregate should be considered as potentially reactive unless testing clearly demonstrates the aggregate combination to be non-reactive. The effect of weathering and alteration on the reactivity of New Zealand andesites is not known.

**Dacite**

Dacite (St John, 1988) is currently limited to material from the Tauhara quarry. Tauhara dacite is reactive and has a sharp pessimum proportion of approximately 10-30%. There is no information on other dacites. Where dacite comprises more than 1% of an aggregate, the aggregate should be considered to be reactive unless testing clearly indicates the aggregate combination to be non-reactive.

**Microcrystalline Quartz and Strained Quartz**

The precast concrete piles of two South Island bridges containing concrete aggregates with microcrystalline quartz and strained quartz have exhibited significant deterioration from ASR/DEF. However it is considered that the extent of the deterioration has resulted from elevated curing temperatures, and the saturated immersed concrete, two conditions which need to be present for DEF.

**Quartzite**

St John, 1988 stated that quartzite is rarely used as an aggregate in New Zealand. One quartzite located near Cobb in the Nelson district has tested as reactive but is not believed to have been used in concrete. Another quartzite, from the Aorere Valley, is quarried but details on this aggregate are not known. Overseas, many quartzites are reactive and care should be taken when deposits of quartzite are investigated for use in concrete.

Quartzite and meta-quartzite (in quartz/feldspar/gneissic sand, and in mixed alluvium) were found in the aggregates used in the South Island Bridges project in which the concrete displayed ASR/DEF (Freitag et al 2010).

Testing quartzite using ASTM C289 (see section 6.4.1) may give unreliable results.
**Rhyolite**

Rhyolite (St John, 1988) is rarely used as a quarried rock, but in some areas of New Zealand occurs extensively as one of the major components present in alluvial sands and ash beds. Rhyolite is the most highly reactive and damaging rock type in New Zealand. It has a sharp pessimum proportion of approximately 10-20%. Rhyolite in the context of this document includes lithoidal rhyolites, ignimbrite and obsidian but excludes pumice. Where rhyolite comprises more than 1% in an aggregate, the aggregate shall be considered to be reactive unless testing clearly indicates the aggregate combination to be non-reactive.

**Volcanic Glass**

Obsidian (St John, 1988), present in some alluvial sands, is reactive and should be treated as for rhyolite. Volcanic glass in the groundmass of rhyolite, dacite and andesite is the principal reactive component of New Zealand rocks. The effect of alteration on the reactivity of the glassy groundmass is not known. Hydrated volcanic glass (pitchstone) is similar to obsidian. Unless cristobalite and tridymite are present, volcanic glass does not exhibit a pessimum proportion.

**Cristobalite and Tridymite**

The silica minerals cristobalite and tridymite (St John, 1988; Katayama, St John & Futagawa, 1989) are significant reactive minerals in rhyolite and dacite and to a lesser extent in andesite. Rocks containing these two minerals will exhibit pessimum proportion. A careful search should be made for the cristobalite and tridymite in the rock and their presence should signal the need for further testing.

**Alluvial gravels and sands**

Alluvial gravels and alluvial sands (St John, 1988) may contain both innocuous and reactive materials. As a general rule alluvial sands are more reactive than alluvial gravels. Alluvial sands from the bed of the Waikato River (Freitag, 1990) or from the Hinuera Formation terraces are known to be highly reactive and should not be used without taking adequate precautions to minimise the risk of ASR.

All North Island alluvial deposits used for concrete aggregate must be checked for the possibility of contamination with volcanic materials. This includes the possibility of ash beds well removed from centres of volcanic activity contributing to otherwise innocuous aggregate sources.

Where acid/intermediate volcanic materials comprise more than 1% of the aggregate combination, the aggregate should be considered to be reactive unless testing or field data indicates the aggregate combination to be non-reactive. For example, Rangitikei River sand contaminated with small amounts of acid and intermediate volcanic material has reacted both in laboratory tests and in site concrete.

Approximately 75% of concrete aggregates used in New Zealand are processed from alluvial sources. Alluvial sources are inherently variable, and where they contain potentially reactive rock types this variability must be allowed for in concrete mix design to minimise ASR.

**Amorphous and cryptocrystalline silicas including opal and chalcedony**

While these minerals occur in some New Zealand rocks (Watters, 1969) none of these rocks are currently used for concrete aggregates. Opal is one of the most reactive of minerals and even small amounts automatically class a rock as reactive. Flint and chert are types of chalcedony and are reactive in certain forms. Flint is not used in New Zealand concrete but some coloured chalcedony (jasper) is. Limited test data indicates this jasper to be non-reactive (St John, 1988). Overseas, submicroscopic particles of amorphous to cryptocrystalline silica are believed to be present in reactive rocks such as siliceous sandstones and limestones, some greywackes and possibly other rocks such as quartzites, granites and gneisses. No problems with these minerals in New Zealand aggregates have yet been identified. Testing these types of aggregates using ASTM C289 may give unreliable results.
6.2.3 Artificial Aggregates

Little information is currently available on the reactivity of artificial aggregates. Laboratory testing of some New Zealand expanded argillites indicated these materials to be slowly expansive (St John & Smith, 1976). Field experience in the USA and Europe indicates that some expanded shales are also expansive in concrete. Experience has shown expanded vermiculite to be non-reactive. Specialist advice should be obtained for information on the reactivity of artificial aggregates.

6.2.4 Recycled Concrete Aggregate

The potential alkali reactivity of recycled concrete aggregate (RCA)* and leftover concrete aggregate (LCAgg)* shall be determined on the basis of the potential reactivity of the parent aggregate*. If the source of the parent aggregate is unknown, the recycled concrete aggregate should be classified as potentially reactive.

Recycled aggregate (RA)* which is predominantly waste concrete, masonry and asphalt shall be classified as potentially reactive in all situations as it contains a wide range of constituents so its reactivity is difficult to determine.

6.2.5 Recycled Glass Aggregate

Crushed glass as aggregate for concrete is classed as potentially alkali reactive. Its use as coarse aggregate is not viable based on the aggregate shape and the surface area exposed to cement alkalis. However crushed glass fine aggregate may be used to replace part of the natural fine aggregate fraction of concrete. Typical replacement rates of crushed glass as a proportion of fine aggregate are between 5% and 20%.

The limiting fraction of fine aggregate is controlled by minimizing the risk of ASR and suppressing the alkali reactivity by using SCM’s. The general principles for minimizing the risk of ASR on a structure are outlined in section 2.1. Guidance on the use of SCM’s for suppressing ASR is given in sections 5.4 and 7.3.

* As defined in CCANZ TR 14

6.3 Aggregate Combinations and Pessimium Proportion

The adjective pessimium indicates a worst possible condition. A pessimium proportion of a reactive aggregate is the proportion that results in the greatest expansion due to ASR. In practice, the pessimium proportion is variable and each potentially reactive aggregate type requires testing to ascertain the position and shape of the pessimium proportion curve. Most reactive aggregates exhibit pessimium proportion, and materials such as Egmont andesite that have no pessimium proportion are less common.

Some potentially reactive aggregates will not cause significant ASR expansion when present in a quantity well outside their pessimium proportion even when used with high alkali cement. Dilution or augmentation of such aggregates with other aggregates to avoid their pessimium proportion must not be used to control ASR expansion without thorough testing to identify non-expansive aggregate combinations.

The silica minerals opal, chaledony, cristobalite and tridymite exhibit marked pessimium proportions. Only a few percent of these minerals needs to be present in the aggregate to achieve maximum expansion with alkali. Our New Zealand rhyolites and dacites have pessimium proportions varying between 15% and 40% because of the presence of some of these minerals. In contrast, volcanic glass shows little pessimium proportion and our andesites, which are often highly glassy in texture, exhibit a limited range of pessimium proportion. Egmont andesite usually has no pessimium proportion while andesites from other regions appear to have broad pessimums. Typical examples of expansion curves for some of these materials are given below.
Assessment of Aggregates

![Graph showing typical pessimum proportion curves for three New Zealand rock types.](image)

Figure 1: Typical pessimum proportion curves for three New Zealand rock types (Expansion at 12 months at 1.5% Na₂O eq.).

The most reactive materials in New Zealand are sands consisting of a mixture of volcanics and less- or non-reactive rock types. Rhyolite and dacite have pronounced pessimum proportions and are most reactive when mixed at their pessimum with innocuous or less reactive materials. For example, sand from the Waikato River contains a pessimum proportion of rhyolite combined with less reactive volcanics and other materials.

The pessimum proportion of an aggregate can only be defined by mortar and concrete tests, preferably confirmed by field data. This document does not require pessimum proportion expansion curves to be determined, as this is a lengthy and costly process. It assumes that if ASTM C289 or other testing identifies an aggregate as being either potentially deleterious or deleterious, then preventive measures to limit ASR must be applied.

6.4 Tests for Alkali Reactivity of Aggregate

If petrographic examination finds that an aggregate is potentially reactive, then either further testing is required to prove it is not, or it must be assumed that the aggregate is reactive and appropriate precautions taken.

Under New Zealand ambient conditions concrete can take five to twenty years to expand significantly. Tests for determining the reactivity of aggregates generally involve subjecting the aggregate to conditions designed to accelerate the reaction so that results can be obtained in a more practical time frame. These are prone to giving both false positive and false negative results. Internationally, researchers have developed many tests to overcome discrepancies between results from standard tests and the observed behaviour of their local aggregates. These include tests on the aggregates themselves, tests on mortar bars, and tests on concretes. The following sections describe tests that are considered to have potential for assessing New Zealand aggregates. Tests at 38°C have been shown to produce reliable results (Grattan-Bellew, 1989); tests at 60°C can produce results more quickly.

