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BUILDING RESILIENCE

TR 3
ALKALI SILICA REACTION

Minimising the Risk of Damage to Concrete

Guidance Notes and Recommended Practice

2021 REVISION

ACKNOWLEDGEMENTS

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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 the development and adoption of appropriate precautions in areas where reactive aggregates were used.

The first edition of TR3, published in 1991, was developed by a Working Party instigated in 1988 after the discovery of AAR in a motorway bridge. Chaired by David Barnard of the Cement & Concrete Association of New Zealand (CCANZ), the Working Party's brief was to prepare guidelines that could be used to minimise the risk of structural damage caused by AAR in future construction in New Zealand. The 1991 edition was based on guidelines published by the UK Concrete Society in 1987, augmented by findings from extensive New Zealand research by DSIR Chemistry Division and the Ministry of Works and Development during the 1950's and 1960's. While all members of the Working Party contributed to information gathering and decisions required to produce this document, D.A. St John of DSIR Chemistry Division was the principal writer, reflecting his considerable experience of New Zealand alkali aggregate reactions and knowledge of international research.

Ongoing work by DSIR/Industrial Research Limited and MWD/Works Consultancy/Opus International Consultants Limited clarified the extent and severity of the reaction in New Zealand, and explained some unusual cases of AAR observed in concrete structures. Subsequent issues of TR3 in 2003 and 2012 incorporated these findings along with developments in international practice. Major changes were adoption in 2003 of the risk-based approach taken by RILEM and the Canadian Standards Association along with model specification clauses for normal concrete and special concrete, and the acknowledgement in 2012 of South Island AAR cases.

By 2016, the widespread use of concrete grades that pushed the limits of TR3 recommendations for normal concrete, plus pressures to optimise the use of increasingly limited aggregate resources, prompted Concrete NZ to reconvene the Working Party to review the provisions of TR3 (2012), particularly the prescriptive limits on concrete alkali content in normal concrete. Between 2017 and 2020, extensive laboratory investigations were carried out by Concrete NZ in conjunction with Cement Concrete & Aggregates Australia (CCAA) using internationally accepted test methods to determine appropriate alkali thresholds for several New Zealand reactive aggregates. The results were used to inform the 2021 edition of TR3.

As noted in the preface to the 1991 edition, AAR is a complex topic. This edition of TR3 provides sufficient information based on current NZ experience and internationally accepted practices to enable the wider concrete industry to minimise the risk of ASR damage in new concrete structures. It will be updated again as research, field experience, and international practice identify further improvements.

THIRD EDITION (2021)

This edition supersedes the 2012 revision. The principal changes incorporated in this edition are:

- The maximum concrete alkali limit for normal concrete has been increased to 2.8 kg/m^3;
- Testing protocols have been updated to reflect international practice;
- The provisions covering the use of supplementary cementitious materials have been updated;
- Appendices have been expanded to include modern testing methodologies.

The recommendations of this technical document comprise an acceptable solution to the requirements of NZS 3104 and NZS 3101 for minimising the risk of ASR damage in new concrete in New Zealand.

CONTENTS

EXECUTIVE SUMMARY

Alkali silica reaction (ASR) in concrete involves hydroxyl ions and associated sodium and potassium ions reacting with reactive silica minerals found in rock types used in some aggregates to produce an expansive alkali silica gel. The rate and extent of the reaction depends on the types of silica mineral present, the concentration of alkalis in the concrete pore solution, and the presence of moisture. The degree to which the gel expands depends on sufficient water being available, and the compositions of the gel and pore solutions. When restrained, the expansive stress generated may be enough to crack the concrete. Unrestrained expansion can cause the entire element to deform, closing movement joints and damaging adjacent elements. Significant deformation may occur in large or lightly reinforced elements.

The principal source of alkalinity in the pore solution of concrete is from Portland cement, which contains sodium and potassium from the rocks used in its production. Other sources of alkalis are from salts found in some aggregates, chemical admixtures and supplementary cementitious materials (SCMs). When SCMs are used in sufficient quantities in concrete, they modify the chemistry of the pore solutions, binder hydration products, and alkali silicate gel thereby reducing the potential for deleterious ASR expansion. The minimum amount of SCM needed to replace Portland cement to limit ASR damage depends on the specific aggregates and SCM used.

Reactive aggregates generally contain glassy or cryptocrystalline silica, such as that found in some acid or intermediate volcanic rocks. In New Zealand, most reactive aggregates are derived from rocks such as andesite, dacite, rhyolite, quartzite, volcanic glass, and some basalts with high silica content. The actual reactivity of a rock type in concrete in concrete depends on its particle size distribution, composition, and on the combination of rock types present as well as the concrete mix design. The potential reactivity of aggregates is assessed by, petrography, chemical methods, accelerated mortar bar testing, concrete prism testing, or field experience.

Minimising the risk of ASR damage in concrete consists of the following aspects:

- Identifying whether any of the aggregates used in the concrete are potentially alkali reactive;
- If a potentially alkali reactive aggregate is to be used, minimising the risk of significant damage by:

  Either: Limiting the total concrete alkali content (this is the default for normal concrete)

  Or: Using sufficient SCM in the concrete to prevent significant ASR damage (minimum quantities recommended are 8% for silica fume or microsilica, 15% for diatomite or metakaolin, 25% for pumicite or ASTM Class F fly ash, and 35% for ASTM Class C fly ash or blast-furnace slag.

Recommended practice for minimising the risk of ASR in New Zealand concrete structures is provided, using the following general principles:

- For normal concrete, the alkali content is limited to a maximum of 2.8 kg/m^3
- For special concrete, different precautions are recommended depending on the risks associated with environmental effects and features of the individual structure or element; options are based on rejecting the reactive aggregate, limiting the concrete alkali content, using an appropriate amount of SCM, or other measures that testing of the proposed project mix demonstrates to be effective in minimising the risk of ASR damage.

This document also summarises aggregate resources of New Zealand and identifies typical reactive and non-reactive materials based on previous experience and laboratory testing. Appendix F presents results from ASTM C289 tests carried out on New Zealand aggregates between 1960 and 1990, representing the most comprehensive summary of aggregate reactivity in New Zealand. Subsequent testing to inform the 2021 revision is summarised in Appendix E.

1.0 INTRODUCTION

Alkali silica reaction (ASR) is a specific type of alkali aggregate reaction (AAR). Another type of AAR, alkali carbonate reaction (ACR), occurs with some types of limestone aggregate. In New Zealand, limestone aggregate is not widely used, consequently no cases of ACR have been observed to date here and the terms AAR and ASR tend to be used interchangeably.

ASR is the specific type of alkali aggregate reaction that is of concern in New Zealand, and is the focus of this document. This document does not apply to other types of AAR unless specifically mentioned.

1.1 SCOPE AND PURPOSE OF TR3

This document provides a comprehensive review of ASR in concrete based on New Zealand and international experience. Guidance is provided for designers, concrete producers, and suppliers of other concrete materials about minimising the risk of significant damage induced by ASR in new concrete structures in New Zealand.

The practice recommended herein applies to concrete containing:

- Any 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 (e.g. from new or untested sources).

The recommendations in this document do NOT apply to concrete containing neither potentially reactive aggregates nor aggregates of unknown reactivity.

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). The approach for normal concrete assumes a 50-year specified intended life in accordance with *New Zealand Building Code* requirements or a 100-year specified intended life for structures designed in accordance with the Waka Kotahi NZ Transport Agency's *Bridge Manual* and that minor ASR damage is acceptable. The designer should consider designating the concrete as special concrete if one or both of these assumptions are inappropriate for an individual structure.

1.3 SCOPE OF 2021 REVISION

This edition incorporates a revision of the text of the 2012 amendment of the 2003 edition plus updated or additional information on the following:

- Review of concrete alkali levels in section 2 based on recent research finding using ASR testing methods considered international best practice at the time of writing (research findings are summarised in Appendix E),
- Update on testing protocol for assessing potential aggregate reactivity; and
- Effects associated with supplementary cementitious material (SCMs).

The methodology used in the 2003 edition has been retained and revisions represent an update of guidance based on research on local materials and international practice in the field. The increased use of SCMs in concrete for a wide range of purposes has also made it important to review these materials, especially when these are used at differing dosages.

Whilst some research has been undertaken in recent years this is not comprehensive enough to replace information based on older methods previously used to characterise the reactivity of New Zealand concrete aggregates.

Apart from minor editorial changes, this revision did not significantly update the content presented in section 4 and 6 and readers are encouraged to seek current information on these topics.

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 3104:2021.

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 using concrete prism testing (CPT) as described in section 7.3. If CPT expansion is below the threshold level, then the aggregate can be treated as non-reactive.

(c) If CPT shows expansion or potential expansion above the threshold level, then assume the aggregate is reactive and proceed to items (d) onwards.

(d) For Normal Concrete containing potentially reactive aggregate, apply a maximum concrete alkali limit of 2.8 kg/m^3.

(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), and appropriate preventative measures (see Table 4).

(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, ensure 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, ascertain whether the proposed quantity of the SCM is less than the recommended minimum for mitigating ASR expansion. If it is, either carry out mortar or concrete tests to ascertain whether the proposed quantity of SCM will mitigate ASR expansion, or 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 containing aggregates that may release alkalis (e.g. some Auckland basalts) and potentially alkali reactive aggregates (e.g. Waikato River sand) unless test results or field experience show that 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.

2.2 RECOMMENDED PRACTICE

Recommended practice in this section is for New Zealand concrete construction and follows the principles originally used by the Canadian Standards Association (CSA, 2000) and RILEM (AAR-7).

2.2.1 SPECIFICATION

The heading and clauses (a) to (c) below are provided as model specification clauses. Dialogue between designer, concrete producer, and suppliers of other materials 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 sections 6 and 7.*

(b) *If the aggregate supplier and/or concrete producer can confirm that the proposed aggregates are non-reactive as defined in clause 6.1 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 then the following precautions shall be taken:*

   i For Normal Concrete as defined by NZS 3104:2021 the concrete producer shall certify that the alkali content in the concrete shall not exceed 2.8 kg/m^3 from all sources.

   ii Where the concrete producer cannot certify that the alkali content does not exceed 2.8 kg/m^3 then the concrete shall be designated as Special Concrete. The designer must be informed of the change in designation, and must take appropriate action.

   iii For Special Concrete, the designer shall evaluate and specify the risks associated with ASR according to clauses 2.2.3 and 2.2.4 of TR3 (2021), and shall specify the appropriate level of precaution required to minimise the risk of ASR damage according to clause 2.2.5 of TR3 (2021). The concrete producer shall then select appropriate preventive measures according to clause 2.2.6 of TR3 (2020), and shall provide supporting evidence such as calculations and/or test results to demonstrate to the designer that the selected preventive measures will be effective.

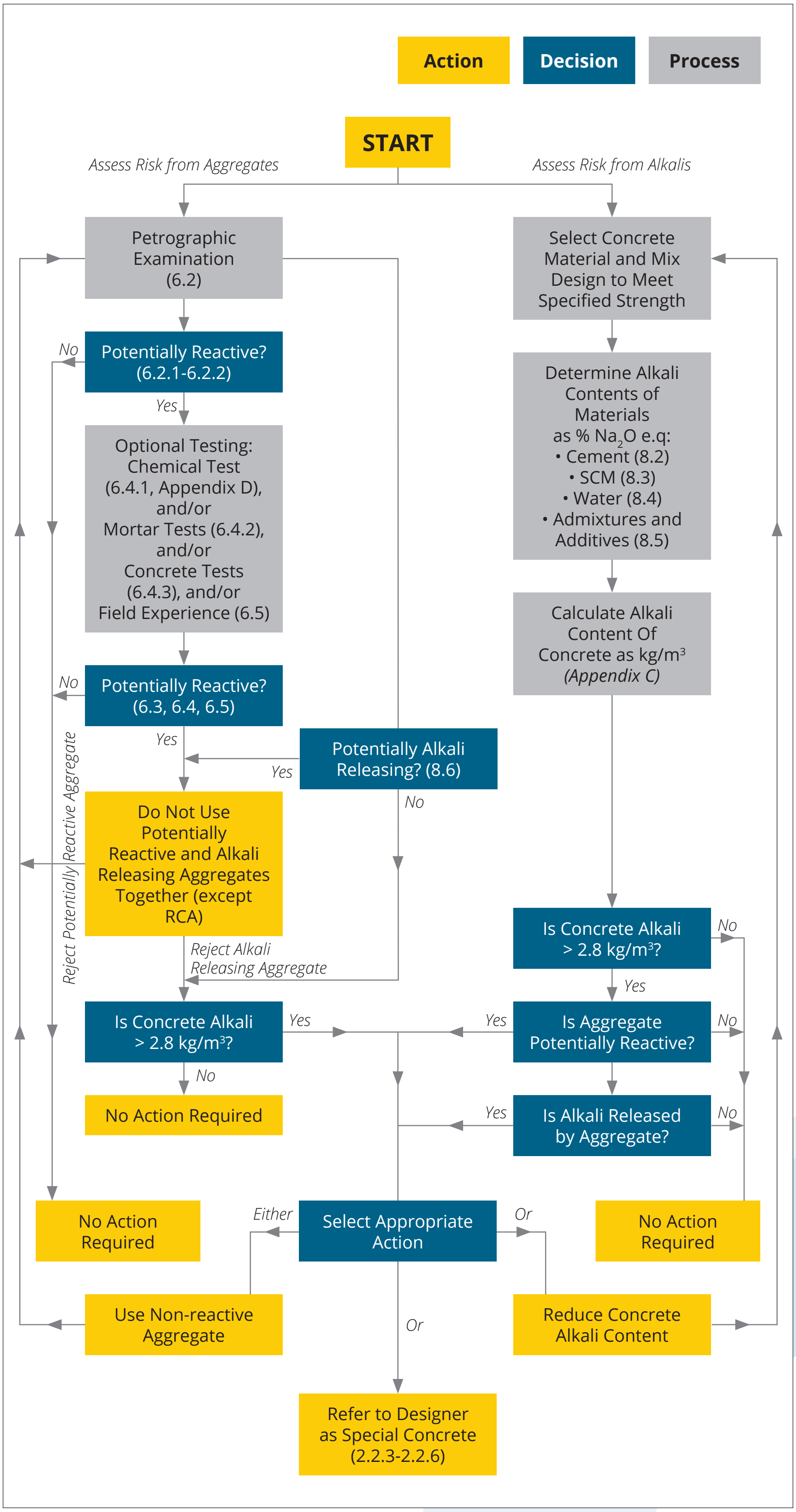
2.2.2 NORMAL CONCRETE

The restriction of total alkalis to 2.8 kg/m^3 or less is based on a level of prevention suitable for a structure designed for a 50-year specified intended life in accordance with *New Zealand Building Code* requirements (or a 100-year specified intended life for a structure designed in accordance with the Waka Kotahi NZ Transport Agency's *Bridge Manual*) and assumes that minor ASR damage is acceptable. The designer should consider designating the concrete as special concrete if for an individual structure the specified intended life or level of acceptable damage differ from these requirements.