Concrete tests are more reliable than chemical and mortar bar tests but expansion is measured for prolonged periods. Often it will be more practical to assume that a potentially reactive aggregate is in fact reactive, and to take the appropriate precautions.

6.4.1 Chemical Test

ASTM C289, known as the “quick chemical test” or the “Miehren test” (Miehren and Benton, 1958), has been found to be a satisfactory initial method for determining the potential reactivity of New Zealand aggregates derived from volcanic rocks. The test categorises aggregates as “innocuous”, “potentially deleterious” or “deleterious”. ASTM C289 is sensitive to sample preparation and requires strict adherence to the test method and should be performed by an experienced laboratory. A range of New
Zealand materials has been tested by this method, and the results are summarised in Appendix D. Site investigations have shown that results generally do reflect the reactivity of New Zealand aggregates in site concrete, and it is recommended therefore that this method continue to be used to assess new aggregate sources.

ASTM C289 may not be reliable in predicting reactivity of slow reacting aggregates (e.g. quartzite).

Some aggregates react at a pessimum proportion that can only be determined from mortar or concrete tests or by evaluation of field data.

At least one week should be allowed for ASTM C289 tests.

6.4.2 Mortar Bar Tests

Mortar bar tests allow a coarse aggregate to be tested by crushing it, separating the crushed material into size fractions and recombining them to a specified grading. Sand may be tested either “as supplied” or to the specified grading requirement. Testing to the specified grading can alter the proportions of reactive minerals in the material, and thereby give misleading results. Testing “as supplied” may mean testing a very stiff or fluid mortar, or adjusting the mortar composition to achieve a specified workability. Acceptance criteria generally apply only to the specified aggregate grading, so results from material tested “as supplied” need to be tested alongside controls of known non-reactive materials and reactive materials so that their results can be interpreted by comparison with the controls. If proposed aggregates are tested individually using the sample gradings specified by the test method and are not tested in combination, the aggregate combination should be assumed to have the reactivity indicated by the most reactive aggregate component.

ASTM C227

NZS 3111: 1986 includes a test method based on ASTM C227, but conditions for storing the test specimens have been modified since NZS 3111: 1986 was published and are under continued review, so the most recent version of ASTM C227 should be used in preference to the NZS 3111: 1986 method. The test requires measurement of the length change of mortar bar specimens stored over water at 38°C for six months, but often measurements have to be carried out for two years to obtain reliable results. ASTM C227 has proved to be a suitable test method for determining the reactivity of New Zealand aggregates derived from volcanic rocks but requires experience to obtain meaningful results. Both the amount of alkali in the cement and the proportion of reactive aggregate used may be varied in this test so that a large number of test combinations are possible. The results are sensitive to the storage conditions (Rogers & Hooton, 1989) of the mortar bars and for results to be comparable with existing test data, cement content, grading of aggregates and workability must not be varied from that specified. Many New Zealand materials used for concrete aggregates have been tested using ASTM C227 (St John, 1988).

ASTM C1260

ASTM C1260, better known as the “rapid mortar bar test” or the “NBRI test”, is based on a procedure developed by the National Building Research Institute [NBRI] in South Africa to overcome ASTM C227’s shortcomings: duration of test, test conditions affecting the results, and slowly-reacting aggregates not being detected. It involves storing mortar bars in highly alkaline solutions at 80°C for a minimum of two weeks (two months is recommended) and measuring their expansion. Japanese and New Zealand research found this method to be suitable for New Zealand volcanic rocks (Yoshioka, Kasami, Ohno & Shinozaki, 1989; Freitag, St John & Goguel, 2000). The aggressive nature of the test causes ASR expansion of aggregates that are known to be non-reactive in concrete. To avoid taking unnecessary precautions to prevent ASR damage with such aggregates, the method is recommended only as a screening test, with aggregates that test as reactive being subjected to further testing before use. The method is also used to measure the ability of SCMs to reduce ASR expansion.

Procedures based on this approach have been adopted by other organisations such as RILEM (AAR-2), CSA (A23.2-25A), and the Roads and Traffic Authority [RTA] in New South Wales (Shayan & Morris, 2001).
Guidelines for acceptance of aggregates on the basis of this approach have been published by several authorities. The appendix to ASTM C1260-01 suggests that expansions less than 0.10% at 14 days usually indicate innocuous behaviour, expansions greater than 0.20% at 14 days indicate potentially deleterious expansion and expansions between 0.10% and 0.20% require extra information to be obtained. CSA23.2-27A defines an aggregate as nonreactive if its 14 day expansion is less than 0.150% and highly reactive if its 14 day expansion is greater than 0.150%. RILEM (AAR-0) suggests that precautions be taken to minimise the risk of ASR damage for concrete containing aggregate combinations that produce AAR-2 results of 0.10% or higher. The Cement and Concrete Association of Australia (1996) uses the CSIRO/RTA test to define three levels of reactivity: reactive (expansions of 0.1% or greater at 10 days), slowly reactive (expansions of 0.1% or greater at 22 days) and non-reactive (expansions less than 0.1% after 22 days), while Shayan and Morris (2001) recommend a limit of either 0.1% in 21 days or 0.08% in 14 days to distinguish reactive and slowly reactive aggregates.

### 6.4.3 Concrete Tests

Like mortar bar tests, concrete tests can be used to evaluate aggregates “as supplied” or in a specified grading. The features of each approach are the same as for mortar bar tests. Again, if proposed aggregates are tested individually using the sample gradings specified by the test method and are not tested in combination, the aggregate combination should be assumed to have the reactivity indicated by the most reactive aggregate component.

**ASTM C1293**

ASTM C1293 describes a procedure for testing the reactivity of aggregate in a standard concrete made with a specified high alkali cement and additional alkali. The test involves measuring the expansions of concrete prisms stored over water at 38°C for at least 12 months. The results are highly sensitive to storage conditions.

Similar procedures have been adopted by RILEM (AAR-3), CSA (A23.2-14A) and AFNOR (P18-587).

CSA 23.2-27A uses this test to define three levels of reactivity: non-reactive (aggregates as having 1 year expansion less than 0.04%), moderately reactive (1 year expansion 0.04-0.12%) and highly reactive (1 year expansion greater than 0.12%). RILEM AAR-0 recommends that precautions be taken to minimise the risk of ASR damage in concrete containing aggregate combinations that produce AAR-3 expansions of 0.05% or higher after one year, in the absence of local experience about their reactivity.

**RILEM AAR-4**

RILEM AAR-4 is similar to ASTM C1293 but is carried out at 60°C and expansions are measured for at least 20 weeks. The method allows for testing of aggregates in a mix design proposed for use in a specific application although concrete alkali contents are still elevated.

Touma, Fowler, Carrasquillo, Folliard & Nelson (2001) found that ASTM C1293 could be effectively accelerated to generate results within three months by increasing the testing temperature to 60°C.

No acceptance criteria have been published for these tests at the time of writing so comparative testing is necessary, although Sims & Nixon (2003a) report that expansions of 0.02% or less at three months were found to indicate non-expansive aggregate combinations in initial trials of the methods.

Queensland Roads has developed a procedure for assessing aggregates in concrete that is to be steam cured (Carse & Dux, 1990). Testing takes at least four months. Cement and Concrete Association of Australia (1996) presents three classifications of reactivity based on the 4-month test results: innocuous (less than 200 microstrain), slowly reactive (200 to 500 microstrain) and substantially reactive (greater than 500 microstrain).
6.5 Field Performance of Aggregates

Long-term field performance may be used to determine whether an aggregate is likely to cause deleterious expansion, either for general application or for a specific construction.

When field performance is to be assessed:

(a) the cement content of the field concrete(s) examined and the alkali content of the cement(s) they were made from contained must be identified. The field concrete’s alkali content must be the same or higher than in any proposed new concrete for which ASR precautions are based on field assessment;

(b) the concrete examined must be at least 10 years old;

(c) the exposure conditions of the field concrete must be at least as severe as those in the proposed structure;

(d) a petrographic analysis should be conducted to demonstrate that the aggregate in the structure is identical to that under investigation in the absence of conclusive documentation;

(e) the possibility of supplementary cementing materials having been used must be considered; and

(f) the water/cementing materials ratio of the field concrete must be considered because this may also affect the effects of ASR.

Field performance must be investigated by personnel experienced in the assessment of AAR in structures.
7.0 Estimation of Alkali Contributed by Concrete Constituents

7.1 Definition of Terms

Although it is actually the concentration of hydroxyl ions, reflected by the highly alkaline pH of the pore solution, that provides the “driving force” of ASR, the alkali content of the concrete is expressed as the concentrations of the oxides of the alkali metal ions sodium and potassium, which are easily measured by instrumental analytical techniques. Calcium hydroxide in the cement hydrate dissolves to balance these ions in solution, so their concentration reflects the amount of hydroxide ions that will be produced. “Alkali content” thus refers to sodium and potassium ions rather than to the pH or hydroxide ion content.

In this document, “reactive alkali” is used to denote alkali that is considered able to move into the pore solution and cause ASR during the service life of the concrete. The way in which alkalis are present in the various components, and the manner in which they move into the pore solution, will dictate their reactivity and therefore the method by which they should be measured and managed. Appendix C gives methods of calculating alkali contributions from concrete constituents.

Alkalis in the cementitious binder are referred to as “total alkali”, “acid soluble alkali”, “available alkali” and “water soluble alkali”. “Total alkali” refers to the total alkali present in the material while “acid soluble alkali” is that extracted by strong acids such as nitric or hydrochloric. For Portland cement these are the same, but this does not necessarily apply to other materials. “Available alkali” is alkali that can be extracted with saturated calcium hydroxide solution, while “water-soluble alkali” is that extracted with water alone. The times of extraction may vary according to the test method. In practice, alkali contents are based on solubility in saturated calcium hydroxide solution or on a total alkali determination.