Natural aggregates that release alkalis should not be used with reactive aggregates in Normal Concrete but should be evaluated as a Special Concrete. Provisions for alkali in recycled concrete aggregate are given in section 8.6.1.

The process of assessing the risk of ASR in Normal Concrete is described in Figure 1.

Figure 1: Procedure for assessment of potential reactivity of aggregate and binder combinations for normal concrete.



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 ***designer*** 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.

Environmental Risk Category	Size and Environmental Exposure of Structure
E1	Non-massive[1] and dry[2], e.g. a damp-proofed floor in dry service conditions
E2	Massive and dry[2,3]; all concrete exposed to humid air, condensation, rain, run-off, groundwater, sea water[4], or other sources of moisture. *e.g. building facades, foundations, concrete elements in a building enclosing a swimming pool or laundry, water-retaining structures.*
E3	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. *e.g. concrete in the splash zone of a marine structure; concrete in tropical environments; concrete exposed to moisture and elevated temperatures such as a cooling tower or chimney*

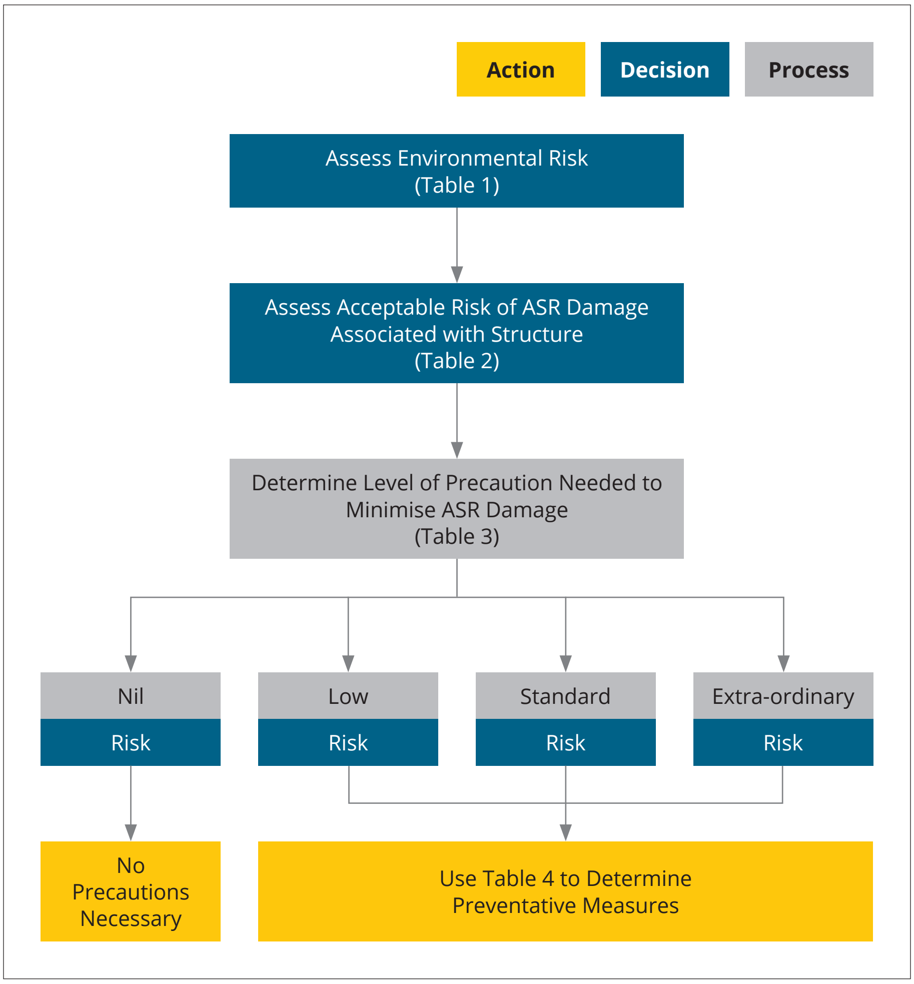
1 A massive element has a least dimension of 0.5 m or more.

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 into concrete designed for constant immersion in sea water is usually limited to a few centimetres.

Figure 2: Procedure for assessing the risk and selecting preventive measures for Special Concrete.



2.2.4 SPECIAL CONCRETE: IDENTIFICATION OF THE RISK OF ASR DAMAGE ASSOCIATED WITH THE STRUCTURE

In accordance with Table 2, the designer 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 maintaining and managing the structure, in co-operation with the designer 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 which 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 more than 100 years 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.

Structure Risk Category	Features of Structure
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 specified intended 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 specified intended life 50 or 100 years[1].*
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 specified intended life of 100 years or longer[1].*

1 Specified intended life in accordance with the New Zealand Building Code, unless defined by the designer according to a different criterion, e.g. the time at which a defined level of deterioration is likely to have developed or the time at which the structure is likely to be obsolete because the user's needs have changed.

2.2.5 SPECIAL CONCRETE: IDENTIFICATION OF LEVEL OF PRECAUTION

The ***designer*** combines the risks of ASR damage associated with the individual structure and its environment (sections 2.2.3 and 2.2.4) to determine the level of precaution required, as shown in 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.

Structure Risk Category	Environmental Risk		
	E1	E2	E3
S1	Nil	Low	Low
S2	Low	Standard	Standard
S3	Standard	Extraordinary	Extraordinary

The level of precaution defined as "Standard" provides protection equivalent to the original maximum alkali limit prescribed by the 1991 edition of TR3 and by the maximum alkali limit specified in this edition for normal concrete, 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 ***concrete producer*** identifies appropriate preventive measures from Table 4, and provides supporting calculations and/or test results or other evidence to demonstrate to the designer that the selected preventive measures will be effective.

Sometimes it will not be possible for concrete with alkali content less than 2.8 kg/m^3 to meet other specified requirements. Alkali contents of less than 2.0 kg/m^3 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 8.3.

Table 4 does not include lithium treatments as preventive measures because at the time of writing (2021) these are not widely used in New Zealand. However, they may be used in accordance with international guidelines (section 5.6) if appropriate.

Table 4: Preventive measures.

Precaution Level (From Table 3)	Measures to Prevent Deleterious Alkali-silica Reaction
Nil	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.
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 no more than 3.3 kg/m^3 Na_2O eq[1]. L3: Use a sufficient amount of effective SCM or combination of effective SCMs to minimise ASR expansion. L4: Use a project mix design that minimises ASR expansion, as demonstrated by laboratory testing[2].
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 no more than 2.8 kg/m^3 Na_2O eq[1]. N3: Use a sufficient amount of effective SCM or combination of effective SCMs to minimise ASR expansion. N4: Use a project mix design that minimises ASR expansion, as demonstrated by laboratory testing[2].
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 no more than 2.0 kg/m^3 Na_2O eq[1]. X3: Use a sufficient amount of effective SCM or combination of effective SCMs to minimise ASR expansion. X4: Use a project mix design that minimises ASR expansion, as demonstrated by laboratory testing (note 2).

Table 4 continued over page.

1 If the concrete contains SCM in sufficient quantity to effectively reduce ASR expansion (section 5.4) then options L3/N3/X3 apply. If SCM is added in smaller quantity than required to effectively reduce ASR expansion, or if the effect of an SCM on ASR is unknown then:

- Either: include the available alkali content of the SCM measured using AS 3583.12 in calculations of the alkali content of concrete (options L2/N2/X2),
- Or: measure the ASR expansion potential for the proposed combination of materials using an accelerated mortar test such as ASTM C1567 or accelerated concrete prism test such as those described in section 8.3 and evaluate the results against criteria published for that test method (options L4/N4/X4).

When calculating concrete alkali content, allowance shall be made for likely variations that will occur in the alkali content of the cementitious components and for variations that will occur in the cementitious content of the concrete.

Blended cements are to be treated as Portland cement plus SCM.

2 Test proposed combinations of mix designs and materials by accelerated mortar bar or concrete prism expansion tests to identify a project mix design that minimises ASR expansion as demonstrated by acceptance criteria based on the test method used. Testing methodologies and evaluation criteria are discussed in section 7.

3.0 ASR IN NEW ZEALAND

This section deals specifically with ASR in New Zealand. It has not been revised in this edition except for minor editorials. A more recent and detailed summary is in Sims and Poole (2017). For more general information on ASR, refer to Appendix B.

3.1 HISTORY OF RESEARCH INTO ASR IN NEW ZEALAND

In 1939, Thomas Stanton 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 (Stanton, 1939). They named this phenomenon alkali-aggregate reaction (AAR). The initial work of Stanton was quickly expanded by other workers at major US agencies such as the Bureau of Reclamation (Mielenz & Witte, 1948) 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 as some of the reactive volcanic rocks present in the USA are similar to those found in the Waikato River valley (Hutton, 1945). 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 & 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) and Opus International Consultants Ltd (Opus)) assisted by DSIR Chemistry (and its successor, 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 et al, 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). The 1991 edition of TR3 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 et al, 2000; Freitag, 1998, 2002; Freitag & St John, 1996; 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).

Ongoing work by Opus in the 2000's focussed on forensic investigations of existing buildings and civil structures showing signs of ASR damage, some of which revealed cases of ASR associated with combinations of materials and conditions not previously recognised as an ASR risk in areas (for example Freitag et al, 2011).

In 2017, CCANZ/Concrete NZ initiated research to inform this revision of TR3, including review of the concrete alkali limit. Initial testing, carried out in conjunction with a wider research programme co-ordinated by Cement Concrete and Aggregates Australia (CCAA), investigated the reactivity of known reactive materials such as Waikato River sand and Bay of Plenty andesite as well as non-reactive greywacke, and compared the results from accelerated concrete prism expansion tests with results from more established methods including petrography, rapid chemical testing and accelerated mortar bar and concrete prism testing (Appendix E, Nsiah-Baafi, 2019). Concrete NZ then extended the work to investigate the effectiveness of several SCMs in mitigating ASR expansion (Mackechnie, 2021).

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 and Freitag et al, 2011) indicates that minor ASR can occur even where low-alkali cement and pozzolan have been used and concrete alkali contents are less than 2.0 kg/m^3: visible damage may be very minor or completely absent. 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. It also indicates that ASR may be relatively common and does not necessarily cause damage. This document focusses on minimising ASR damage, not ASR itself.

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 surfaces. ASR damage could be more common and/or severe in buried concrete that is permanently immersed or buried (Freitag & Rowe, 1987; Rowe, et al, 1989; St John, 1988; St John, 1989, Freitag et al, 2011).

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.

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. Nevertheless, overall expansion of elements may cause closure of joints and damage to abutting structures, and ASR cracking may facilitate the ingress of agents that promote other deterioration mechanism and, on highly visible surfaces, may cause public concern.

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 & 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. 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% Na_2O equivalent. From 1974, NZS 3122 has allowed an alkali content of less than 0.60% Na_2O equivalent to be specified if the cement is to be used with potentially reactive aggregate. Since 2015 the alkali limit of 0.60% has been the default for cement types GP and GB. 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 most other cases have not required maintenance. As a result, there is a perception in the wider concrete 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;
- Success in managing ASR to date and loss of local expertise within the wider concrete industry has led to a lack of in-depth understanding about ASR in New Zealand and the tools available for its management;
- 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.
- Ongoing testing to assess new materials used to reduce Portland cement or dispose of waste materials.

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 standards and guidance documents such as those published by the Cement and Concrete Association of Australia/Standards Australia, the UK Concrete Society, the Building Research Establishment (BRE), ACI, ASTM, AASHTO, RILEM, the Canadian Standards Association (CSA), and Sims and Poole (2017) 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. RILEM has for many years been developing and maintaining universal guidelines to minimise the risk of ASR damage based on the contributions of contributing members from many countries. The guidelines presented in the 2003 edition of TR3 were based on an early draft of RILEM's recommendations (RILEM, 2003) and were subsequently adopted by SA HB 79:2015. This edition is based on more recent guidance such as presented in Nixon and Sims (2016) and subsequent RILEM draft recommendations.

4.0 ALKALI-AGGREGATE REACTIONS

Section 4.0 has not been revised in this edition except for minor editorials. Readers interested in specific reaction mechanisms are encouraged to seek more current information. At the time of writing, Sims and Poole (2017) represents a comprehensive summary of current international expertise on AAR and its management.

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, hornfels 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.

ASR 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

This section refers to temperate environmental exposure conditions in New Zealand and may not be relevant to other service conditions.

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.

In 2006, routine inspections on two South Island bridges with a pile bent substructure revealed significant cracking on precast prestressed piles below high tide level. Forensic investigation by Opus International Consultants revealed that ASR and Delayed Ettringite Formation (DEF, another expansive reaction within concrete), had occurred. The findings suggested that concrete temperatures above 70°C may initiate ASR in some combinations of materials (Freitag et al, 2011). Such temperatures may be reached during heat curing or in large cross sections if concrete temperature during the curing process is not controlled and monitored.

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 75-100 mm 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 (Fookes, 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. Investigations into the incidence of ASR/DEF in South Island bridges with pile bent substructures revealed some cases of combined ASR and DEF. This type of deterioration, referred to herein as ASR/DEF, requires damp or wet conditions, and may be significant for buried or immersed parts of piles and other foundation elements subject to elevated concrete temperatures during curing if concrete temperature during the curing process is not controlled and monitored. (Freitag et al, 2011).

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 (Fookes, 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 40 mm 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 50 mm depth. Bridges in the Tongariro region contained 0.00-0.01% chloride at the surface and at approximately 40 mm depth. The contributions from the chlorides in the aggregates are not known, although geochemical 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/m^3 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.

Canadian work found that exposure to sodium chloride did not change the chemistry of the pore solution at depths greater than 60-80 mm 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. Institute of Structural Engineers (1992), SA HB 79 (2015) and Saouma (2021) describe the potential effects of ASR on structural performance and methods of assessing the effects on structures.

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. There are reports of significant serviceability issues in large hydro-structures caused by ASR-induced expansion of concrete (Batista & Gomes, 2020).

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 and other deterioration mechanisms. Microcracking presents less of a risk because it is not continuous. Wide cracks present a 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 concern 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 a specified intended life of 50 years in accordance with the *New Zealand Building Code* or a 100-year specified intended life for structures designed in accordance with the Waka Kotahi NZ Transport Agency's *Bridge Manual*, and that minor ASR damage is acceptable. The designer should consider designating the concrete as special concrete if for an individual structure the specified intended life or level of acceptable damage differ from these requirements.

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. **This document assumes that unless field or test data indicate an aggregate to be non-reactive then the concrete producer (and, for special concrete, the designer) must 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. For most structures, however, the aim is to prevent significant ASR damage and only one of these controls is necessary.

Where the most suitable 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 8. 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. To date it has been the most practical method of control in New Zealand, effectively used by limiting cement alkali contents in public infrastructure projects since the 1940's.

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 1940's that found a 0.6% cement alkali limit to be an acceptable compromise between economy and risk of ASR damage (Stanton, 1940; Hill, 1996).

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 available alkali content (SCMs offer additional advantages as described in section 5.4).

The total binder content of an individual batch of concrete supplied to a project mix may differ from that specified or nominated by the producer. Normal fluctuations about the assumed figure will vary according to the conditions under which the concrete is mixed but should be less than 1% of target. 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 8.2. These variations must be accounted for.

Since the effect of concrete alkali content on ASR expansion was recognised, different countries and agencies adopted different concrete alkali limits based on their individual needs and experience with their local aggregates. Limits ranged from 2.5 to 4.5 kg/m^3. Many jurisdictions now specify a range of limits depending on the reactivity of the aggregates used and other risk factors. Some have adopted other methods of control, such as a ASTM C1778.

The 2.5 kg/m^3 limit on concrete alkali set in the 1991 edition of TR3 was based on UK guidelines at the time, and the observed behaviours of New Zealand aggregates and of concrete above ground and exposed to cycles of wetting and drying. This limit was selected because it would reliably ensure a low risk of damage, rather than be a maximum above which the risk would be higher. Laboratory testing carried out to inform the 2021 revision (ref/s) indicated that a slightly higher limit would not significantly increase the risk of damage, and consequently the limit has now been increased to 2.8 kg/m^3. Most of the concrete on which the New Zealand limit is 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 for example (i.e. these may not be the only cases):

- 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 8 describes how to ascertain the alkali content of individual concrete constituents.

5.4 SUPPLEMENTARY CEMENTITIOUS MATERIALS (SCMS)

SCMs can be added to concrete with Portland cement to reduce the cost or carbon footprint of fresh concrete, to enhance its fresh and hardened properties, and to reduce the risk of ASR. This document only considers their effectiveness in mitigating ASR damage.

SCMs considered in this document are materials such as fly ash, blast furnace slag, silica fume, amorphous silica, and pozzolans (natural or manufactured) that react hydraulically or pozzolanically and conform to relevant standards for the specific type of SCM. They may be added to the concrete as a blended cement or as separate products.

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 blast furnace 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 imported fly ashes have been used in concrete since the late 1990’s. Natural and manufactured pozzolans may become more widely used in New Zealand in future.

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 equilibrium concentrations of alkalis in pore solution, hydration products, and ASR gel can vary significantly during the first few years after the concrete is manufactured (Shehata & Thomas 2006);
- 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% Na_2O 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 and Duchesne (1993) to see the role of silica fume in terms of postponing rather than avoiding ASR. Subsequent work reported by Shehata and Thomas (2006) showed similar effects, and demonstrated the potential advantage of using binders containing more than one type of SCM. 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 8.3). Boddy, et al (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.

Pozzolans may be from natural sources or manufactured as by products from other industries. Natural pozzolans including pumicite, diatomite, and amorphous silicas from geothermal activity have been widely used in New Zealand, often for purposes other than alleviating ASR. With appropriate processing to control particle size, natural pozzolans can be effective in reducing ASR expansion. Pozzolans containing significant aluminium are particularly effective because the aluminium helps bind alkalis so that they are not available for ASR. In New Zealand many natural pozzolans have been shown to be effective in controlling ASR expansion. Manufactured pozzolans, for example materials such as ground glass produced from waste products of other industries have been used overseas. Prescriptive limits for using pozzolan to reduce ASR damage have not been developed due to the wide variety of pozzolan types and properties. (Research on New Zealand natural pozzolan recommends 25%).

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.

Fly ash is classified in AS/NZS 3582.1:2016 into three grades based on fineness, loss of ignition and strength index. It also specifies minimum requirements for the chemical composition of glassy phases (expressed as the oxides SiO_2, Al_2O_3 and Fe_2O_3, which are determined by chemical analysis). Fly ash used in Australia is required to contain at least 70% of these glass phases and fly ash used in New Zealand must contain at least 60% (SiO_2, Al_2O_3 and Fe_2O_3). The 60% limit was based on the performance of fly ashes historically produced at Huntly power station, and is consistent with AS/NZS 3582.1:2016 provisions for fly ash from a single source.

Fly ashes from sources other than Huntly power station are widely used in New Zealand, and the range offly ash sources and compositions used here in future is likely to further increase as supplies diminish. Consequently, TR3 (2021) strongly recommends that fly ash selected for mitigating ASR in New Zealand meets the AS/NZS 3582.1:2016 limit of 70% (SiO_2, Al_2O_3 and Fe_2O_3) unless testing and performance history shows the fly ash conforms with the AS/NZS 3582.1:2016 provisions for fly ash from a single source. Alternatively, the fly ash may conform with ASTM C618 requirements (see below).

Many aspects of the performance of fly ash in concrete are known to be related to its calcium content rather than by the composition of the glassy phases. Reflecting this, in 2019 ASTM C618 changed from classifying fly ash solely by (SiO_2, Al_2O_3 and Fe_2O_3) to classifying by CaO, as shown in the following table:

Table 5: ASTM classification for fly ash by chemical composition.

	(SiO_2, $Al_2O_3Fe_2O_3$)		CaO	
	Class F	**Class C**	**Class F**	**Class C**
C618-16	Min 70%	Min 50%	-	-
C618-19	Min 50%	Min 50%	Max 18.0%	>18.0

The Canadian standard CSA A3001:2018 defines three classes in terms of CaO content: Class F < 15%, Class CI 15-20%, Class CH > 20%. Prior to 2010 the boundary between classes F and CI was 8%. The intermediate class 'CI' allows for the inevitable uncertainty of how materials close to the class F/C boundary will actually behave.

Historically fly ash has been shown to reduce ASR expansion when used in sufficient quantity. Thomas (2007) showed that for controlling ASR expansion in concrete, fly ashes with CaO contents less than 20% and alkali contents less than 4% are generally more efficient than fly ashes with higher CaO and alkali contents, which require higher fly ash contents to reduce expansion. The relationship is not precise, therefore the optimum amount of an individual fly ash for reducing ASR expansion can only be determined by testing.

Calcined clays such as metakaolin are largely amorphous alumino-silicate that acts as a highly reactive pozzolan. They differ from other SCMs because these are 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 these could prevent deleterious expansion in concrete when comprising 10-15% of the total binder content by mass. BRE (2002) recommends that only metakaolin with SiO_2 content greater than 45% by mass be used to minimise ASR, and that it be used as a 10-15% cement replacement.

SCM Replacement For ASR Mitigation

Where SCM is added to concrete to protect against ASR-induced expansion, the quantity needed will depend on the type of SCM. Recommended minimum cement replacement levels needed to minimise ASR damage in New Zealand are as follows (Thomas, 2007; Mackechnie, 2021; ASTM C1778, 2020):

- Silica fume, microsilica and geothermal silica: at least 8%
- Diatomite, calcined clay, and metakaolin: at least 15%
- ASTM Class F fly ash and natural pozzolans: at least 25%
- ASTM Class C fly ash: at least 35%
- Blast-furnace slag (GGBS): at least 35%

Higher levels may be needed depending on the precaution level (as determined in section 2.2).

The level of cement replacement needed to control ASR expansion will also vary with the aggregate, the cement, the presence of other SCMs and may be determined by performance testing to optimise mix designs to minimise ASR risk associated with specific combinations of materials. The level of cement replacement must not compromise other concrete properties such as workability.

When two or more SCMs are used, the quantities required to mitigate ASR damage need to be determined by performance-based testing. Further information on the use of more than one SCM may be given in ACI and CSA publications (examples ACI 201.2R, ACI 232 series and CSA A23.1/23.2).

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 blast-furnace slag.

5.4.2 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 6.

Table 6: New Zealand, Australian and ASTM standard specifications and test methods for SCMs.

Material	Specification		Test Method for Determining Effectiveness in Reducing ASR Expansion	
Portland and Blended Cements	NZS 3122	Specification for Portland and blended cements.	AS 2350.2	Test method for determining the alkali content of Portland cement
	AS 3972	Portland and blended cements.	N/A	Test method for determining the alkali content of Portland cement
	ASTM C595	Standard specification for blended hydraulic cements.	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.).
	ASTM C1157	Standard performance specification for hydraulic cement.	ASTM C227	Test method for potential alkali reactivity of cement-aggregate combination (mortar bar method). (ASTM C1157 acceptance criteria based on 14d and 56d results for the hydraulic cement, and on 91d result for pozzolan components).
Silica fume and other amorphous silica	AS/NZS 3582: Part 3	Supplementary cementitious materials for use with Portland and blended cement: Part 3: amorphous silica.	AS 3583: Part 12	Methods of test for supplementary cementitious materials for use with Portland cement: Part 12: Available alkali. (Test takes a minimum of 4 weeks).
Fly ash	AS 3582: Part 1	Supplementary cementitious materials for use with Portland and blended cement: Part 1: Fly ash.	AS 3583: Part 12	Methods of test for supplementary cementitious materials for use with Portland cement: Part 12: Available alkali. (Test takes a minimum of 4 weeks).
	ASTM C618 &	Specification for coal fly ash and raw or calcined natural pozzolan for use as mineral admixture in concrete.	ASTM C1778	Standard test methods for sampling and testing fly ash and natural pozzolans for use as a mineral admixture in Portland cement concrete. (ASTM C618 acceptance criteria based on 14d results).

Material	Specification		Test Method for Determining Effectiveness in Reducing ASR Expansion	
Natural pozzolan	ASTM C618 & NZS 3123	Specification for coal fly ash and raw or calcined natural pozzolan for use as mineral admixture in concrete.	AASTM C1778	Standard test methods for sampling and testing fly ash and natural pozzolans for use as a mineral admixture in Portland cement concrete. (ASTM C618 acceptance criteria based on 14d results).
Blast furnace slag	AS 3582: Part 2	Supplementary cementitious materials for use with Portland and blended cement: Part 2: Slag – ground granulated iron slag.	AS 3583: Part 12	Methods of test for supplementary cementitious materials for use with Portland cement: Part 12: Available alkali. (Test takes a minimum of 4 weeks).
	ASTM C989	Specification for ground granulated blast furnace slag for use in concrete and mortars.	ASTM C1778	Standard test method for effectiveness of mineral admixtures in preventing excessive expansion of concrete due to alkali aggregate reaction. (ASTM C989 acceptance criteria based on 14d test results).
Note: These specifications provide guidance for the general use of materials but do not take precedence over the guidance and recommendations in TR3, which is intended specifically for New Zealand.				

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 designer may require a certificate giving the type and amount used, the reactive alkali content of the blended cement (section 8.2), 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 to the concrete can be determined using the methods and acceptance criteria in the relevant ASTM test methods and specifications listed in Table 5.

ASTM C1778 describes protocols for determining the amount of SCM to use, including accelerated mortar bar and concrete expansion tests. RILEM guidelines (Nixon & Sims, 2016) also include methodologies for testing combinations of materials and mix designs. Sims and Poole (2017) summarise methodologies used by various jurisdictions including RILEM. The conditions used in accelerated expansion tests on which many current guidelines and specifications are based are designed to accelerate ASR and 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 - including from deposits near Huntly, imported coals and blends of Huntly and imported coals. Tighter controls of coal blend composition have produced more consistent fly ash. The composition of fly ashes, blast furnace slags, silica fumes, amorphous silicas, and pozzolans varies significantly between sources and suppliers. Suppliers of SCMs may change their sources from time to time, therefore it should never be assumed that successive purchases are from the same source and product data must be provided for each purchase. The properties of natural products will also vary with time as different parts of a deposit are worked, so must also be monitored. For example, over the years that Whirinaki diatomaceous pumicite 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. For example, accelerated concrete prism expansion testing to ascertain the effectiveness of SCMs can take more than two years. Therefore testing to optimise the use of an SCM may be limited to major projects or assessment of new sources of aggregate or SCM.

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. Blast furnace slag, fly ash, or natural pozzolan 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 blast furnace slag or silica fume to add for minimising ASR damage 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 OTHER TREATMENTS

Chemical admixtures have been developed to mitigate the risk of ASR-related expansion. Most are are based on lithium salts but some include materials such as super-absorbent polymers, aluminium and nano-silica (Sims & Poole, 2017). Such admixtures are rarely used in new construction since SCMs are generally more economic to use when dealing with reactive aggregates.

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 blast furnace 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).

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. Based on more recent work, RILEM AAR-7.1 (Nixon & Sims, 2016) also reports that the lithium dose does not solely depend on the reactivity of the specific aggregate. It recommends testing to determine effective dosages. Thomas et al (Sims & Poole, 2017) report that the apparent effectiveness varies with the test method used, and that no consensus about appropriate accelerated tests for determining Li dose has been reached.

It is reasonable to assume the same uncertainties about determining effective dosages also apply to other admixtures claiming to reduce ASR expansion.

No specific guidance to the use of lithium treatments and other admixtures 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 (HB79, 2015), the Canadian Standards Association (CSA, 2018), ASTM C1778-20 (2020), and AASHTO (2010).

RILEM technical committee TC ARP and its sucessors TC106 (1988-2001), TC191-ARP (2001-2006), TC 219-ACS (2006-2014), TC-258-AAA (2014-2000) and TC-ASR (2020-) developed protocols and accompanying tests for assessing aggregate reactivity and the potential for aggregate combinations to produce expansive ASR in concrete (Nixon 2000a, 2000b; Sims & Nixon, 2001, 2003a, 2003b, Nixon & Sims, 2016). RILEM's tests and protocols are subject to continuing development. New Zealand is represented on these committees.