Alkalis sourced from aggregate are not included in the above terminology (see section 7.6).

7.2 Alkali in Portland and Portland-limestone Cements

The reactive alkali content of Portland cement is the “total alkali” content of the cement as measured by acid dissolution or equivalent instrumental analysis. Where limestone is interground with the Portland cement, no distinction is made and any acid soluble alkali from the limestone is included. Methods of alkali analysis are given in ASTM C114 and BS-EN196-21 (National Annex NA).

The alkali content of cement is expressed as its sodium oxide equivalent (Na₂Oeq), which combines its sodium and potassium oxide contents according to the formula:

\[ \text{Na}_2\text{O equivalent} = \% \text{Na}_2\text{O} + (0.658 \times \% \text{K}_2\text{O}) \]

Alkali contents of Type GP cement now manufactured in New Zealand range from 0.4% to 0.6% Na₂O equivalent. NZS 3122 allows a maximum alkali content of 0.60% Na₂O equivalent to be specified for cement to be used with potentially alkali reactive aggregate, although as noted in section 5.3 this alone may not be enough to minimise the risk of ASR damage.

Although the results for sodium oxide equivalent are reported to the nearest 0.01%, the reproducibility between laboratories is such that laboratories’ results for the same sample can differ by ±0.05%. In addition, there can be significant differences between the alkali contents of different batches of the same cement. Both these variations need to be taken into account. Thus:

- calculations of concrete alkali content should be based on the maximum alkali equivalent quoted on the cement manufacturer’s certificate, rather than on the average value; and
• a cement may be considered to comply with a requirement for maximum alkali content if over time the averaged values of the alkali content do not exceed the specified maximum, and that results of individual batches do not exceed the maximum by more than 0.05%.

7.3 Alkali in SCMs

The reactive alkali content of SCMs varies with the different types and products. It used to be considered a significant factor in determining appropriate materials and mix proportions. Current understanding is that the effect of SCMs on ASR expansion is determined by the amount used and properties such as their particle size distribution, mineralogy and chemistry, rather than their reactive alkali content. Thus a high alkali SCM might still reduce expansion to an acceptable level.

Examples of how different guidelines manage SCM alkali contents are given below:

(a) The UK Concrete Society (1999) and BRE (1999) do not require the alkali contents of blastfurnace slag or fly ash to be included in the calculation of concrete alkali content, provided that more than a minimum amount of blastfurnace slag or fly ash is used, and depending on the reactivity of the aggregate. When less than the minimum amount is used, a proportion of the alkali content is taken into account. For blastfurnace slag, none of the alkali is included if the blastfurnace slag comprises more than 41% of the binder. 50% is included if the blastfurnace slag comprises 25-41% of the binder, and all the blastfurnace slag alkali is included if the blastfurnace slag comprises less than 25% of the binder. For fly ash, none of the alkali is included if the fly ash comprises more than 25% of the binder. 20% is included if the fly ash comprises 20-25% of the binder, and all the fly ash alkali is included if the fly ash comprises less than 20% of the binder. These two publications require that the acid soluble (or total) alkali content of blastfurnace slag not exceed 1.0% Na₂Oeq and the acid soluble alkali content of fly ash not exceed 5.0% Na₂Oeq. These precautions were taken because existing knowledge about performance of these materials was based on low alkali products, and knowledge about the performance of higher alkali products was limited, rather than because the high alkali products perform poorly.

BRE (2002) places no limit on the alkali content of silica fume, but does require that its total alkali content be included in the concrete alkali content. It does not limit the alkali content of metakaolinite, and does not require its alkali content to be included in the concrete alkali content.

(b) In Canadian Standard CSA A23.2-00, concrete alkali limits only take into account alkali contributions from the Portland cement fraction, but the total alkali content of SCM is taken into account when determining cement replacement levels. Cement replacement levels are given up to a certain SCM total alkali content. SCM with higher total alkali content can be used if testing shows it to be effective in reducing ASR expansion.

(c) Cement and Concrete Association of Australia (1996) considers that 50% of the total alkali in blastfurnace slag, 15-20% of that in fly ash and most of that in silica fume contribute to the alkali content of pore solution and must be taken into account. It recommends that the total alkali content of fly ash be limited to a maximum of 3% and that the total alkali content of blastfurnace slag and silica fume be limited to a maximum of 1%, with approximate cement replacement levels of 10% for silica fume, 25% for fly ash and 65% for blastfurnace slag, unless proposed mixes are to be assessed by accelerated concrete testing.

(d) Clauses in ASTM specifications for blastfurnace slag, fly ash/natural pozzolan and silica fume that cover reactivity with cement alkalis or the mitigation of ASR expansion, are based on mortar bar expansion tests rather than alkali contents.

The change in emphasis from reactive alkali content to total alkali content is probably due to an almost universal use of instrumental rather than wet chemical analysis, and appreciation that the reactive alkali content of an SCM is not the critical factor in determining its effectiveness in reducing ASR expansion. In addition, measurement of available or reactive alkali is difficult in blended cements containing blastfurnace slag or fly ash - the alkali contents of the Portland cement and the SCM must be measured.
separately before intergrinding, and the reactive alkali content of the blastfurnace slag or fly ash is then estimated from the relative proportions of each constituent. Therefore it is not possible to determine the reactive alkali content of blended cement as received, and it must be obtained from the manufacturer if required.

Total alkali contents of silica fume can be as high as 5.5% (BRE, 2002). The average alkali equivalent of metakaolin samples analysed by BRE (2000) was 0.65%. Examples of alkali contents of other SCMs are given in Table 8. Table 8 is neither a complete list of all SCMs, nor a list of “typical” values, but is intended to demonstrate the range of alkali contents of different types of SCM and individual products.

Table 8: Examples of alkali contents of SCMs

<table>
<thead>
<tr>
<th>Type</th>
<th>Product</th>
<th>Total alkali content* (% Na₂Oeq)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silica fume</td>
<td>Typical Range</td>
<td>0.44 - 3.9%</td>
</tr>
<tr>
<td>Geothermal silica</td>
<td>Product A</td>
<td>0.46</td>
</tr>
<tr>
<td>Fly ash</td>
<td>Product C, sample 1</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>Product C, sample 2</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td>Product D</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>Product E</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>Product F</td>
<td>3.2</td>
</tr>
<tr>
<td>Metakaolin</td>
<td>Product G</td>
<td>0.68</td>
</tr>
<tr>
<td>Blastfurnace slag</td>
<td>Product H</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>Product I</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td>Product J</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>Product K</td>
<td>0.33</td>
</tr>
</tbody>
</table>

* These alkali contents are from particular samples and are given to demonstrate the range of alkali contents of different types of SCM and individual products. They must not be used to calculate alkali contents of proposed concretes.

7.4 Alkali in Mix Water

When the 1991 edition of this document was written, the alkalis in municipal water supplies in the upper half of the North Island varied from 7-320 mg/l for sodium and 1-11 mg/l for potassium. In approximately 70% of the waters the sodium was less than 20 mg/l. This data indicates that in the worst case, mixing water containing 320 mg/l of sodium and 11 mg/l of potassium used in concrete containing 380 kg/m³ of cement at a water/cement ratio of 0.5 will contribute 0.084 kg/m³ of reactive alkali (expressed as Na₂Oeq) to the concrete. Thus the contribution of reactive alkali to concrete from potable waters in New Zealand is not significant and can usually be ignored.

Most local authorities will have results of analyses of their water supplies carried out for health purposes. These results are available in the Water Information for New Zealand (WINZ) database, which is compiled for the Ministry of Health by the Water Group of the Institute of Environmental Science and Research Ltd (ESR). However sodium, potassium or chlorine analyses are not required by the Ministry of Health programmes for which WINZ was established, and ESR stopped collecting them in 1995. Data collected prior to 1995 are contained in the JASPER database, and can be accessed by ESR on request. Some local authorities may collect data other than those required for health purposes and store them on their copy of WINZ.
Because the composition of drinking water does not vary greatly with time, old water analyses are generally acceptable unless there has been a major change in the supply (such as its source) since the most recent analyses were performed. Specific chemical analysis of town supplies used for mixing water should not be necessary.

The alkali content of water is normally reported as mg/l of sodium and potassium ions. This differs from cement analysis, where alkalis are reported as sodium and potassium oxides and for convenience are represented as sodium oxide equivalent. The method of calculation given in Appendix C allows for these different methods of reporting alkalis. The accuracy of the analyses is about ±1 mg/l for 10 mg/l of sodium increasing to ± 10 mg/l for 200-300 mg/l of sodium.

7.4.1 Recycled Water

Where the mixing water is drawn from a source other than a town supply its alkali content should be determined. This is particularly important when water used to clean concrete mixers and trucks is recycled and used to mix concrete. Water that has been in contact with cement usually contains significant quantities of alkalis that arise from the initial hydration of cement; so recycled wash water will contain higher alkali contents than potable water. Therefore its alkali content should be considered and added to the alkali contributed by other components, particularly if the total concrete alkali content is close to the maximum specified limit. For these purposes alkali content is determined by chemical analysis of sodium and potassium rather than pH or total alkalinity. ASTM C94 places a limit of 600 ppm (0.06%) on the alkali content of wash water expressed as Na₂Oeq.

Analysis by CCANZ of nine samples of recycled wash water from one ready-mix concrete plant in 2003 gave a maximum sodium equivalent of 93mg/l (ppm). This is well within the ASTM C94 limit of 600 ppm. Individual plants should monitor their recycled wash water to determine their operating parameters.

7.5 Alkali from Chemical Admixtures and Pigments

Although the principal source of alkali in concrete is from the cementitious materials, chemical admixtures may also make a significant contribution. For example, analyses of one supplier’s range of commercial admixtures in 2002 gave the results in Table 9.