To date, RILEM has developed a total of 13 methodologies, including processes for determining the potential reactivity of aggregates (AAR-0 to -5), identifying combinations of aggregates and binders to reduce AAR expansion in concrete (AAR-8 to 13), minimising the risk of deleterious reactions in new concrete structures (AAR-7), and management of AAR in existing structures (AAR-6). These are summarised in the table below. At the time of writing, methods AAR-0 to 7 had been published in RILEM State of the art reports 12, 17 & 31. A summary of these RILEM protocols is given in Appendix D.

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.

Much of the 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 from site investigations by concrete materials specialists in areas where potentially reactive aggregates are available, including North Island structures and a limited selection of South 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. Instead, it could mean that the aggregate combination is 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.

This document is based on preventing ASR expansion that causes damage, not on preventing ASR altogether. Therefore aggregates are classified by the likelihood that they may react to an extent that causes significant damage. An aggregate classified as 'reactive' or 'potentially reactive' will not necessarily produce significant ASR damage. Similarly, an aggregate classified as non-reactive may undergo minor reaction under some circumstances but the limited extent of the reaction is highly unlikely to cause significant expansion and damage.

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). Petrography is the most convenient method of assessing an aggregate's potential reactivity. It involves the examination of a very small sample of the material visually and using specialist microscope techniques, therefore the chance of a minor but reactive component being missed from the sample or overlooked in the examination is relatively high. In addition, the assessment is relatively subjective. Consequently, many jurisdictions consider it less reliable than laboratory testing or field experience. The criteria listed below are in order of increasing reliability.

If the aggregate is deemed not potentially reactive, no further testing is needed. If petrographic examination finds it to be 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 7, or its likely reactivity may be assessed from its performance in existing structures (section 7.4). If available, data from concrete testing or 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 accelerated mortar bar expansion) test results at 14 days show it to be non-reactive; or
- ASTM C1293 (or equivalent accelerated concrete prism expansion) 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 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 7.1 to 7.3.

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 6.3.1 and 6.3.2, using recognised methods of examination that are considered appropriate by the geologist. ASTM C295 provides a method for the petrographic examination of aggregates for concrete and RILEM method AAR-1 (Sims & Nixon, 2016) is a petrographic method for identifying rock types and minerals that might be alkali reactive.

6.3 REACTIVITY OF NEW ZEALAND AGGREGATES

6.3.1 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 7 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 or in concrete with alkali contents higher than 2.8 kg/m^3. 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 6.3.2.

“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 8.6.

Table 7: New Zealand aggregates believed to be non-expansionary from field experience (with low alkali cement) or laboratory testing.

Greywacke	Limestone	Rhyolitic pumice
Basalt <50% SiO_2	Schist	Perlite
Phonolite	Quartz Sands	Vermiculite
Granite	Quartz-feldspar sands	

Greywacke and Argillite

In New Zealand, aggregates derived from greywacke-suite rocks are used extensively, constituting approximately 80% of the aggregates used in concrete. New Zealand greywackes 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 and aggregate properties. 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 mortar bar 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. Basalt containing volcanic glass may also be reactive depending on the silica content of the glass, rhyolitic glass (more than 65% silica) being potentially highly reactive (Katayama, St John & Futagawa, 1989).

Literature review, and testing of basalts from seven countries including three from New Zealand, suggest devitrified glass and swelling clays (resulting from the alteration of volcanic glass in basalts) can also contribute to basalt reactivity and that particle size or particle size distribution of the reactive material is a key factor in determining the potential reactivity of an aggregate (Medeiros et al, 2021). Accelerated concrete prism tests by the same authors produced greater expansion with sands containing potentially reactive basalt than with coarse aggregate fractions of the same material.

Petrographic examination must be supplemented by chemical analysis to determine the silica content of the aggregate and the silica content of volcanic glass or devitrified glass, and hence ascertain the need for further investigation of its reactivity.

The nepheline basanites 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 8.6).

Phonolite

There are no known cases of ASR involving phonolite, which was 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. Petrographic examination of concrete samples from several Southland and structures, plus accelerated mortar bar and concrete expansion tests on samples from two sources indicate that strained quartz, microcrystalline quartz, and volcanic and metavolcanic components of Southland gravels are potentially reactive but are unlikely to cause significant ASR damage except in exceptional circumstances, such as where high early age concrete temperatures and subsequent wet exposure conditions in service result in ASR/DEF.

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. New Zealand 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.

6.3.2 POTENTIALLY REACTIVE NATURAL AGGREGATES

Laboratory testing and field observations indicate that the most common 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. Potentially reactive volcanic glasses may also be present in some basic rocks. Natural aggregates of mixed composition, such as alluvial or beach deposits, that contain potentially reactive components should be considered potentially reactive particularly if the potentially reactive material is present in sand-size particles.

The aggregates listed in Table 8 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 6.7.

"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 8.6.

Table 8: New Zealand aggregates or minerals known to be potentially reactive either from field experience or laboratory testing.

Basalt (including phonolite and trachyte) >50% SiO_2	Rhyolite	Cristobalite
Cristobalite	Volcanic glass	Tridymite
Dacite	Quartzite	
Amorphous and cryptocrystalline silicas including opal and chalcedony		

Following the observation of deterioration in precast concrete piles on two Southland Bridges, inspection of 51 South Island bridges and a review of the potential reactivity of South Island aggregates (Skinner, 2009) found that:

- Dunedin Volcanic Group rocks containing more than 50% silica may be alkali-reactive, including alkali-rich phonolites and trachytes with silica contents over 55% and high levels of sodium and potassium.
- ASR appears to have caused visible damage to precast concrete containing alluvial aggregates from Southland and Nelson that contain quartzite, meta-quartzite, acid meta-volcanics, andesite, tuff-like material, gneissic material and siltstones of a particular composition. Gravels derived from the Dun Mountain-Maitai and Murihiku terranes and found in the catchments of the Mataura, Oreti, Aparima, and Waiau Rivers in Southland and the Waimea River in Nelson may include such materials. Non-basaltic volcanic and intrusive components of the Brook St terrane, found in the Aparima, Waiau, and Waimea River catchments may also be reactive.
- Laboratory tests suggest that Southland alluvial aggregates (specifically those from Oreti Beach) 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, (including phonolite and trachyte)

Where the silica content of the bulk composition exceeds 50% (Katayama, St John & Futagawa, 1989), or where glass present in the basalt contains more than 65% silica, basalt may be potentially reactive and should be treated as if it is an andesite in terms of reactivity.

Dunedin Volcanic Group rocks containing more than 50% silica may be alkali-reactive, including alkali-rich phonolites and trachytes with silica contents over 55% and high levels of sodium and potassium. To date, no cases of ASR have been reported in concretes containing these materials.

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 alluvial aggregates with microcrystalline quartz and strained quartz have exhibited significant deterioration from ASR/DEF (Freitag et al, 2011). However, it is considered that the extent of the deterioration resulted from curing temperatures above 70 0C, and the saturated immersed concrete, two conditions which need to be present for DEF. Nevertheless these two forms of quartz are considered potentially reaction (Nixon & Sims, 2016).

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 concretes displaying ASR/DEF (Freitag et al, 2011).

Testing quartzite using ASTM C289 (see section 7.1) 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.

Aggregates of mixed composition (alluvial and beach sources)

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.

ASR appears to have caused visible damage to precast concrete containing alluvial aggregates from Southland and Nelson that contain quartzite, meta-quartzite, acid meta-volcanics, andesite, tuff-like material, gneissic material and siltstones of a particular composition. Gravels derived from the Dun Mountain-Maitai and Murihiku terranes and found in the catchments of the Mataura, Oreti, Aparima, and Waiau Rivers in Southland and the Waimea River in Nelson may include such materials. Non-basaltic volcanic and intrusive components of the Brook St terrane, found in the Aparima, Waiau, and Waimea River catchments may also be reactive.

Petrographic examination of concrete samples from several Southland and Nelson structures, plus accelerated mortar bar and concrete expansion tests on samples from two sources indicate that strained quartz, microcrystalline quartz, and volcanic and metavolcanic components of Southland gravels are potentially reactive but are unlikely to cause significant ASR damage except in exceptional circumstances, such as where high early age concrete temperatures and subsequent wet exposure conditions in service result in ASR/DEF, particularly if the concrete contains sufficient alkali.

A significant proportion 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.

6.4 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.

6.5 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) as defined by CCANZ TR14, which is predominantly waste concrete, masonry and asphalt shall be classified as potentially reactive in all situations as it may be contaminated with reactive material (CCANZ, 2011).

6.6 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. Nevertheless, finely ground sand from crushed glass 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%. Testing of the proposed concrete mix should be undertaken to assess the risk of deleterious ASR.

The limiting fraction of fine aggregate is controlled by minimizing the risk of ASR and suppressing the alkali reactivity by using SCMs. The general principles for minimizing the risk of ASR on a structure are outlined in section 2.1. Guidance on the use of SCMs for suppressing ASR is given in sections 5.4 and 8.3.

Powdered glass manufactured from recycled glass waste has been used overseas as an SCM to reduce ASR expansion. Provisions for its use are included in CSA 3001–18 (2018).

Recycled glass in any form may release alkali and should not be used in conjunction with potentially reactive aggregates (see section 8.6).

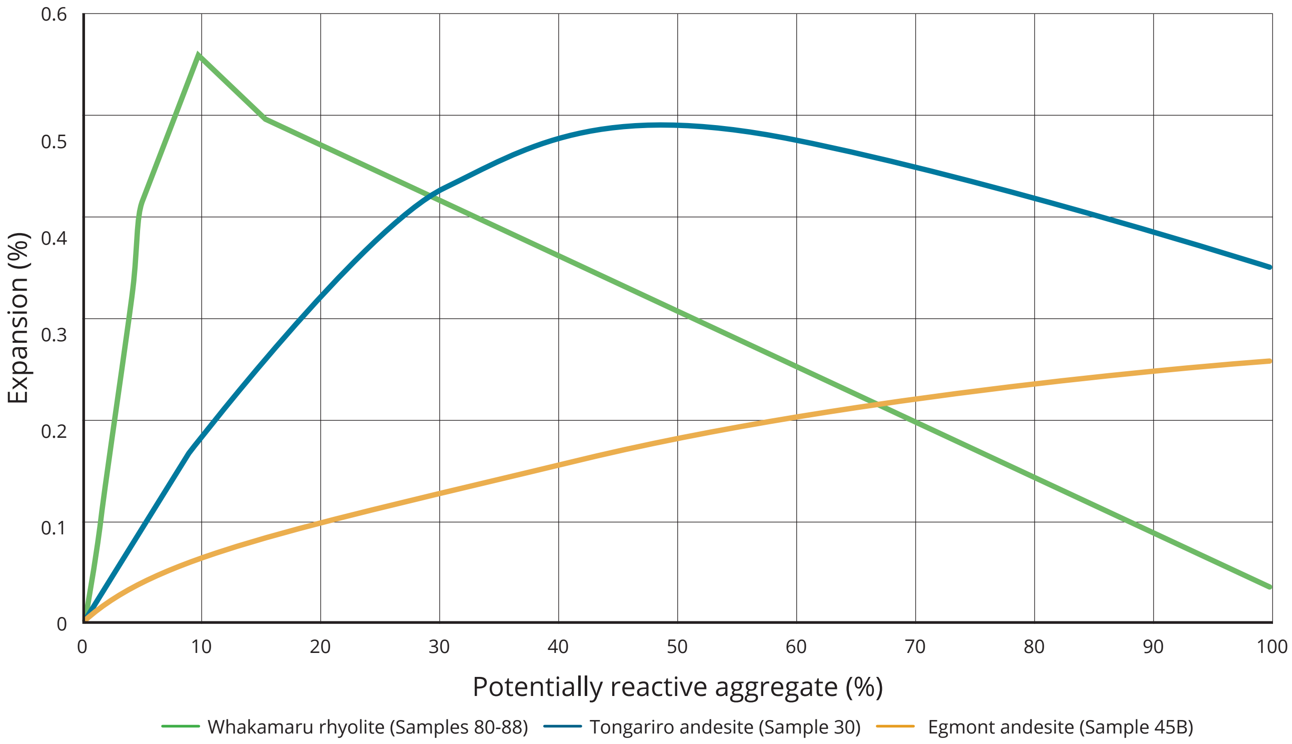
6.7 AGGREGATE COMBINATIONS AND PESSIMUM PROPORTION

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

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

The silica minerals opal, chalcedony, cristobalite and tridymite exhibit marked pessimum 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 pessimum proportions varying between 15% and 40% because of the presence of some of these minerals. In contrast, volcanic glass shows little pessimum proportion and our andesites, which are often highly glassy in texture, exhibit a limited range of pessimum proportion. Egmont andesite usually has no pessimum proportion while andesites from other regions appear to have broad pessimums. Typical examples of expansion curves for some of these materials are given below.

Figure 3: Typical pessimum proportion curves for three New Zealand rock types (Expansion at 12 months at 1.5% Na_2O 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.

7.0 TEST METHODS

There are two reasons for testing; to assess the potential reactivity of an aggregate or aggregate combination, or to assess the likely expansion of a proposed concrete mix (including the effectiveness of a particular SCM in reducing expansion).

If petrographic examination finds that an aggregate is potentially reactive then further testing may be required to provide more definitive evidence of the reactivity of the aggregate. Expansion from alkali silica reaction can take five to twenty years to be seen under ambient conditions, which makes accelerated testing the only practical means of assessing potential reactivity of aggregates. These assessment techniques include tests on the aggregates themselves, tests on mortar bars and tests on concrete. Longer-term concrete tests are generally more reliable than chemical or mortar bar expansion tests, which may produce false positive results in some cases. False negative results from chemical and mortar bar expansion tests have been reported for some types of 'slowly reactive' aggregates overseas, but such inconsistencies have not been observed with New Zealand aggregates.

7.1 CHEMICAL TEST ASTM C289 (WITHDRAWN 2016)

ASTM C289, known as the "quick chemical test" or the "Mielenz test" (Mielenz & 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 F. 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). The test method is longer supported by ASTM and may become harder to procure in future.

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

7.2 MORTAR BAR EXPANSION 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.

Mortar bar expansion test ASTM C227 (Withdrawn in 2018)

NZS 3111:1986 includes a test method based on ASTM C227, but ASTM C227 conditions for storing the test specimens were modified since NZS 3111:1986 was published, so the most recent version of ASTM C227 should be followed rather than the NZS 3111:1986 method if this type of mortar test is to be used. The test requires measurement of the length change of mortar bar specimens stored over water at 38ºC for six months, but often measurements have to be carried out for two years to obtain reliable results. Significant amounts of alkali can be leached from the specimens during the test if precautions are not taken.

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 workability, and test conditions must not be varied from those previously used. Many New Zealand materials used for concrete aggregates have been tested using ASTM C227 (St John, 1988).

Rapid mortar bar expansion tests (e.g. ASTM C1260, AS 1141.60.1, RILEM AAR-2)

The rapid mortar bar expansion test was 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 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 same approach is also used to measure the ability of some SCMs to reduce ASR expansion (e.g. ASTM C1567) .

Procedures based on this approach have been adopted by many organisations and .guidelines for acceptance of aggregates on the basis of this approach have been widely published (e.g. SA HB 79:2015, RILEM AAR-0, ASTM C1778). Criteria for interpreting the test results vary slightly between jurisdictions depending on the behaviour of local aggregates. Typically an expansion less than 0.10% at 14 days is considered to indicate the aggregate is non-reactive. AS 1141.60.1 classifies aggregate reactivity based on mortar bar expansions at 10 days and 21 days rather than at 14 days to distinguish between non-reactive, slowly-reactive, and reactive aggregates, and also distinguishes between crushed materials tested in a standard grading and natural fine aggregates tested 'as supplied'.

7.3 CONCRETE EXPANSION TESTS

Concrete prism tests are used as performance measures of the alkali reactivity of aggregates where 75x75x280 mm concrete prisms are stored over water for longer periods than the rapid mortar bar test. The advantages of this method are that it can be used to assses aggregate reactivity and the potential expansion of individual concrete mixes, and the standard technique uses a test temperature of 38°C, which is within the maximum credible limit of ambient conditions at 38°C and less likely to induce reactions that would not occur at normal ambient exposure conditions. The same test principles may also be used to assess the potential for expansion related to specific binder combinations, specific project mix designs, and/or exposure to external sources of alkalis (see appendix D for list of RILEM tests as at 2021).

RILEM AAR-3.1, ASTM C1293, AS 1141.60-2014

These concrete prism tests allow aggregates to be assessed for ASR-related expansion using a standard concrete mix design and are conducted at 38°C for a period of 12-24 months. Concrete alkali contents of 5.25 kg/m^3 are used either using a high alkali cement or more commonly by adding additional sodium hydroxide into the mixing water. Expansion limits indicating potentially reactive aggregate are either 0.03% (RILEM) or 0.04% (ASTM) after 12 months exposure to 38°C. Slowly reactive aggregates may require longer testing over 24 months to establish potential reactivity. Results from ASTM C1293 may be used to classify the relative reactivity of aggregate using guidance given in ASTM C1778.

RILEM AAR-3.2

This test is similar to RILEM AAR-3.1 but varies the concrete alkali content to determine alkali-reactive thresholds for a particular aggregate combination. Alkali levels in the concrete are varied from 2.50-5.25 kg/m^3 Na_2O-eq. and expansion is monitored for a period of at least 12 months but preferably for 24 months (reactive expansion is assumed to be values exceeding 0.03%).

RILEM AAR-4.1

RILEM AAR-4.1 is similar to RILEM AAR-3.1 but is carried out at 60°C and expansions are measured for a period of 15 weeks to eight months. Potential reactivity are expansion of greater than 0.03% after 15 weeks or after 8 months for slowly reactive aggregates indicates potential reactivity. Research recently undertaken on behalf of Concrete New Zealand showed a good correlation between the results found for RILEM AAR-3.1 and AAR-4.1, (see Appendix F).

Concrete testing has the following limitations:

- Concrete prism tests use extra alkali in the mix water that will have a subtle effect on the microstructure of concrete.
- Concrete prisms relatively small in size (e.g. 75x75x280 mm) may allow some leaching of alkalis during testing; use of larger prisms is sometimes recommended to reduce the effects of leaching. Some test methods provide a method for evaluating the amount of alkali leached during storage, and others use larger prisms.
- Test results from different methods are not directly comparable unless the mix designs (eg water to binder ratio, alkali content) and storage conditions are the same.
- Criteria for evaluating test results vary between test methods and are based on limited comparisons of field and laboratory results. In addition, repeatability and reproducibility of the test may not be high. Therefore care should be taken when using test results for acceptance or rejection of materials and mix designs.
- If at the end of the test period the specimens are still expanding but total expansion is less than the value considered significant then continuing the test after the standard test period will ascertain whether the materials are 'slowly reactive' and thus likely to cause significant damage.
- Other mechanisms may cause or contribute to concrete expansion during the test. If the test results indicate deleterious expansion at the end of the test period, the test specimens should be subject to a petrographic examination to ascertain whether the expansion was caused by ASR.

7.4 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.

8.0 ESTIMATION OF ALKALI CONTRIBUTED BY CONCRETE CONSTITUENTS

8.1 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 and may be tested in accordance with AS 3583.12. "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 based on solubility in saturated calcium hydroxide solution or on a total alkali determination are used to determine the contribution of alkali from cementitious materials to the concrete mix .

Alkalis sourced from aggregate are not included in the above terminology (see section 8.6).

8.2 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 the Portland cement product contains a mineral addition, any acid soluble alkali from the mineral addition is included. Methods of alkali analysis are given in ASTM C114 and BS-EN196-21 (National Annex NA) and AS 2350.2.

The alkali content of cement is expressed as its sodium oxide equivalent (Na_2Oeq), which combines its sodium and potassium oxide contents according to the formula:

$$\mathbf{Na_2O\ equivalent = \%\ Na_2O + (0.658\ x\ \%\ K2O)}$$

Alkali contents of Type GP cement now manufactured in New Zealand range from 0.4% to 0.6% Na_2O equivalent. NZS 3122 limits the maximum total alkali content of 0.60% Na_2O 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%.

Alkali contents of Portland cement are reported as total alkalis (Na_2O-eq) even though it is known that the actively soluble component is typically about 85% of this value (Oberholster 2009). The remaining 15% of alkalis are insoluble with a component coming from non-clinker sources such as clays found in gypsum. Given that the difference is not large for Portland cement and allowable limits for alkali-reactive aggregates were developed for total alkalis, the total alkali content is traditionally used in calculations.

8.3 ALKALI IN SCMS

Blended cements and SCMs may contain significant alkalis and the difference between total and active alkalis may be quite appreciable. Alkali contents in binders such as fly ash and slag must be calculated in terms of available alkalis, since more than half may be insoluble and therefore not involved in any ASR expansion. Many fly ash sources may have total alkali contents above 1.0% Na_2O-eq, but active alkali levels are generally lower than for low alkali cement (i.e. below 0.6%).

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

(a) The UK Concrete Society (1999) and BRE (2003) 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 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% Na_2Oeq and the acid soluble alkali content of fly ash not exceed 5.0% Na_2Oeq. 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.

(b) 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 metakaolin, and does not require its alkali content to be included in the concrete alkali content.

(c) In Canadian Standard CSA A23.2-00 (2009) 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.

(d) Cement and Concrete Association of Australia (2015) considers that 50% of the total alkali in blastfurnace slag, 20% of that in fly ash and 100% in silica fume contribute to ASR. 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 low calcium fly ash and 65% for blastfurnace slag, unless proposed mixes are to be assessed by accelerated concrete testing.

(e) 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 and concrete prism expansion tests rather than alkali contents. ASTM C1778 has an option for limiting concrete alkali content by considering alkalis from Portland cement only.

(f) CSA also acknowledge alkalis released from powdered glass. For example, CSA 3001 categorises ground glass according to its alkali content.

The change in emphasis from reactive alkali content to total alkali content in many jurisdictions 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.

Examples of alkali contents of other SCMs are given in Table 9. This table 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 9: Examples of alkali contents of SCMs (data from 2003 edition).

Type	Product	Total alkali content* (% Na_2Oeq)
Silica fume	Typical Range	0.44 - 3.9%
Geothermal silica	Product A	0.46
Fly ash	Product C, sample 1	0.9
	Product C, sample 2	1.9
	Product D	0.6
	Product E	2.5
	Product F	3.2
Metakaolin	Product G	0.68
Blastfurnace slag	Product H	0.51
	Product I	0.57
	Product J	1.12
	Product K	0.33

* *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.*

8.4 ALKALI IN MIX WATER

When the 1991 edition of this document was written, the alkalis in municipal drinking 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/m^3 of cement at a water/cement ratio of 0.5 will contribute 0.084 kg/m^3 of reactive alkali (expressed as Na_2Oeq) to the concrete. Thus the contribution of reactive alkalis to concrete from potable waters in New Zealand is not significant and can usually be ignored.

Information on the quality of New Zealand drinking water supplies is compiled for the Ministry of Health by the Institute of Environmental Science and Research Ltd (ESR) and held in a national database. Information on individual drinking water supplies is available from the water supplier (usually the local authority). Information on natural water sources (groundwater, lake water, river water) that may be used for concrete in the absence of a networked water supply is available from regional councils. At the time of publication (2021) this is available from Land, Air, Water Aotearoa (LAWA).

Sodium, potassium, and chloride analyses are not required for managing drinking water quality, 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 agencies may collect data other than those required for public health purposes, and may be able to provide this information on request. For example, regional councils may use chloride analyses as an indicator of contamination by seawater (which also contains sodium and potassium in known proportions) or by fertilisers or wastewater (where sodium and potassium may be present in different proportions to seawater).

As the composition of drinking water does not vary greatly with time, historical water analyses are generally acceptable unless there has been a major change in the supply (such as its source) since the reported 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.

8.4.1 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 Na_2Oeq.

Analysis by CCANZ of nine samples of recycled wash water from one ready-mix concrete plant in 2003 gave a maximum sodium equivalent of 93 mg/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.

8.5 ALKALI FROM CHEMICAL ADMIXTURES AND ADDITIVES

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 10.

Table 10: Alkali contents of concrete admixtures (data from 2003 edition).

Admixture type[1]	Na_2O (%)[2]
Water reducing	< 0.1 – 5.0
Air entraining	< 0.2 – 0.5
Super-plasticiser	1.8 – 5.0
High early strength super-plasticiser	~ 11.0
Set accelerating	< 0.1 – 1.8
Set retarding	~ 5.3
Pump aid	< 0.1
Shrinkage reducing	< 0.1

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 K_2O so are not alkali equivalents. Sodium is often the prevalent alkali.

From Table 10, contributions from this superplasticiser or water reducer added at the rate of 1% (by cement weight) to concrete containing 400 kg/m^3 of cement could result in up to 0.2 kg/m^3 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 and other additives also needs to be considered. Again, this information should be obtained from the supplier of the material and included in the concrete alkali content.

8.6 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 observed in New Zealand aggregates strongly decreased in the above order (Goguel, 1995; Goguel, 1996; Goguel & Milestone, 1997); other authors report slightly different rankings (Menendez et al 2021).

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).

Not all alkalis within an aggregate 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. Menendez et al (2021) report that the susceptibility of an aggregate to release alkalis and the rate at which it does so is not only related to its alkali content, but also to its mineral composition, the crystallinity and grain size of individual minerals and rock particles, the aggregate particle size, the presence of microcracks and other planes of weakness within mineral grains and rock particles, and the overall rock texture, as well as the pore structure of the cementitious binder, the pore solution chemistry, and the temperature of exposure. Although RILEM is developing a test method to measure the amount of alkalis an aggregate may release in concrete (RILEM AAR-8), the significance of the test results to the overall risk of ASR expansion in concrete remains uncertain. In the absence of agreed guidelines for interpreting AAR-8 results, local experience of aggregates' alkali release behaviour should take precedence.

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 alkaline 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 New Zealand concrete was not recognised until relatively 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 Na_2Oeq 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 EN 1744–1 (2009) 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, 1000 kg of sand with a chloride content of 0.1% will contribute 0.76 kg/m^3 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.

8.6.1 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. Note that recycled glass aggregates may also release alkalis and should not be used with reactive aggregates.

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

- 0.2 kg Na_2O 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.

Nevertheless, 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

Accuracy	A statistical term that estimates the error in the absolute value of an analytical result: "how close is the result to the truth?"
Acid rocks	Volcanic rock types which contain more than 66% SiO_2. Includes rhyolite, pumice, obsidian and ignimbrite.
Acid soluble alkali	The alkali extracted from Portland and blended cements and from SCMs by dissolution in strong acid.
Additive	A material, normally solid but excluding cementitious material and aggregate, added to concrete during batching to modify concrete properties.
Admixture	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).
Alkali aggregate reaction (AAR)	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.
Alkali carbonate reaction (ACR)	Reaction between alkalis in concrete pore solutions and silica minerals in carbonate rocks such as limestone and dolomite.
Alkali content	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.
Alkali equivalent	Total combined sodium and potassium oxides in cement expressed as weight percent, based on the formula: $Na_2Oeq = \% Na_2O + (0.658 \times \%K_2O)$ where 0.658 is the molecular weight ratio of Na_2O to K_2O (see alkalis).
Alkali silica gel	The gel that forms in concrete upon reaction between the alkalis in the pore solution and 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_2O and K_2O in cement analyses).
Available alkali	Alkali extracted from a material by calcium hydroxide solution as described by ASTM C311 and AS 3583.12.
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 the designer and supplier of the material in question.
Blastfurnace slag	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 ([g]gbs).
Blended cement (type GB)	A hydraulic cement complying with NZS 3122:2009 that contains Portland cement plus 10-35% fly ash or pozzolan, 10-75% ground granulated blast furnace slag, or up to 10% amorphous silica. May be produced by intergrinding or blending the cement and SCM, which is referred to as type GB cement.
Cementitious material	Hydraulic cement and also any admixture capable of forming cement hydrates.

Chalcedony	A cryptocrystalline form of quartz found in flints and chert that can be reactive. Not common in New Zealand aggregates.
Chemical admixture	A material, which is normally liquid, that is added to fresh concrete to modify or control fresh and/or hardened properties of concrete.
Cristobalite	A silica mineral that is a minor component in basalt, andesite, dacite and rhyolite. It is reactive with alkalis.
Delayed Ettringite Formation (DEF)	Internal sulphate attack on concrete resulting from high curing temperatures to form Ettringite, causing volumetric expansion around the aggregate and resultant cracking and spalling.
Deleterious aggregate	A category of aggregate reactivity determined by ASTM C289 or C1260.
Diatomite	Alluvial deposit of opaline silica derived from the relics of diatoms that settled on sea or lake beds.
Fly ash	Fine, silica-rich ash extracted from the flue gases of a boiler fired with pulverised coal.
High-alkali cement	Cement in which the alkali equivalent exceeds 0.60% Na_2O equivalent.
Hydraulic cement	Cement that hardens by chemical interaction with water and is capable of doing so under water.
Innocuous aggregate	A category of aggregate reactivity determined by ASTM C289.
Intermediate rocks	Volcanic rocks in which the SiO_2 content lies between 52% and 66%. Includes andesite, phonolite and dacite.
Low-alkali cement	Cement in which the alkali equivalent does not exceed 0.60% Na_2O equivalent.
Microsilica	Generic term that refers to extremely finely divided powders of almost pure, often amorphous, silica. Includes silica fume and natural silicas.
Mineral admixture	Term has been superseded by 'supplementary cementitious material'.
Non-reactive aggregate	Aggregate shown by both testing and field experience not to be reactive with cement alkalis.
Opal	A highly reactive form of amorphous silica present in some rocks. Rare in New Zealand aggregates.
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 as in NZS 3122:2009 type GB.
Portland cement (type GP)	Hydraulic cement with a minimum Portland cement content of 90% in accordance with NZS 3122.