Table 9: Alkali contents of concrete admixtures

<table>
<thead>
<tr>
<th>Admixture type¹</th>
<th>Na₂O (%)²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water reducing</td>
<td>&lt; 0.1 – 5.0</td>
</tr>
<tr>
<td>Air entraining</td>
<td>&lt; 0.2 – 0.5</td>
</tr>
<tr>
<td>Superplasticising</td>
<td>1.8 – 5.0</td>
</tr>
<tr>
<td>High early strength superplasticising</td>
<td>~ 11.0</td>
</tr>
<tr>
<td>Set accelerating</td>
<td>&lt; 0.1 – 1.8</td>
</tr>
<tr>
<td>Set retarding</td>
<td>~ 5.3</td>
</tr>
<tr>
<td>Pump aid</td>
<td>&lt; 0.1</td>
</tr>
<tr>
<td>Shrinkage reducing</td>
<td>&lt; 0.1</td>
</tr>
</tbody>
</table>

¹ These data represent products from one manufacturer only. They must not be used to calculate alkali contents of proposed concretes.

² These values are analyses for sodium only. They do not include K₂O so are not alkali equivalents. Sodium is often the prevalent alkali.
From Table 9, contributions from this superplasticiser or water reducer added at the rate of 1% (by cement weight) to concrete containing 400 kg/m³ of cement could result in up to 0.2 kg/m³ of sodium oxide being added to the concrete. Most admixtures are soluble in the pore solutions so all the alkali is available to react. Alkali contents of admixtures proposed for use in concrete containing reactive aggregate should be obtained from the admixture supplier and included in the concrete alkali content.

The alkali contribution from pigments also needs to be considered. Again, this information should be obtained from the supplier and included in the concrete alkali content.

7.6 Alkali Associated with Concrete Aggregates

Alkalis are present in the minerals in aggregates. The alkaline pore solutions of concrete or mortar may dissolve these minerals along accessible intergranular surfaces, releasing alkalis to the pore solution. Ion exchange phenomena do not play a significant role because aggregates containing significant amounts of zeolites and clay minerals are not normally used in concrete.

The alkali-bearing minerals in aggregates are aluminosilicates, feldspathoids (nepheline, leucite), glass, feldspars, and micas. The rate of dissolution strongly decreases in the above order (Goguel, 1995; Goguel, 1996; Goguel & Milestone, 1997). Not all alkalis are released into pore solutions, so chemical analysis of an aggregate (e.g. by X-ray fluorescence [XRF]) will not identify whether alkali release will be a problem. New Zealand concretes examined to date show no in-situ evidence for major alkali release from aggregates other than from feldspathoid minerals. Substantial alkali release from feldspar-bearing aggregates has been reported from Canada (Bérubé, Duchesne & Rivest, 1996), with lower levels measured in the USA (Constantiner, 1994) and the UK (Savage, et al., 1992). These variations are likely to be due to the differences in crystallinity and surface characteristics that develop in different rock forming processes, but may also reflect differences in laboratory treatment of the aggregate prior to analysis (Goguel, 1995).

Research at IRL using experimental mortars concluded that alkali released by acidic rocks (rhyolites, granites, dacites) imbedded in concrete does not add to the alkalinity of the pore solutions, but is taken up by solid silicate phases. However, these alkalis can be included in the analyses of total concrete alkali content, depending on the extraction method used.

Feldspathoid minerals readily dissolve in an alkalinie environment (also in an acid one), and release alkalis to the pore solutions of concrete. However, they are absent from most aggregates, which is why major alkali release from aggregates in concrete was not recognised until recently (Goguel, 1996). Aggregates known to contain significant amounts of the feldspathoid nepheline and minor amounts of leucite are the Central and South Auckland basalts which may be classified as nepheline basanite, and phonolite that occurs on the Otago Peninsula. About half of the alkali in Central and many of the South Auckland nepheline basanites is associated with feldspathoids, and can amount to about 20 kg Na₂Oeq per ton of basalt. Work with experimental mortars showed that only about half that amount is released; of which between 50 and 80% becomes fixed in the paste hydrates. This typically raises the pH of the pore solution up to 14 (Goguel & Milestone, 2000).

Under ambient conditions in site concrete, high humidity and several years of alkali release are required to attain the high alkali levels in the pore solution needed to generate ASR and significant subsequent expansion.

Alkali release from aggregates depends on the combination of all of the following conditions:

- Presence of alkali releasing mineral phases, i.e. feldspathoids and to a lesser extent alkali rich glass and poorly crystallised feldspars;
- Readily-accessible surfaces of the alkali releasing phases in the sand fraction; and
- High humidity: Residual pore solutions in concrete exposed to a dry environment are unlikely to leach alkali from aggregate. Alkali release will take place in structures that retain soil and/or
water and those that are subject to condensation, such as foundations, retaining walls, bridges and pavements.

Although aggregates with the potential to release significant amounts of alkali can be identified, it is more difficult to predict the actual amount that will be released in a particular concrete element on a structure. Therefore it is recommended that such aggregates not be used in the same concrete as alkali reactive aggregates unless risks associated with ASR damage are sufficiently low that the level of precaution required is “Nil” (see section 2.2).

Aggregates may be contaminated with sea salts, especially if they are derived from a marine source, and the sodium chloride will contribute reactive alkali to the concrete. The reactive alkali present in the aggregates is estimated from the water soluble chloride content of the aggregates determined according to BS 812: Part 117. Since sodium chloride is not the only salt present, a correction factor is required to convert chloride to sodium oxide equivalent (see Appendix C). The method used for determining chloride in aggregates can detect chloride levels as small as 0.005%.

As aggregates form a large portion of the volume of concrete, only small amounts of chloride contamination of the aggregate can contribute significant reactive alkali. For example, 1000kg of sand with a chloride content of 0.1% will contribute 0.76 kg/m³ of reactive alkali to the concrete based on the mean composition of seawater. Salt contamination of aggregates must be monitored to minimise the risk of reinforcement corrosion, as well as ASR.

### 7.6.1 Recycled Concrete Aggregate

The adhered mortar in recycled concrete aggregate will contribute to the total alkali in concrete from all sources, as outlined in this section.

The alkali contribution from recycled aggregate shall be assumed to be:

- 0.2 kg Na₂O eq. per 100 kg of recycled concrete aggregate, or

- Where the adhered mortar content and alkali content of the parent concrete (the concrete from which the recycled aggregate is derived) is known, the alkali content can be calculated from the alkali content of the parent concrete.

Section 2.2.2 cautions against using potentially reactive aggregate and alkali-releasing aggregate together in the same concrete. This is because the amount of releasable alkali from natural aggregate cannot be calculated or measured.

However potentially reactive aggregate may be used in conjunction with recycled concrete aggregate as the alkali contributed by recycled concrete aggregate can be measured or calculated. The precautions specified in Section 2 for normal or special concrete, must be adhered to.
## Appendix A: Glossary

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Accuracy</strong></td>
<td>A statistical term that estimates the error in the absolute value of an analytical result: “how close is the result to the truth?”</td>
</tr>
<tr>
<td><strong>Acid rocks</strong></td>
<td>Volcanic rock types which contain more than 66% SiO$_2$. Includes rhyolite, pumice, obsidian and ignimbrite.</td>
</tr>
<tr>
<td><strong>Acid soluble alkali</strong></td>
<td>The alkali extracted from Portland and blended cements and from SCMs by dissolution in strong acid.</td>
</tr>
<tr>
<td><strong>Additive</strong></td>
<td>An obsolete term now replaced by admixture.</td>
</tr>
<tr>
<td><strong>Admixture</strong></td>
<td>A material either added to the cement or during the mixing of concrete to enhance or change the properties of the concrete. (See chemical admixture, mineral admixture).</td>
</tr>
<tr>
<td><strong>Alkali aggregate reaction (AAR)</strong></td>
<td>Generic term for the chemical reactions that occur between minerals in concrete aggregate and alkalis in concrete pore solutions. Includes alkali silica reactions and alkali carbonate reactions.</td>
</tr>
<tr>
<td><strong>Alkali carbonate reaction (ACR)</strong></td>
<td>Reaction between alkalis in concrete pore solutions and silica minerals in carbonate rocks such as limestone and dolomite.</td>
</tr>
<tr>
<td><strong>Alkali content</strong></td>
<td>The alkali equivalent of cement, expressed as a weight percentage, or the alkali content of concrete calculated from the cement alkali equivalent and the cement content of the concrete and expressed as kg/m$^3$ of concrete.</td>
</tr>
</tbody>
</table>
| **Alkali equivalent**                   | Total combined sodium and potassium oxides in cement expressed as weight percent, based on the formula \[ \text{Na}_2\text{O}_{eq} = \% \text{Na}_2\text{O} + (0.658 \times \%\text{K}_2\text{O}) \]

where 0.658 is the molecular weight ratio of Na$_2$O to K$_2$O (see alkalis). |
| **Alkali silica gel**                   | The gel that forms in concrete upon reaction between the alkalis in the pore solution and reactive SiO$_2$ in the aggregates.                                                                              |
| **Alkali silica reaction (ASR)**        | Reaction between alkalis in concrete pore solution and silica-rich minerals in concrete aggregates.                                                                                                        |
| **Alkalis**                             | Sodium and/or potassium ions (usually expressed as their oxides Na$_2$O and K$_2$O in cement analyses).                                                                                               |
| **Available alkali**                    | Alkali extracted from a material by calcium hydroxide solution as described by ASTM C311 and AS 3583.12. Not necessarily the same as reactive alkali.                                                        |
| **Basic rocks**                         | Volcanic rock types (mainly basalt) which contain less than approximately 52% SiO$_2$.                                                                                                               |
| **Batch or shipment**                   | A quantity of material to be agreed between specifier and supplier.                                                                                                                                     |
### Appendix A: Glossary