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. Included in NZS 3122 type GB if pozzolan content is 10-35%.
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 or siliceous and aluminous 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, volcanic ash or pyroclastic deposits and geothermal silica. The term 'natural pozzolan' is used to describe pozzolanic materials form natural sources, usually volcanic or sedimentary. Natural pozzolans are further divided into class N and class C (heat treated materials such as metakaolin, calcined clay, and calcined shale).
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 accelerated mortar bar expansion 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. Available alkali is the measurable estimate.
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.
Relative humidity	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.
Salt	In this document refers to sodium chloride.
Silica (SiO_2)	The chemical species that reacts with alkalis. Silica minerals such as opal, chalcedony, cristobalite and tridymite contain SiO_2 in a reactive form. Alumino-silicate glasses such as volcanic and commercial glasses may contain SiO_2 in a form that may react with alkalis under certain circumstances. Many other minerals contain SiO_2 in a form that does not react with alkalis.
Silica fume	Very fine amorphous silica, a by-product of the reduction of quartz in the manufacture of silicon and ferro-silicon.

Sodium oxide equivalent	See Alkali equivalent.
Strong acids	In the context of cement analysis usually refers to either nitric or hydrochloric acids unless otherwise specified.
Supplementary cementitious material (SCM)	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 pozzolans.
Total alkali	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.
Tridymite	A silica mineral that is a minor component in basalt, andesite, dacite and rhyolite, and is reactive with alkalis.
Volcanic glass	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, and can be found in rocks of basaltic composition.
Water soluble alkali	Alkali extracted by water.

APPENDIX B: CHEMISTRY OF THE ALKALI SILICA REACTION

This section has not been revised in this edition except for minor editorials. Readers interested in specific reaction mechanisms are encouraged to seek more current information. At the time of writing (2021) Sims and Poole (2017) represents a comprehensive summary of current international expertise on AAR and its management.

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 Na_2SO_4 and K_2SO_4 or as the mixed salt $(Na,K)_2SO_4$. 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 sulphoaluminate), 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^{0}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. Quillan (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 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 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 **alkali content of a concrete mix** is calculated as follows:

$$A = Ac + B + H + W + D + R$$

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 = available alkali content of SCM (section 8.3)

H = reactive alkali contribution made by sodium chloride contamination of both the fine and coarse aggregate (section 8.6)

W = total alkali contribution made by the mixing water (section 8.4)

D = total alkali contribution made by chemical admixtures and pigments (8.5)

R = total alkali contribution made by the aggregate (section 8.6)

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/m^3

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

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 (Na_2O) calculated from:

$$\%\ Na_2O \text{ equivalent} = \%\ Na_2O + 0.658 \times \%\ K_2O$$

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 available alkali contributed by supplementary cementitious materials shall be calculated from:

$$B = \frac{E \times f}{100}$$

where B = average available alkali content contributed by the SCM

E = the SCM content of the concrete in kg/m^3

f = available alkali content of SCM

The method used in determining the available alkali content of SCM shall comply with the methods specified in the relevant standard specification for the SCM(s). Note that for any one SCM where the replacement level exceeds the limits given in 5.4.1, ASR expansion is suppressed and alkali content is not required. Where lower replacement levels of SCMs are used, the available alkali content is determined by regular testing in accordance with AS 3583.12 or other relevant method.

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/m^3 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/m^3

NC = chloride ion content of the coarse aggregate as a percentage by mass of dry aggregates

MC = coarse aggregate content in kg/m^3

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.

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

$$W = \frac{(Na \times 1.35 + 0.658 \times K \times 1.20) \times C \times W/C}{100{,}000{,}000}$$

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 additives** shall be calculated from:

$$D = \frac{y}{100} \times \frac{C}{100} \times Z, \text{ or}$$

$$D = \frac{y}{100} \times \frac{C}{100} \times \frac{Z_1 d}{100}$$

where D = alkali contribution made to the concrete by chemical admixtures and/or additives 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 additives

Z = weight of solid admixture or additive added to the concrete per 100 kg of cement

Z_1 = volume of liquid admixture or additive added to the per 100 kg of cement

d = density of liquid admixture oradditive

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_2O equivalent

g = the weight of recycled aggregate kg/m^3 of concrete

APPENDIX D: SUMMARY OF RILEM GUIDES AND TESTING PROTOCOLS

Publication details are available on RILEM website.

General	Topic	Specific methods	Topic
AAR-0	Guide to the use of RILEM methods		
AAR-1	Detection of potential alkali reactivity	AAR-1.1 AAR-1.2	Petrographic examination of aggregate Petrographic atlas
AAR-2	Detection of potential alkali reactivity Accelerated mortar bar test method (80°C)	AAR-2.1 AAR-2.2	Potential reactivity of an aggregate (25x25x285 mm mortar bars) Potential reactivity of an aggregate (40x40x160 mm mortar bars)
AAR-3	Detection of potential alkali reactivity 38°C concrete prism test method	AAR-3.1 AAR-3.2	Potential reactivity of an aggregate or aggregate combination Determining the critical alkali threshold for an aggregate or aggregate combination
AAR-4	Detection of potential alkali reactivity - 60°C concrete prism test method	AAR-4.1	Potential reactivity of an aggregate or aggregate combination
AAR-5	Detection of potential alkali reactivity – screening test for carbonate aggregates. Accelerated mortar bar test method (80°C)	AAR-5	Potential reactivity of carbonate aggregate (40x40x160 mm mortar bars)
AAR-6	Diagnosis and prognosis of AAR affected structures (both methods were combined in 2021)	AAR-6.1 AAR-6.2	Diagnosis Prognosis, including accelerated expansion tests and modelling of structural effects
AAR-7	International specification to minimise damage from alkali reactions in concrete	AAR-7.1 AAR-7.2 AAR-7.3	Alkali silica reaction Alkali carbonate reaction Concrete dams and other hydro structures
AAR-8	Alkali release from aggregates	AAR-8	Chemical test to determine the amount of releasable sodium and potassium ions in an aggregate
AAR-9	[Was subsumed into AAR-6]	-	-
AAR-10	Binder combinations for non-reactive mix designs - 38°C concrete prism test method	AAR-10.1 AAR-10.2	Determining minimum SCM content or maximum cementitious alkali content for a specific aggregate or aggregate combination Determining maximum alkali content (from components other than SCM) for a worst-case local aggregate combination

General	Topic	Specific methods	Topic
AAR-11	Binder combinations for non-reactive mix designs - 60°C concrete prism test method	AAR-11.1	Determining minimum SCM content or maximum cementitious alkali content for a specific aggregate or aggregate combination
		AAR-11.2	Determining maximum alkali content (from components other than SCM) for a worst-case local aggregate combination
		AAR-11.3	Determining the risk of ASR expansion associated with a specific concrete mix design (eg a proposed project mix)
AAR-12	Binder combinations for non-reactive mix designs - 60°C concrete prism test method with external alkali supply (cyclic wetting/drying and exposure to NaCl).	AAR-12.1	Determining minimum SCM content or maximum cementitious alkali content for a specific aggregate or aggregate combination
		AAR-12.2	Determining maximum alkali content (from components other than SCM) for a worst-case local aggregate combination
		AAR-12.3	Determining the risk of ASR expansion associated with a specific concrete mix design (e.g. a proposed project mix)
AAR-13	Alkali wrapping as an alternative storage condition for concrete prism expansion tests	AAR-13	Method for wrapping concrete prisms in paper containing an alkaline solution to reduce alkali leaching in concrete prism test methods AAR-3, AAR-4, and AAR-10 to 12.

APPENDIX E: RESULTS OF TESTING AGGREGATES USING CONCRETE PRISM TESTING (CPT)

INTRODUCTION

Concrete prism testing (CPT) was undertaken from 2017 to 2020 to investigate the reliability of two techniques using known reactive and unreactive aggregate combinations. This research was also undertaken to inform the 2021 revision of this document. Laboratory testing consisted of CPT-38 and CPT-60 where concrete prisms are exposed to either 38°C or 60°C temperature and expansion measured in accordance with AS 1141.60.2:2014. CPT-38 testing has been widely adopted overseas but requires moderately long periods of monitoring, typically 12-18 months. CPT-60 is a more rapid technique that typically requires 4-8 months to confirm the potential reactivity of aggregate combinations in concrete.

This appendix presents results from testing carried out to evaluate whether CPT-38 and CPT-60 results accurately reflect the reactivity of four New Zealand aggregates from in-situ or historical testing. Testing was also undertaken to assess whether the results of the CPT-60 test can be used instead of the slower CPT-38 test. For more details of the wider testing programme used to support this revision of TR3 see Freitag and Mackechnie (2018).

EXPERIMENTAL

Four concrete mixes were assessed as part of the CPT-38 testing programme using similar materials but including an extra concrete mix containing exclusively Auckland greywacke aggregate:

- 2a containing Waikato River sand with non-reactive fine and coarse aggregate
- 2c containing Waikato River sand with Auckland fine and coarse aggregate
- 2d containing Auckland greywacke fine and coarse aggregate
- 2e containing Rangitikei sand with non-reactive coarse aggregate

Four concrete mixes were assessed as part of the CPT-60 testing programme using potentially reactive material used in the Auckland, Bay of Plenty and lower North Island construction markets:

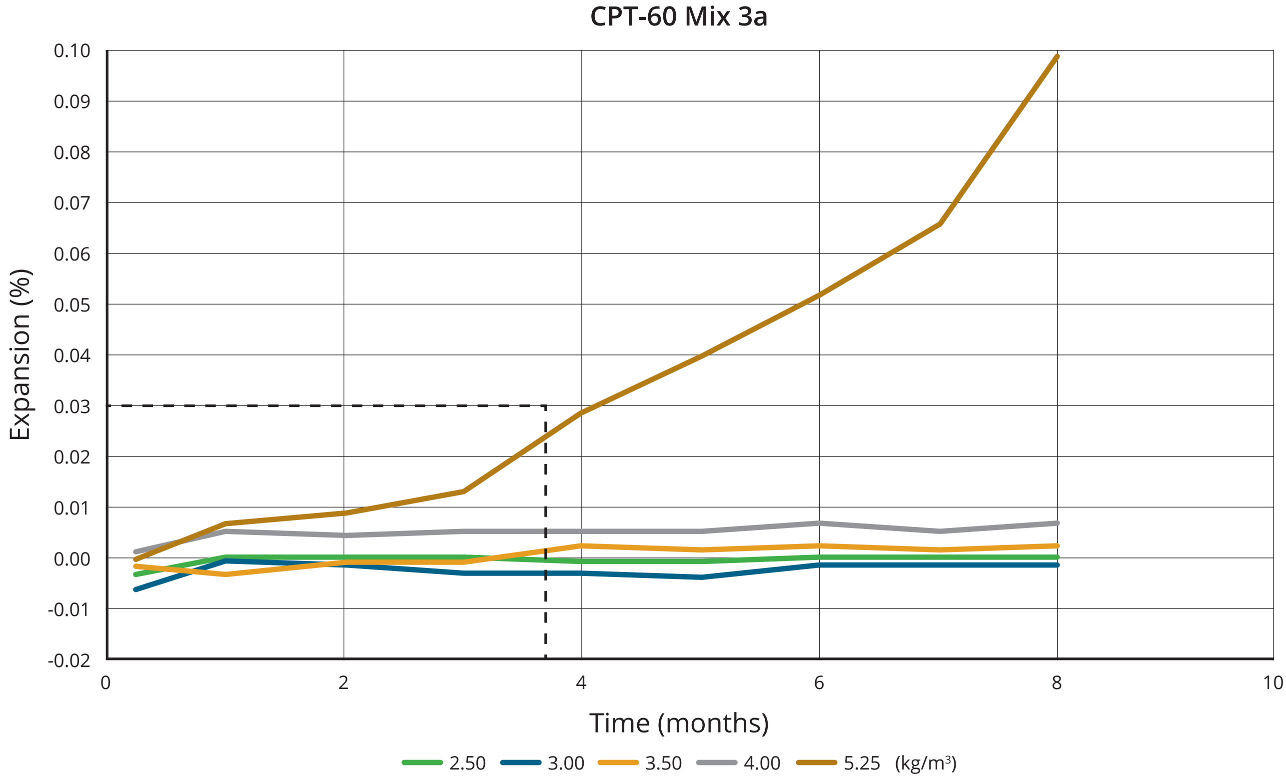
- 3a containing Waikato River sand with non-reactive fine and coarse aggregate
- 3c containing Waikato River sand with Auckland greywacke fine and coarse aggregate
- 3e containing Rangitikei sand with non-reactive coarse aggregate
- 3f containing BoP andesite PAP and non-reactive fine and coarse aggregate

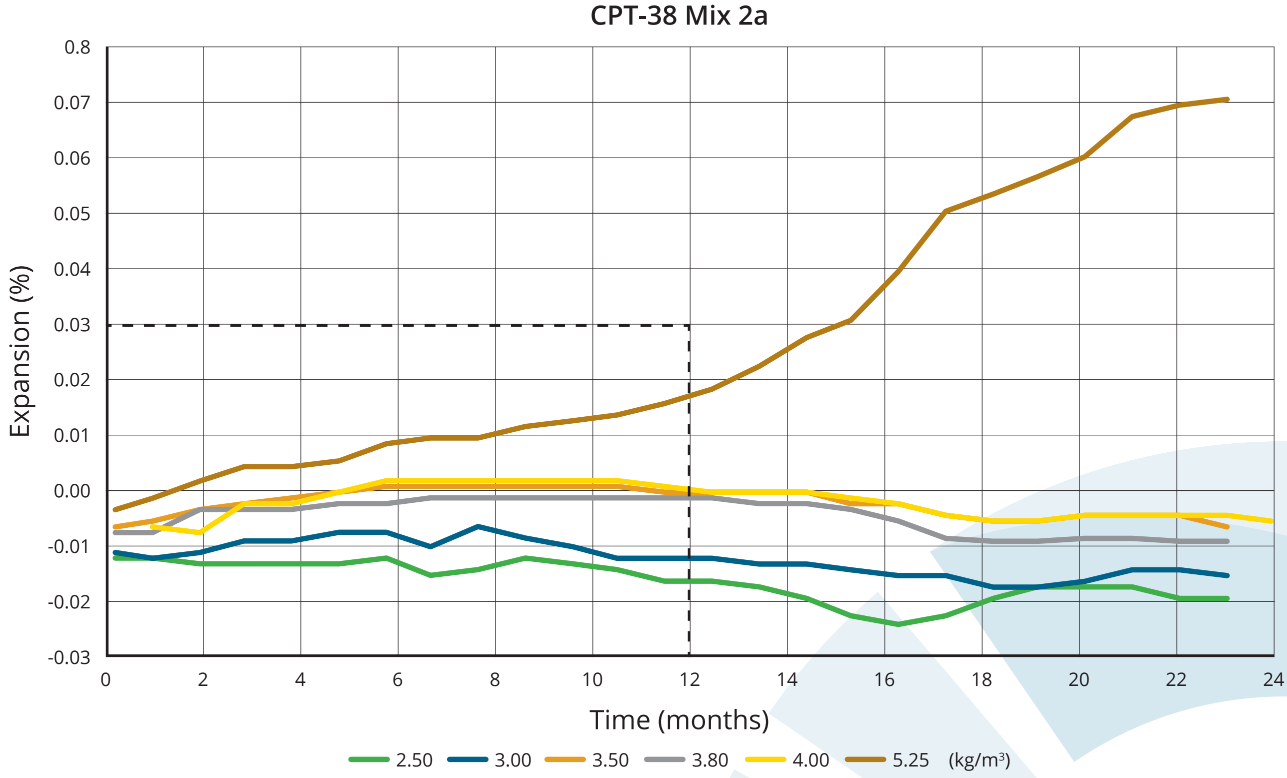
Concrete was tested at varying concrete alkali contents (2.5-5.8 kg/m^3) using sodium hydroxide added during mixing of each concrete mix. Given the size of concrete prisms (75x75x280 mm) it is expected that there will be some leaching during storage. Testing after 18 months at 38^{o}C found that alkali leaching was less than 5% in all cases and was not significant when using a high strength and carefully design concrete mix.

EFFECT OF CONCRETE ALKALI CONTENT ON ASR EXPANSION

Testing for alkali silica reactivity using CPT-60 has not been standardised internationally and running parallel testing on the same concrete mixes was undertaken to compare performance. The criterion for assessing the risk of ASR expansion for both testing methods was an expansion of 0.03% after a period of 15 and 52 weeks when run at 60^{o}C and 38^{o}C respectively. Figure E1 shows the results of CPT-60 and CPT-38 testing on concrete containing Waikato River sand and non-reactive aggregate.

Figure E1: CPT-60 and CPT-38 expansion of Waikato sand for alkali contents of 2.50-5.25 kg/m³



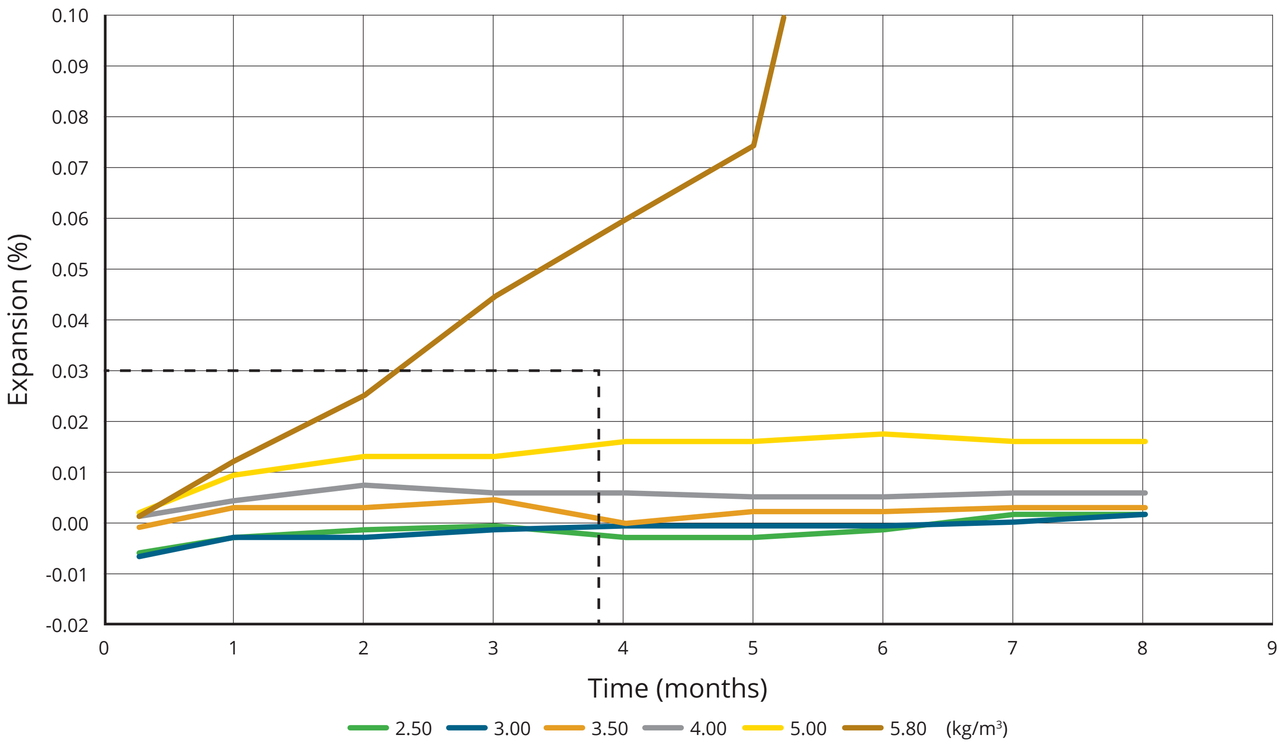


Analysis of test data from both techniques found similarities in that alkali levels needed to be above 5 kg/m³ for concrete to exhibit any significant expansion. Expansion increased after three months for concrete containing 5.25 kg/m³ alkali when tested with CPT-60 whereas similar concrete when tested with CPT-38 only started to expand significantly after 12 months. The higher temperature of CPT-60 tended to increase expansion levels.

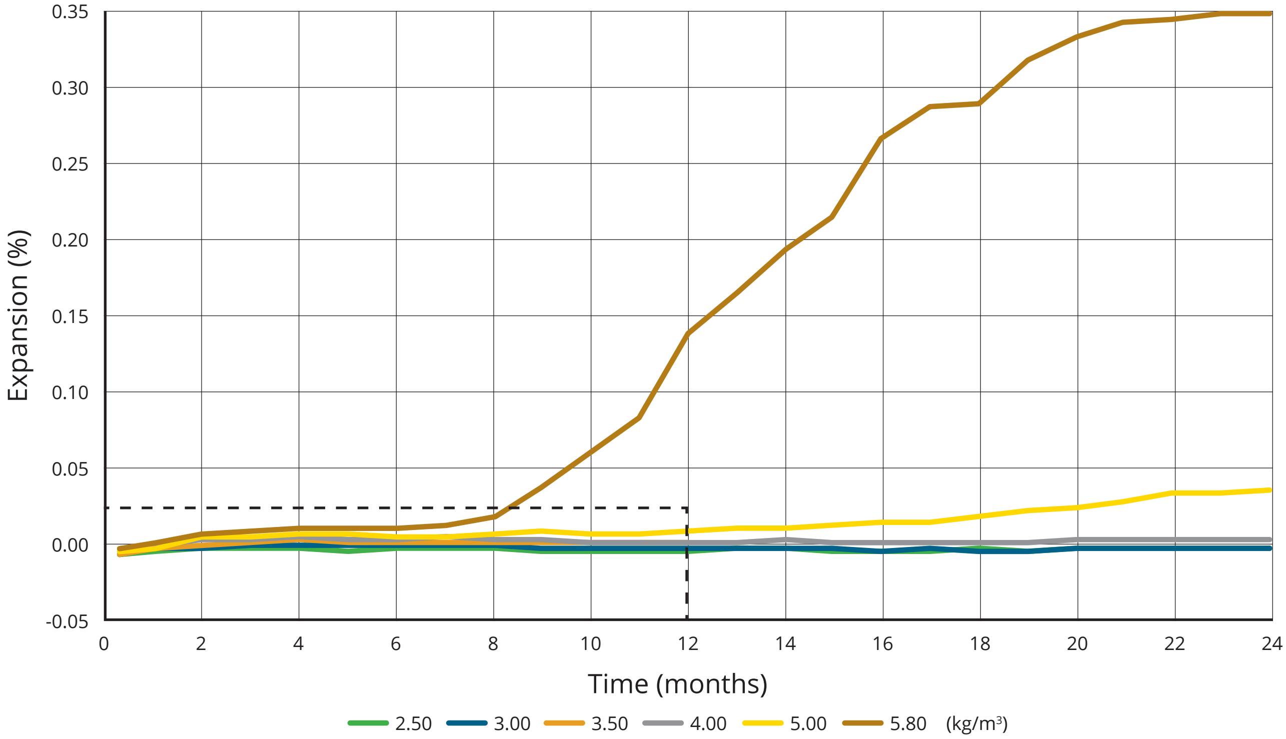
The only other aggregate combination that was reactive at high alkali levels was Rangitikei sand used in samples 2e (CPT-38) and 3e (CPT-60). Figure E2 shows expansion for these tests with the higher temperature samples expanding faster and more than those stored at lower temperatures.

Figure E2: CPT-60 and CPT-38 expansion of Rangitikei mix for alkali contents of 2.5-5.8 kg/m³

CPT-60 Mix 3e



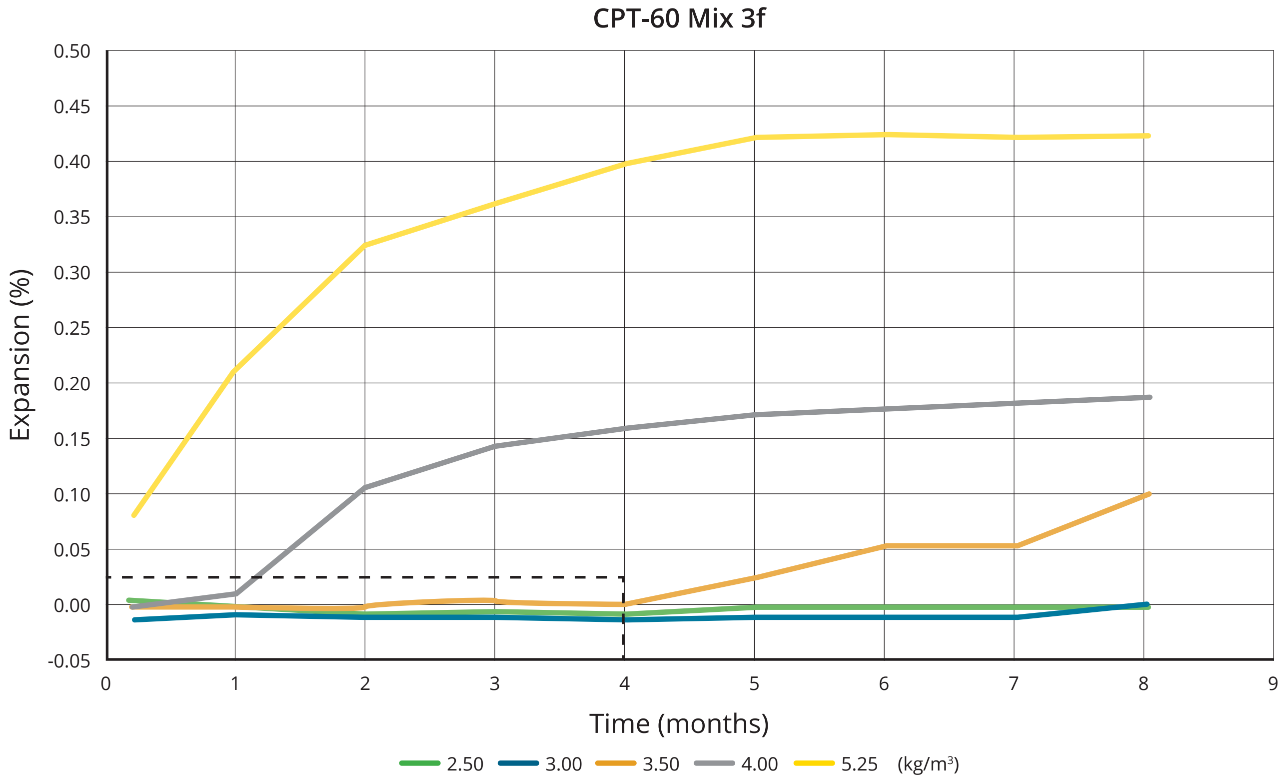
CPT-38 Mix 2e



CPT-60 testing was continued for 8 months, which is more than twice the 15 weeks recommended for assessing reactivity and expansion was only observed in some concrete mixes (e.g. mixes 3a and 3e) when exposed to alkali levels above 5 kg/m³, which is equivalent to over 750 kg/m³ of Portland cement. CPT-38 testing after 24 months had similar but slower trends. While expansions reported for each test differ, especially at high alkali levels, similar trends and rankings were found for both techniques.

Further testing in 2019 was undertaken using andesite aggregate from the Bay of Plenty, which is known to be alkali silica reactive (see Figure E3). To comply with the RILEM protocols for assessing aggregate reactivity, only the fine aggregate was used in concrete mixes and non-reactive fine and coarse aggregate was used. The concrete mix was the same as used previously.