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blastfurnace slag</td>
<td>Glassy calcium alumino-silicate by-product from the blast furnaces of the iron and steel industry. For use in concrete, it is granulated and dried before being interground with Portland clinker or blended with the cement. Often called [ground] granulated blastfurnace slag (ggbfs).</td>
</tr>
<tr>
<td>Blended cement</td>
<td>A hydraulic cement complying with NZS 3122:1995 that contains Portland cement and more than 5% of fly ash or blastfurnace slag or both.</td>
</tr>
<tr>
<td>Cementitious material</td>
<td>Hydraulic cement and also any admixture capable of forming cement hydrates.</td>
</tr>
<tr>
<td>Chalcedony</td>
<td>A cryptocrystalline form of quartz found in flints and chert that can be reactive. Not common in New Zealand aggregates.</td>
</tr>
<tr>
<td>Chemical admixture</td>
<td>Chemicals added to concrete during mixing to entrain air, reduce water demand or increase workability, etc.</td>
</tr>
<tr>
<td>Cristobalite</td>
<td>A silica mineral that is a minor component in basalt, andesite, dacite and rhyolite. It is reactive with alcalis.</td>
</tr>
<tr>
<td>Delayed Ettringite Formation (DEF)</td>
<td>Internal sulphate attack on concrete resulting from high curing temperatures to form Ettringite, causing volumetric expansion around the aggregate and resultant cracking and spalling.</td>
</tr>
<tr>
<td>Deleterious aggregate</td>
<td>A category of aggregate reactivity determined by ASTM C289 or C1260.</td>
</tr>
<tr>
<td>Diatomite</td>
<td>Alluvial deposit of opaline silica derived from the relics of diatoms that settled on sea or lake beds.</td>
</tr>
<tr>
<td>Fly ash</td>
<td>Fine, silica-rich ash extracted from the flue gases of a boiler fired with pulverised coal.</td>
</tr>
<tr>
<td>High-alkali cement</td>
<td>Cement in which the alkali exceeds 0.60% Na2O equivalent.</td>
</tr>
<tr>
<td>Hydraulic cement</td>
<td>Cement that hardens by chemical interaction with water and is capable of doing so under water.</td>
</tr>
<tr>
<td>Innocuous aggregate</td>
<td>A category of aggregate reactivity determined by ASTM C289.</td>
</tr>
<tr>
<td>Intermediate rocks</td>
<td>Volcanic rocks in which the SiO₂ content lies between 52% and 66%. Includes andesite, phonolite and dacite.</td>
</tr>
<tr>
<td>Low-alkali cement</td>
<td>Cement in which the alkali does not exceed 0.60% Na₂O equivalent.</td>
</tr>
<tr>
<td>Microsilica</td>
<td>Generic term that refers to extremely finely divided powders of almost pure, often amorphous, silica. Includes silica fume and natural silicas.</td>
</tr>
<tr>
<td>Mineral admixture</td>
<td>See supplementary cementitious material.</td>
</tr>
<tr>
<td>Non-reactive aggregate</td>
<td>Aggregate shown by both testing and field experience not to be reactive with cement alcalis.</td>
</tr>
<tr>
<td>Opal</td>
<td>A highly reactive form of amorphous silica present in some rocks. Rare in New Zealand aggregates.</td>
</tr>
</tbody>
</table>
Appendix A: Glossary

Pessimum proportion
A proportion, less than 100%, of a reactive mineral or species in the aggregates at which maximum concrete expansion occurs. Not all reactive species exhibit this behaviour.

Petrographic examination
The examination of rocks and aggregates in the quarry and the laboratory. It may include any examination or analytical technique considered necessary to characterise aggregate and cement phases in relation to AAR.

Pore solution
The water present in the pores of a concrete. It contains soluble materials such as alkalis, sulphates and calcium. The amount of water in the pores may vary from saturation to zero.

Portland blastfurnace slag cement
Portland cement blended or interground with ground, granulated blastfurnace slag.

Portland limestone cement
A blended hydraulic cement complying with NZS 3125 that contains Portland cement and 5-15% of limestone.

Portland pozzolan cement
A blended hydraulic cement that contains Portland cement interground or blended with pozzolan.

Potentially deleterious aggregate
A category of aggregate reactivity determined by ASTM C289. The term is used to indicate that the aggregates may show pessimum proportion, or may test as deleterious but not react in site concrete.

Potentially reactive aggregate
Aggregate indicated by laboratory testing to be reactive. Such aggregates are only considered potentially reactive unless reaction has occurred under field conditions.

Pozzolan
Finely ground, siliceous material that will react with calcium hydroxide from Portland cement during moist conditions to form additional hydrated cementing products. Common pozzolans are fly ash, silica fume, diatomite, pumicite and geothermal silica.

ppm
Parts per million. Equivalent to mg/kg.

Precision
A statistical term that describes the error of a repeated measurement.

Pumicite
An alluvial deposit of fine material derived from pumiceous rhyolite.

Quartz
A crystalline silica mineral present in many rock types.

Quartzite
Rocks composed of granular quartz which can be either metamorphic or sandstone types. Can be reactive with alkalis.

Quick chemical test
Colloquial name for ASTM C289.

Rapid mortar bar test
Colloquial name for ASTM C1260 and other similar tests.

Reactive aggregate
Aggregate known to have reacted in concrete structures.

Reactive alkali
The alkali in the concrete materials that is available to move into the pore solution of a concrete and take part in AAR.

Reduction in expansion
The difference between the expansion of test specimens of a particular composition designed to reduce ASR and the expansion of control specimens tested at the same time.
### Appendix A: Glossary

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Relative humidity</strong></td>
<td>The percentage of moisture in an atmosphere relative to the amount required for its saturation at any given temperature. Used to define the moisture content of a concrete that is not saturated with water.</td>
</tr>
<tr>
<td><strong>Salt</strong></td>
<td>In this document refers to sodium chloride.</td>
</tr>
<tr>
<td><strong>Silica (SiO₂)</strong></td>
<td>The chemical species that reacts with alkalis. Silica minerals such as opal, chalcedony, cristobalite and tridymite contain SiO₂ in a reactive form. Alumino-silicate glasses such as volcanic and commercial glasses may contain SiO₂ in a form that may react with alkalis under certain circumstances. Many other minerals contain SiO₂ in a form that does not react with alkalis.</td>
</tr>
<tr>
<td><strong>Silica fume</strong></td>
<td>Very fine amorphous silica, a by-product of the reduction of quartz in the manufacture of silicon and ferro-silicon.</td>
</tr>
<tr>
<td><strong>Sodium oxide equivalent</strong></td>
<td>See alkali equivalent</td>
</tr>
<tr>
<td><strong>Specifier</strong></td>
<td>Any person responsible for specifying the materials to be used in a concrete.</td>
</tr>
<tr>
<td><strong>Strong acids</strong></td>
<td>In the context of cement analysis usually refers to either nitric or hydrochloric acids unless otherwise specified.</td>
</tr>
<tr>
<td><strong>Supplementary cementitious material (SCM)</strong></td>
<td>Fine material added to the concrete during mixing to modify fresh and/or hardened properties, in particular those relating to permeability and alkali reactivity. Reacts with or hydrates itself in the presence of portland cement. Includes fly ash, silica fume, blastfurnace slag, metakaolin and natural pozzolans.</td>
</tr>
<tr>
<td><strong>Total alkali</strong></td>
<td>The total alkali content of a material. For Portland cement it is identical to the acid soluble content. For other materials the total alkali will always be greater than either the available or reactive alkalis.</td>
</tr>
<tr>
<td><strong>Tridymite</strong></td>
<td>A silica mineral that is a minor component in basalt, andesite, dacite and rhyolite, and is reactive with alkalis.</td>
</tr>
<tr>
<td><strong>Volcanic glass</strong></td>
<td>The glassy matrix of rocks such as andesite, dacite and rhyolite. These glasses are the main reactive component in New Zealand volcanic rocks. Can also occur as obsidian.</td>
</tr>
<tr>
<td><strong>Water soluble alkali</strong></td>
<td>Alkali extracted by water.</td>
</tr>
</tbody>
</table>
Appendix B: Chemistry of the Alkali Silica Reaction

The idea that aggregates are merely chemically inert fillers used to reduce the volume of cement paste in a concrete to practical levels is no longer tenable. Probably even the most inert of aggregates undergo some superficial chemical reaction with the pore solution. Pore solutions are moderately to strongly alkaline (pH 12.5-14). It has been suggested that many of these superficial reactions confer some physical benefits on the concrete as the outer layers of the aggregate become part of the cement matrix. However, one chemical reaction in concrete that occurs between the alkalis in pore solution and certain varieties of silica in the aggregates can be very damaging. The following chemical description gives a broad view of ASR. Although some of the details of the mechanisms are still being investigated, the basic reaction is now generally understood.

In Portland cement, the alkali metal ions are usually present as soluble sulphates Na₂SO₄ and K₂SO₄ or as the mixed salt (Na,K)₂SO₄. Smaller amounts occur in solid solution in the cement minerals that may be substantially released as the cement hydrates. Alkalis may also be introduced to concrete as sodium chloride, most significantly as contamination of marine derived aggregates by seawater, recycled wash water or, less significantly, from de-icing salts or deposited salt spray. Admixtures are another source of alkalis. Certain aggregates can also release alkalis over time.

When water is added to Portland cement, these alkali sulphates dissolve and the sulphate ions react with the hydrating tricalcium aluminate and calcium hydroxide to precipitate ettringite (calcium sulphaaluminate), releasing the alkalis as sodium and potassium ions into the pore solution. A large proportion of the alkalis are taken up by the cement hydrates as they form. The pore solution in concrete containing Portland cement contains considerable concentrations of sodium and potassium ions and very low concentrations of other ions such as calcium, sulphate and chloride. This concentration of alkalis in the pore solution results in significant hydroxyl concentrations giving rise to pH 13 to 14. It is this high hydroxyl concentration that provides the chemical driving force for ASR.

Sodium chloride is readily soluble in the pore solution and reacts with the aluminate phases in an analogous way to sulphates to form chloroaluminates. Once again the sodium ions are released into the pore solution and contribute to the overall alkali concentration. The extent to which the chloroaluminates form is dependent on both the concentrations of chloride and the aluminate phases present. At the levels of chloride permitted by NZS 3109 in reinforced concrete, complete conversion to chloroaluminates with release of sodium ions can be assumed. This reaction of sodium chloride can take place in both the plastic and hardened states of concrete.

The alkali-aggregate reaction is essentially an attack by hydroxyl ions, with concomitant sodium and potassium ions, on varieties of silica, to produce alkali silicate gel. The rate of this attack will depend both on the types of silica present and the concentration of the alkalis in the pore solution. The formation of the alkali silicate gel consumes alkali and reduces the pH. Only when there is sufficient alkali in the pore solution, equivalent to an initial pH of at least 13.65, does significant attack develop. The gels that form consist of a calcium, sodium, and potassium silicate of variable composition. In the presence of calcium hydroxide, such gels are capable of absorbing water into their structure and expanding. It is this expansive force that creates tensile stress within the concrete and will cause cracking when the local, unrestrained, tensile strength of the concrete is exceeded. The expansive forces are believed to rarely exceed 6-7 MPa, but the force varies, both with the composition of the gel, and with the total amount and distribution of gel present in the concrete.

Spatially, the reaction between aggregate and alkalis to form gel is rarely uniform and nearly always appears as separated point sources of reaction. The point sources are often restricted to a few local areas but in more severe cases will be widespread. It is this tendency to occur as point sources of expansive force that gives the typical, somewhat random to regular pattern cracking seen in concretes.
affected by ASR. In heavily reinforced concrete the crack pattern will follow the direction of the reinforcement.

The amount of gel also depends on the amount of available reactive silica and therefore, up to a point, an increase in the amount of reactive silica produces an increase in expansion. However, above a certain proportion of reactive silica to alkali so much alkali is absorbed that the concentration of hydroxide in solution is insufficient to maintain the same degree of attack and the expansion decreases again. This is the reason for the critical, or "pessimum", proportion that occurs with so many aggregates. Among other properties, SCMs act to reduce the alkalis in solution by incorporating them into additional cementing hydrates formed by reaction of Ca(OH)₂. This reduces the potential for deleterious expansion.

Ettringite is a reaction product of Portland cement hydration at normal in situ curing temperatures. A different product forms when the temperature of the concrete exceeds about 70°C during the early stages of curing. This may happen, for example, in mass concrete or in accelerated curing if temperature increases are not well managed. Once cooler temperatures are restored, it will convert back to ettringite if sufficient water is available. Ettringite takes up more space than the original product so the conversion generates internal stress, which, like ASR, can be enough to crack the concrete. Quillian (2001) summarises the general principles of DEF. The risk of DEF is determined primarily by the early age curing temperatures, although some cement compositions and aggregate types may further increase the risk.

**DEF (Delayed Ettringite Formation)** is often associated with ASR, and there has been considerable debate about the relationship between the two reactions. Where both occur, ASR generally precedes DEF, providing microcracks in which ettringite can crystallise, and a chemical environment in which ettringite is stable. DEF can occur without ASR if all high-risk parameters are present. Bruce et al (2008) summarise the relationship between ASR and DEF as applied to the two damaged Southland bridges.

DEF is very sensitive to small changes in physical and chemical conditions, so the reaction and its effects may be highly localised within a structure, leading to inconsistent observations that make it difficult to determine the cause of damage even when DEF is suspected (Thomas et al 2008). DEF does not necessarily crack the concrete unless sufficient expansive stress is generated. It may, however, expand or extend microcracks initiated by other mechanisms, such as wetting and drying, thermal stress, freezing and thawing, dynamic loads, or localised high stresses in prestressed elements.

DEF does not proceed to a sufficient extent to crack concrete unless enough water is available. Therefore, severe DEF deterioration is generally limited to concrete elements such as piles or foundations that are immersed or partly immersed in water, although elements exposed to rain can also develop DEF. Thus wharves and bridges with foundations in a waterway are more susceptible than other structures. Such elements are also difficult to inspect, so this type of damage may easily be overlooked, particularly in the early stages when it may be masked by biological growth or surface abrasion/erosion.

ASR may be a precursor to DEF damage, but also causes cracking itself. ASR needs less moisture than DEF, and therefore affects not only immersed concrete elements but also any element exposed to rain, run-off, soil moisture, condensation or high humidity.
Appendix C: Sample Calculations

The total alkali content of a concrete mix is calculated as follows:

\[ A = Ac + B + H + W + D \]

where

- \( A \) = total alkali content of the concrete mix
- \( Ac \) = total alkali content of the Portland cement or Portland-Limestone cement (section 7.2)
- \( B \) = total alkali content of SCM admixture (section 7.3)
- \( H \) = reactive alkali contribution made by sodium chloride contamination of both the fine and coarse aggregate (section 7.6)
- \( W \) = total alkali contribution made by the mixing water (section 7.4)
- \( D \) = total alkali contribution made by chemical admixtures and pigments (7.5)
- \( R \) = total alkali contribution made by the aggregate (section 7.6)

The alkali contributed to the concrete mix by the Portland cement or Portland-limestone cement shall be calculated from:

\[ Ac = \frac{C \times a}{100} \]

where

- \( Ac \) = total alkali content of the Portland cement or Portland-limestone cement to the nearest 0.1 kg/m³
- \( C \) = the Portland cement or Portland-limestone cement content of the concrete in kg/m³
- \( a \) = certified maximum percentage of acid soluble alkali content of the Portland cement or Portland-limestone cement.

The term “acid soluble alkali” shall refer to the alkali metals sodium and potassium expressed as their oxides. The alkali content of Portland cements and Portland-limestone cements shall be defined as the percentage mass of equivalent sodium oxide (Na₂O) calculated from:

\[ \% \text{ Na}_2\text{O equivalent} = \% \text{ Na}_2\text{O} + 0.658 \times \% \text{ K}_2\text{O} \]

The method used in determining the acid soluble alkali content of Portland cement and Portland-limestone cement shall comply with the methods given in ASTM C114.
The total alkali contributed by supplementary cementitious materials shall be calculated from:

\[ B = \frac{E \times f}{100} \]

where

- \( B \) = average total alkali content contributed by the SCM
- \( E \) = the SCM content of the concrete in kg/m³
- \( F \) = total alkali content of SCM

The method used in determining the total alkali content of SCM shall comply with the methods specified in the relevant standard specification for the SCM(s). Note that for blastfurnace slag and fly ash it may be necessary to include all, some or none of the alkali depending on the quantity of the material used.

The reactive alkali contributed by sodium chloride contamination of the aggregates shall be calculated from:

\[ H = \frac{0.76 \times [(NF \times MF) + (NC \times MC)]}{100} \]

where

- \( H \) = reactive alkali contribution made to the concrete by the sodium chloride present in the aggregates expressed as kg/m³ sodium oxide equivalent
- \( NF \) = chloride ion content of the fine aggregates as a percentage by mass of dry aggregates
- \( MF \) = fine aggregate content in kg/m³
- \( NC \) = chloride ion content of the coarse aggregate as a percentage by mass of dry aggregates
- \( MC \) = coarse aggregate content in kg/m³

The factor 0.76 is derived from the conversion of chloride ion to sodium oxide equivalent and the composition of seawater.

The chloride ion content of the coarse and fine aggregates used in the concrete shall be determined at agreed intervals in accordance with BS 812: Part 117. When the total chloride ion level in the aggregates is less than 0.005% it shall be regarded as nil.
Appendix C: Sample Calculations

The alkali contributed by the mixing water to the concrete shall be calculated from:

\[ W = \frac{(Na \times 1.35 + 0.658 \times K \times 1.20) \times C \times W/C}{1,000,000} \]

where  \( W \) = equivalent alkali contributions made to the concrete by the alkali ions sodium and potassium present in the water in kg/m\(^3\)

\( Na \) = sodium ion content present in the water as parts per million (mg/l)

\( K \) = potassium ion content present in the water as parts per million (mg/l)

\( C \) = Portland cement or Portland-limestone cement content of the concrete in kg/m\(^3\)

\( W/C \) = the water cement ratio

The water shall be analysed for sodium and potassium ions by recognised methods such as those published by the American Public Health Association. Where the equivalent alkali (calculated as Na + 0.585K) present in the water is less than 190 mg/L (ppm) it shall be treated as nil.

The alkali contributed to the concrete by chemical admixtures and/or pigments shall be calculated from:

\[ D = \frac{y \times C}{100} \times \frac{Z}{100} \times 100 \quad \text{or} \]

\[ D = \frac{y \times C}{100} \times \frac{Z \times d}{100} \]

where  \( D \) = alkali contribution made to the concrete by chemical admixtures and/or pigments to the concrete in kg/m\(^3\)

\( C \) = the Portland cement or Portland-limestone cement content of the concrete in kg/m\(^3\)

\( y \) = sum of the percentage reactive alkali contents of chemical admixtures and/or pigments

\( Z \) = weight of solid admixture or pigment added to the concrete per 100kg of cement

\( Z_1 \) = volume of liquid admixture or pigment added to the per 100 kg of cement

\( d \) = density of liquid admixture or pigment

The alkali contributed to the concrete by recycled concrete aggregate shall be calculated from:

\[ R = g \times 0.002 \]

where  \( R \) = alkali contribution made by the recycled aggregate in Na\(_2\)O equivalent

\( g \) = the weight of recycled aggregate kg/m\(^3\) of concrete
The following example shows how the alkali content of a concrete containing no SCM might be evaluated. It is based on Appendix 2 of the 1991 edition of this publication, and is not intended to represent a commercial mix design.

A structure is to be built to a 50 year service life requirement. A compressive strength of 45 MPa is specified, with no other special requirements so the concrete is deemed to be Normal Concrete. Non-reactive aggregate is uneconomic, and the strength can be achieved without an SCM. Hence the concrete needs to have a total alkali content of 2.5 kg/m³ or less.

The supplier proposes a mix design with Portland cement content of 385 kg/m³. The cement the supplier proposes to use has a maximum acid soluble alkali content of 0.60% Na₂O equivalent.

The alkali content of the mix constituents is:

<table>
<thead>
<tr>
<th>Material</th>
<th>Mix Proportions</th>
<th>Alkalis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement</td>
<td>385 kg/m³</td>
<td>Na₂O equivalent 0.60%</td>
</tr>
<tr>
<td>Coarse aggregates</td>
<td>1,125 kg/m³</td>
<td>-</td>
</tr>
<tr>
<td>Fine aggregates:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Source (a)</td>
<td>420 kg/m³</td>
<td>-</td>
</tr>
<tr>
<td>Source (b)</td>
<td>380 kg/m³</td>
<td>Chloride ion content 0.03%</td>
</tr>
</tbody>
</table>

**Case A**

<table>
<thead>
<tr>
<th>Typical admixture</th>
<th>Manufacturer's recommended dosage</th>
<th>Na₂O equivalent 0.01kg per 100kg Portland cement when used at recommended dosage</th>
</tr>
</thead>
</table>

**Case B**

<table>
<thead>
<tr>
<th>Superplasticiser</th>
<th>Manufacturer's recommended dosage</th>
<th>Na₂O equivalent 0.1kg per 100kg Portland cement</th>
</tr>
</thead>
</table>

The total alkali content of the concrete in each case is calculated as shown in the table on the following page.

In case A the total alkali content is 2.4kg/m³, which is acceptable, but in case B it is above the 2.5 kg/m³ limit. The options for case B are to:

(a) Select a superplasticiser with an alkali content that will reduce the concrete alkalis to less than 2.5 kg/m³; or

(b) Consider the use of an appropriate amount of SCM as part replacement of the Portland cement; or

(c) Specifically request that cement be supplied with an acid soluble alkali level of 0.50% or less.

The most appropriate option is selected by the supplier. If an SCM is used (option [b]) and the concrete alkali content remains above 2.5 kg/m³, the concrete becomes Special Concrete. The supplier must then advise the specifier of the change, and must provide evidence such as calculations or test results to demonstrate that this will provide adequate protection against ASR damage in accordance with Section 5.4.1.
### Appendix C: Sample Calculations

<table>
<thead>
<tr>
<th></th>
<th>Case A</th>
<th></th>
<th>Case B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Alkali content using typical admixture (kg/m³)</td>
<td>Alkali content using a superplasticiser (kg/m³)</td>
<td></td>
</tr>
</tbody>
</table>
| Portland cement         | \[
\frac{385 \times 0.60}{100} = 2.3
\] | \[
\frac{385 \times 0.60}{100} = 2.3
\] |
| Coarse Aggregate        | -       | -                        | -       |
| Fine aggregates:        |         |                          |         |
| Source (a)              | -       |                          | -       |
| Source (b)              | \[
\frac{260 \times 0.03 \times 0.76}{100} = 0.06
\] | \[
\frac{380 \times 0.03 \times 0.76}{100} = 0.09
\] |
| Admixture               | \[
\frac{385 \times 0.01}{100} = 0.04
\] | \[
\frac{385 \times 0.1}{100} = 0.39
\] |
| Water                   | Alkali > 190ppm = 0.00 | = 0.00 |
| Total concrete alkali content | **2.40** | **2.78** |
Appendix D: Results of Testing Aggregates Using ASTM C289

The results of aggregate tests using ASTM C289 that were carried out by DSIR Chemistry are given in Figures D1-D6. The diagram used is divided into two areas by the solid curve given in each figure. Samples that plot to the left of the curve (area A) are designated innocuous and those that fall to the right of the curve (areas B and C) are considered to be potentially reactive. The right hand area is subdivided into areas B and C. Results that plot in area B are designated potentially deleterious and may show pessimum proportion. Those in area C are designated as deleterious and are unlikely to exhibit pronounced pessimum proportion.

When interpreting the results of ASTM C289, care should be taken to avoid over reliance on individual results. The demarcation curve is not a sharp boundary but rather the central point of a region where reactivity grades into non-reactivity. If a result plots close to the line in the innocuous area, while other results for similar material derived from the same general locality plot to the right, it is unwise to accept the aggregate as innocuous without further sampling and retesting. ASTM C289 is sensitive to sample preparation. This is illustrated by the results for samples 54 and 55 where the difference found is due to rewashing the prepared test sample. It will often be found that repeated tests on aggregates give some variation in results. Usually $R_c$ is more variable than $S_c$ and as a result these variations are normally not unduly significant.

Results from testing volcanic aggregates are plotted in broad groups that follow the classical petrographic subdivision of rocks by their silica content. These are:

- Acid rocks, 66-75% silica - Rhyolite and Dacite
- Intermediate rocks, 52-66% silica - Andesite
- Basic rocks, 40-52% silica - Basalt

The three groups grade into each other, and near the boundaries of the groups other petrographic criteria may be used to decide in which group they belong. Although this subdivision of rocks based on silica content is broad, in New Zealand it is convenient for practical purposes because most basalt is innocuous, most andesite rocks are reactive, and dacite and rhyolite are almost invariably highly reactive.

Results of testing Egmont andesite from the Taranaki area are plotted in Figure D1. All the samples tested are alluvial except for sample 136, which is from a lahar. Most of the rocks clearly test as deleterious without a pessimum proportion, which has been confirmed by both mortar bar and some concrete tests. In addition, field experience has shown the rocks to be reactive. Some of the old samples test as innocuous. In the case of 44B, it is now known that this fine fraction contained iron sand, which is absent from the coarse fraction of sample 44A and this may explain the result. In the cases of samples 45 and 47 the differences between the coarse and fine aggregate fractions are contradictory, while the results from samples 49 and 50 indicate that these samples need further testing. This retesting has now been carried out and clearly shows the deleterious nature of these samples. The results from testing and field experience with Egmont andesite indicate this andesite should be considered to be potentially reactive.

Basalt aggregate is used extensively in the upper half of the North Island and has tended to be considered as innocuous and not require testing. In some cases this may be a dangerous assumption as the results plotted in Figure D2 show. Ongaroto basalt, sample 52, is clearly deleterious, a result confirmed by limited mortar bar tests. Initially it was considered that parts of Ongaroto basalt may be doleritic, but recent work shows that it has a silica content of 51% and the rock is verging on an andesite. The assumption that basalt is innocuous is reasonable, provided that it is true basalt and not tendency to andesite. For example, samples 147 and 148 from a major Auckland quarry contain a silica content of
Appendix D: ASTM C289 Aggregate Test Results

approximately 45% and clearly test as innocuous. Basalt that is trending to andesite is difficult to detect in the field and may not be distinguished under the petrographic microscope without the aid of chemical analysis. This is the case for sample 144, which was taken from different depths of a drill core, and shows a marked movement towards reactivity.

The results from the andesite samples from the central North Island volcanic region are plotted in Figure D3, together with two samples from isolated andesite cones to the north of Lake Taupo. A wide range of results is apparent indicating the samples are variable. This is not unexpected given the large extent and complexity of this area of andesite rocks. A further complicating factor is that the samples represent both hard rock and alluvial aggregates. Limited mortar bar tests show that the andesite can be reactive with a broad pebblesmooth proportion. In some cases it is difficult to see why large differences have occurred, as for instance between samples 35 and 36. While from different rivers, both samples contain similar alluvial materials and should give similar results. Once again it must be stressed that it is unwise to base interpretation on an individual result. Andesite from the central region finds limited use as a concrete aggregate. It is mainly of concern as it is present in many of the rivers that drain this central North Island area. However, rhyolite is also often present in these rivers and tends to be the major cause of the reactivity found in these alluvial aggregates.

The andesites from the Bay of Plenty and Coromandel areas form a complex and variable group of rocks that are somewhat older and may be more altered than the more recent andesites of the central and Taranaki regions. The scatter of results plotted in Figure D4 are indicative of this complexity. Most of the samples tested appear reactive. Some of the innocuous samples are altered rocks which may explain their lack of reactivity.

The results of testing rhyolite and dacite are plotted in Figure D5. It includes material from both the central North Island and Bay of Plenty regions. Apart from samples 64 to 69, which are alluvial greywackes mixed with rhyolite, all the rocks are reactive. Of greatest interest are the rhyolite rock samples 80 to 88, excavated from the bed of the upper Waikato River during hydro construction. These samples are potentially deleterious in the extreme. It is similar rhyolite, somewhat diluted by other materials as it travels down the river, which is extensively dredged from the lower Waikato River and used as concrete sand in the Auckland area. These are represented by samples 106, 107, and 151 to 157. Overall, the results clearly show that all concrete aggregates containing rhyolite must be considered as reactive with high-alkali cements unless adequate investigation shows otherwise. Field evidence indicates that preventative measures must be applied to avoid a damaging reaction from occurring with the use of these aggregates in structures.

Figure D6 contains the results from samples of greywacke, quartzite, marine quartz sands, foreign and synthetic samples. All the New Zealand greywacke samples test as innocuous. Extensive laboratory testing and field experience confirms that our greywackes are non-reactive in concrete at all alkali levels. This is not the case for many foreign greywackes and argillites so this overseas experience should not be extrapolated to our New Zealand rocks. The Cobb quartzite, samples 77 to 79, is deleterious and has been confirmed by mortar bar tests. This parallels overseas experience where many quartzites are proving to be reactive. Sample 112, which is a siliceous magnesian limestone containing highly reactive opaline material, was the first reactive aggregate to be identified by Stanton in California. It is potentially deleterious and exhibits extreme pebblesmooth proportion in the mortar bar test. In contrast sample 117, a hornfelsed shale from Malmesbury in the South Western Cape of South Africa plots on the demarcation line. Mortar bar tests have failed to show that it is very reactive but field experience indicates that extensive damage in hundreds of structures is occurring where this aggregate has been used with high-alkali cement. It is results of this nature which indicate that while ASTM C289 is applicable to volcanic rocks and those containing opal and chalcedony, its use is more limited for other rock types.

Alkali silica reactions associated with high curing temperatures and the forms of silica observed in the concretes sampled in the South Island bridges project tend to be slower than the ASR associated with glass in fresh volcanic rock and other amorphous types of silica. Consequently, the reactivity of aggregates containing these more crystalline forms is not always detected by the ASTM C289 quick chemical test. Therefore, in countries where ASR has been associated with the more crystalline silica forms, more aggressive tests (such as the rapid mortar bar tests described in section 6.4.2) are now used instead of ASTM C289 as rapid screening tests.
Overall, ASTM C289 is a rapid indicator of the potential reactivity of New Zealand volcanic rocks and has been adequate for indicating whether control of the alkali in concrete is necessary and whether further investigation of their use as concrete aggregates may be required. Provided that relevant geological and petrographic data are taken into account when making the assessments, the test is an important and useful tool for the concrete engineer. However, in regions such as the Bay of Plenty/Coromandel and Northland, sufficient laboratory testing confirmed by field experience is not yet available and caution is required when interpreting the results of testing aggregates from these regions by ASTM C289.

List of Aggregates Tested

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Date tested</th>
<th>Testing Agency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eastern andesites</td>
<td>1 - 17</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>118 - 124</td>
<td>1988</td>
</tr>
<tr>
<td>Central andesites</td>
<td>18 - 43</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>138</td>
<td>1988</td>
</tr>
<tr>
<td>Taranaki andesites</td>
<td>44 - 50</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>130 - 136</td>
<td>1988</td>
</tr>
<tr>
<td>Basalt</td>
<td>51 - 53</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>139 - 145</td>
<td>1988</td>
</tr>
<tr>
<td></td>
<td>147 - 150</td>
<td>1990</td>
</tr>
<tr>
<td>Dacite</td>
<td>54 - 55</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>137</td>
<td>1988</td>
</tr>
<tr>
<td>Greywacke + volcanics</td>
<td>64 - 69</td>
<td>1960</td>
</tr>
<tr>
<td>Greywacke</td>
<td>56 - 63</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>163 - 167</td>
<td>1990</td>
</tr>
<tr>
<td>Ignimbrite</td>
<td>70 - 71</td>
<td>1960</td>
</tr>
<tr>
<td>Jasper</td>
<td>72</td>
<td>1960</td>
</tr>
<tr>
<td>Obsidian</td>
<td>73</td>
<td>1960</td>
</tr>
<tr>
<td>Pitchstone</td>
<td>74</td>
<td>1960</td>
</tr>
<tr>
<td>Pumice</td>
<td>75 - 76</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>129</td>
<td>1988</td>
</tr>
<tr>
<td></td>
<td>157</td>
<td>1990</td>
</tr>
<tr>
<td>Quartzite</td>
<td>77 - 79</td>
<td>1960</td>
</tr>
<tr>
<td>Rhyolite</td>
<td>80 - 93</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>125 - 128</td>
<td>1988</td>
</tr>
<tr>
<td>Waikato River</td>
<td>94 - 107</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>151 - 156</td>
<td>1990</td>
</tr>
<tr>
<td>Marine sands</td>
<td>108 - 110</td>
<td>1960</td>
</tr>
<tr>
<td></td>
<td>158 - 162</td>
<td>1990</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>111 - 117</td>
<td>1960</td>
</tr>
</tbody>
</table>
### Appendix D: ASTM C289 Aggregate Test Results

<table>
<thead>
<tr>
<th>Quarry or Source</th>
<th>Location</th>
<th>Rock Type(s)</th>
<th>Metric Grid Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2 Woodlands</td>
<td>Woodlands, Tauranga</td>
<td>Eastern andesite</td>
<td>T13/659063</td>
</tr>
<tr>
<td>3 Thompsons Track (NC)</td>
<td>Aongatete</td>
<td>Eastern andesite</td>
<td>T14/64-93-</td>
</tr>
<tr>
<td>4 Te Poi (NC?)</td>
<td>Te Poi</td>
<td>Eastern andesite</td>
<td>T14/650728</td>
</tr>
<tr>
<td>5 Coles</td>
<td>Te Poi</td>
<td>Eastern andesite</td>
<td>T14/650727</td>
</tr>
<tr>
<td>6 Barton Road</td>
<td>Okauia</td>
<td>Eastern andesite</td>
<td>T12/432415</td>
</tr>
<tr>
<td>7 Matatoki</td>
<td>Matatoki</td>
<td>Eastern andesite</td>
<td>T13/44-14-</td>
</tr>
<tr>
<td>8 Tirohia (NC?)</td>
<td>Tirohia</td>
<td>Eastern andesite</td>
<td>T13/457153</td>
</tr>
<tr>
<td>9-9A Leachs</td>
<td>Tirohia</td>
<td>Eastern andesite</td>
<td>U14/995805</td>
</tr>
<tr>
<td>10 McNaughtons</td>
<td>Papamoa</td>
<td>Eastern andesite</td>
<td>U14/992802</td>
</tr>
<tr>
<td>11 Duncans</td>
<td>Papamoa</td>
<td>Andesite, rhyolite</td>
<td>U14/992848</td>
</tr>
<tr>
<td>12 Papamoa Beach</td>
<td>Otaramakau</td>
<td>Andesite, rhyolite</td>
<td>V15/259667</td>
</tr>
<tr>
<td>13-14 Otaramakau Beach</td>
<td>Te Aroha</td>
<td>Eastern andesite</td>
<td>T13/504033</td>
</tr>
<tr>
<td>15 Tauranga River (NC?)</td>
<td>Tauranga County</td>
<td>Quartz, volcanic glass</td>
<td>?</td>
</tr>
<tr>
<td>16 Paegaroa River (NC?)</td>
<td>Tauranga County</td>
<td>Quartz, volcanic glass</td>
<td>?</td>
</tr>
<tr>
<td>17 Borough Council</td>
<td>Te Aroha</td>
<td>Eastern andesite</td>
<td>T19/524442</td>
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<tr>
<td>18A-B Parakaua Gorge (NC)</td>
<td>Orakeikorako</td>
<td>Central andesite</td>
<td>U17/868941</td>
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<tr>
<td>19 Rolles Peak (NC)</td>
<td>Tauhara</td>
<td>Central andesite</td>
<td>U18/920768</td>
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<tr>
<td>20A-B Tongariro River</td>
<td>Turangi</td>
<td>Andesite, greywacke</td>
<td>T19/524442</td>
</tr>
<tr>
<td>21 Mt Tihia (NC)</td>
<td>Turangi</td>
<td>Central andesite</td>
<td>S19/256287</td>
</tr>
<tr>
<td>22 Papamanuka Stream</td>
<td>Taurewa</td>
<td>Central andesite</td>
<td>T19/444406</td>
</tr>
<tr>
<td>23 Wanganui River</td>
<td>Tongariro</td>
<td>Central andesite</td>
<td>T19/320342</td>
</tr>
<tr>
<td>24A-B Mangatepopo Prison</td>
<td>Taurewa</td>
<td>Central andesite</td>
<td>T19/320342</td>
</tr>
<tr>
<td>25 Tongariro Stream</td>
<td>Access 15, Desert Road</td>
<td>Greywacke, rhyolite, andesite</td>
<td>T20/503157</td>
</tr>
<tr>
<td>26 Waikaha Stream</td>
<td>Rangipo</td>
<td>Argillite, greywacke, volcs</td>
<td>T19/547255</td>
</tr>
<tr>
<td>27 Papakai (NC)</td>
<td>Papakai</td>
<td>Central andesite</td>
<td>T19/365358</td>
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<td>T19/365358</td>
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<td>Otukou</td>
<td>Central andesite</td>
<td>T19/392379</td>
</tr>
<tr>
<td>30 Otukou</td>
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<td>38 Mangateotoenui Stream</td>
<td>Waiouru</td>
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<td>Tokoroa</td>
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</table>
## Appendix D: ASTM C289 Aggregate Test Results

<table>
<thead>
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### Appendix D: ASTM C289 Aggregate Test Results

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### Appendix D: ASTM C289 Aggregate Test Results

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Figure D1: Results of testing Egmont andesite from Taranaki by ASTM C289.
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Figure D2: Results of testing basalt by ASTM C289.
Figure D3: Results of testing andesite from the central North Island by ASTM C289.
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Figure D4:  Results of testing andesite from the eastern region of the North Island by ASTM C289.
Figure D5: Results of testing rhyolite, dacite and some alluvial materials containing these rock types by ASTM C289.
Figure D6: Results of testing greywacke, miscellaneous, foreign and synthetic samples by ASTM C289.
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