Figure E3: CPT-60 for concrete containing Bay of Plenty andesite PAP for alkali levels from 2.5-5.25 kg/m³



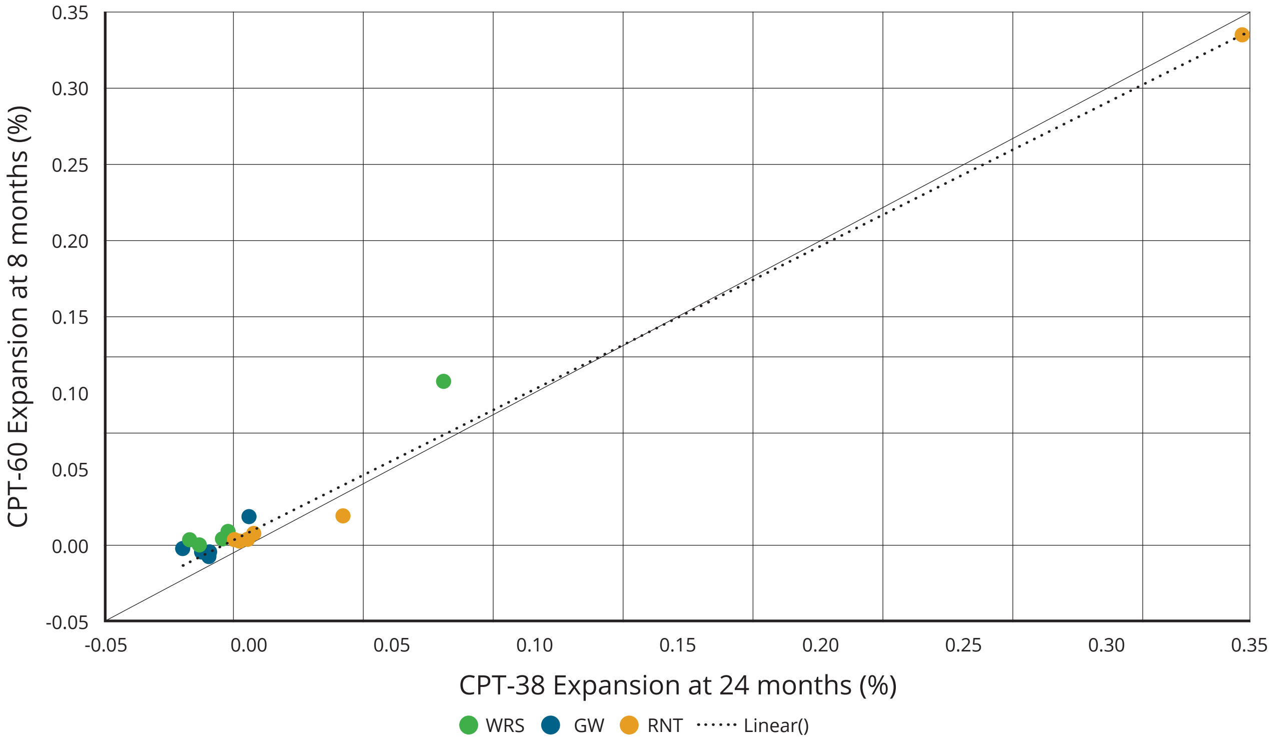
Relatively rapid expansion of concrete containing Bay of Plenty andesite was found to occur at alkalis levels of 4.0 kg/m³ and above. No evidence of expansion was seen in concrete at alkali levels of below 3.5 kg/m³.

COMPARISON OF CONCRETE PRISM TESTING AT 38°C AND 60°C

Running parallel testing at 38 and 60ºC was found to produce similar trends and rankings of potential reactivity for different aggregate combinations. Findings from this testing programme, while limited in terms of possible types of aggregate reactivity, showed that CPT-60 testing would provide a cheaper and quicker assessment of potential reactivity without producing unnecessary amounts of false positive results. Figure E4 shows the comparison between CPT-38 after 24 months and CPT-60 after 8 months, which shows a reasonable correlation for this limited survey of aggregate combinations.

The results do not imply that concrete alkali levels as high as 3.5 kg/m³ can safely be used for all aggregates. The results do however indicate that the prescribed maximum limit of 2.8 kg/m³ is unlikely to produce significant ASR damage with three widely used New Zealand reactive aggregates.

Figure E4: Comparison between CPT-38 and CPT-60 expansion results for concrete containing Waikato River sand (WRS), Rangatikei River sand (RNT) and Auckland greywacke (GW)



Using a combination of both concrete prism tests is recommended to build greater confidence in these techniques. Both techniques need to be run for longer than the suggested periods of 15 weeks for CPT-60 and 12 months for CPT-38 to ensure that slowly reactive aggregates are properly assessed during testing. This means that CPT-60 should be run for six months while CPT-38 should run for 24 months.

APPENDIX F: 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 F1-F6. 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 F1. 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 F2 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 trending to andesite. For example, samples 147 and 148 from a major Auckland quarry contain a silica content of 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 F3, 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 pessimum 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 F4 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 F5. 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 F6 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 pessimum proportion in the mortar bar test. In contrast sample 117, a hornsfelsed 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 7.2) 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

	Sample No.	Date tested	Testing
Eastern andesites	1 - 17 118 - 124	1960 1988	DSIR DSIR
Central andesites	18 - 43 138	1960 1988	DSIR DSIR
Taranaki andesites	44 - 50 130 - 136	1960 1988	DSIR DSIR
Basalt	51 - 53 139 - 145 147 - 150	1960 1988 1990	DSIR DSIR CCANZ
Dacite	54 - 55 137	1960 1988	DSIR DSIR
Greywacke + volcanics	64 - 69	1960	DSIR
Greywacke	56 - 63 163 - 167	1960 1990	DSIR CCANZ
Ignimbrite	70 - 71	1960	DSIR
Jasper	72	1960	DSIR
Obsidian	73	1960	DSIR
Pitchstone	74	1960	DSIR
Pumice	75 - 76 129 157	1960 1988 1990	DSIR DSIR CCANZ
Quartzite	77 - 79	1960	DSIR
Rhyolite	80 - 93 125 - 128	1960 1988	DSIR DSIR
Waikato River	94 - 107 151 - 156	1960 1990	DSIR CCANZ
Marine sands	108 - 110 158 - 162	1960 1990	DSIR CCANZ
Miscellaneous	111 - 117	1960	DSIR

	Quarry or Source	Location	Rock Type(s)	Metric Grid Reference
1-2	Woodlands	Woodlands, Tauranga	Eastern andesite	T13/659063
3	Thompsons Track (NC)	Aongatete	Eastern andesite	T14/64-93-
4	Te Poi (NC?)	Te Poi	Eastern andesite	
5	Coles	Te Poi	Eastern andesite	T14/650728
6	Barton Road	Okauia	Eastern andesite	T14/650727
7	Matatoki	Matatoki	Eastern andesite	T12/432415
8	Tirohia (NC?)	Tirohia	Eastern andesite	T13/44-14-?
9-9A	Leachs	Tirohia	Eastern andesite	T13/457153
10	McNaughtons	Papamoa	Eastern andesite	U14/995805
11	Duncans	Papamoa	Eastern andesite	U14/992802
12	Papamoa Beach	Papamoa	Andesite, rhyolite	U14/992848
13-14	Otaramakau Beach	Otaramakau	Andesite, rhyolite	V15/259667
15	Tauranga River (NC?)	Tauranga County	Quartz, volcanic glass	?
16	Paegaroa River (NC?)	Tauranga County	Quartz, volcanic glass	?
17	Borough Council	Te Aroha	Eastern andesite	T13/504033
18A-B	Parakaua Gorge (NC)	Orakeikorako	Central andesite	U17/868941
19	Rolles Peak (NC)	Tauhara	Central andesite	U18/920768
20A-B	Tongariro River	Turangi	Andesite, greywacke	T19/524442
21	Mt Tihia (NC)	Turangi	Central andesite	
22	Papamanuka Stream	Taurewa	Central andesite	S19/256287
23	Wanganui River	Tongariro	Central andesite	T19/444406
24A-B	Mangatepopo Prison	Taurewa	Central andesite	T19/320342
25	Tongariro Stream	Access 15, Desert Road	Greywacke, rhyolite, andesite	T20503157
26	Waihaha Stream	Rangipo	Argillite, greywacke, volcs	T19/547255
27	Papakai (NC)	Papakai	Central andesite	T19/365358
28	Papakai (NC)	Papakai	Central andesite	T19/365358
29	Otukou	Otukou	Central andesite	T19/392379
30	Otukou	Otukou	Central andesite	T19/392379
31	Tongariro River	Rangipo	Andesite, greywacke	T19/532245
32	Moawhango River	Waiouru	Greywacke, argillite, volcs	T20/474977
33	Tongariro River	Rangipo	Andesite, greywacke	T19/541368
34A-B	Whakapapa River	Rangipo	Andesite, rhyolite	S19/225291
35	Tongariro River	Rangipo	Andesite, rhyolite, greywacke	T19/527238
36	Whangaehu River	Waiouru	Central andesite	T20/428998
37	Otukou	Otukou	Andesite, rhyolite	T19/399387
38	Mangateotoenui Stream	Waiouru	Central andesite	T20/462152

	Quarry or Source	Location	Rock Type(s)	Metric Grid Reference
39	Tongariro River	Rangipo	Central andesite	T19/527240
40	Turakina River	Turakina	Greywacke, rhyolite, andesite	S23/986282
41	Wanganui River	Taumaranui	Greywacke, rhyolite, andesite	S18/111556
42	Wanganui River	Atene	Central andesite	S21/941625
43	Wanganui River	Atene	Central andesite	S21/941625
44A-B	Waiwakaiho River	New Plymouth	Taranaki andesite	P19/084333
45A-C	Stony River	Okato	Taranaki andesite	P19/847234
46	Kapuni Stream	Tokoroa	Taranaki andesite	Q21/139802
47A-B	Waitara River	Waitara	Taranaki andesite	Q19/171431
48	Kaupokonui Stream	Rowan	Taranaki andesite	P20/079011
49	Waitara River	Waitara	Taranaki andesite	Q19/178427?
50	Manganui River	Midhurst	Taranaki andesite	Q20/126120?
51	K-Trig basalt (NC?)	Taupo	K-trig basalt	U18/735762
52A-B	Watts	Ongaroto	Ongaroto basalt	U17/700062
53A-B	Smeeds	Tuakau	Franklin basalt	R12/862329
54-55	Burrows	Taupo	Tauhara dacite	U18/864758
56A-D	Rimutaka Hill	Kaitoke	Greywacke	
57	Waotu (Muku)	Waotu	Greywacke	T16/448304
58	Whitehall	Whitehall	Greywacke	T15/357654
59A-B	Roxburgh Hydro	Roxburgh	Greywacke	G43/21-18-
60-A	Hutt River	Melling	Greywacke	R27/707991
61	Stevensons	Drury	Greywacke	R12/865506
62	Benmore Hydro	Otematata	Greywacke	H39/86-22-
63	Aviemore Hydro	Omarama	Greywacke	I40/99-13-
64-65	Waimarino River	Korohe	Greywacke, volcanics	T19/57-45-
66-67	Tauranga-Taupo River	Te Rangi-ita	Greywacke, volcanics	T19/662495
68-69	Waimarino River	Korohe	Greywacke, volcanics	T19/57-45-
70	Parkinsons	Lichfield	Ignimbrite	T16/577379
71	Hinuera Stone	Piarere	Ignimbrite	T15/459614
72	McCallums Island	Kawakawa Bay	Jasper	S11/022740
73	Poihipi Road (NC)	Taupo	Obsidian	U18/76-78-?
74	Gebbies Pass (NC)	Gebbies Pass	Pitchstone	M36/808247
75	Wanganui River	Matahiwi	Pumice	S21/956754
76	Te Teko	Te Teko	Pumice	V15/369461
77-79	Cobb Valley Hydro	Cobb	Quartzite	M26/83-10-
80-88	Whakamaru Hydro	Whakamaru	Rhyolite	T16/501131

	Quarry or Source	Location	Rock Type(s)	Metric Grid Reference
89	McClaren Falls	Lower Kaimai	Rhyolite	U14/783728
90	Pohaturoa	Ongaroto	Rhyolite	U17/728069
91	Ngatuku	Ngatuku	Rhyolite	U16/742111
92	Mangatawa	Mangatawa	Rhyolite	U14/962841
93	Aongatete Stream	Aongatete	Rhyolite, ignimbrite	T14/690947
94	Waikato River	Karapiro	Rhyolite-andesite	T15/32-60-
95-96	Waikato River	Mangakino	Rhyolite-andesite	T16/52-10-
97-98	Waikato River	Arapuni	Rhyolite-andesite	T15/41-44-
99A-B	Waikato River	Atiamuri	Rhyolite-andesite	U17/74-07-
100-102	Waikato River	Arapuni	Rhyolite-andesite	T15/41-44-
103	Waikato River	Maraetai	Rhyolite-andesite	T16/50-13-
104-105	Waikato River	Whakamaru	Rhyolite-andesite	T17/55-04-
106-107	Waikato River	Huntly	Rhyolite-andesite	S14/003989
108	Gisborne Beach	Gisborne	Greywacke, volc glass	N98/31-30-
109-110	Waihi Beach	Waihi	Quartz, obsidian	U13/70-15-?
111	Pyrex glass	Jobling, UK	Boro-silicate glass	
112	Sil-mag-limestone	California	Sil-mag-limestone	
113	Standard quartz sand	Leighton Buzzard	Quartz	
114-115	Synthetic cristobalite		Cristobalite	
116	Steel slag	Glenbrook steel mill	Ti steel slag	
117	Hornsfelsed shale	Malmesbury, South Africa	Hornsfelsed shale	
118	McBeths	Hikuai	Eastern andesite	T12/599505
119	Tairua	Whenuakite	Eastern andesite	T11/594695
120	Leachs	Tirohia	Eastern andesite	T13/459153
121	Wharawhara Road	Katikati	Eastern andesite	T14/631998
122	Barton Road	Okauia South	Eastern andesite	T14/650728
123	Kaitemako	Tauranga	Eastern andesite	U14/933768
124	Poplar Lane	Papamoa	Eastern andesite	U14/991791
125	Te Puna	Tauranga	Rhyolite	U14/787840
126	Mangatawa	Te Maunga, Tauranga	Rhyo-dacite	U14/963842
127	Daltons	Matamata	Rhyolite, ignimbrite, pumice	T15/522696
128	Schwartzs	Tirau	Rhyolite	T15/522557
129	Renners	Otaramakau	Pumice	V15/270659
130	Stony River	Okato	Taranaki andesite	P19/847234
131	Waiwakaiho River	New Plymouth	Taranaki andesite	P19/084333
132	Manutahi Road	Hillsborough, New Plymouth	Taranaki andesite	Q19/108375

	Quarry or Source	Location	Rock Type(s)	Metric Grid Reference
133	Waitara River	Waitara	Taranaki andesite	Q19/172425
134	Mangonui River	Midhurst	Taranaki andesite	Q20/128121
135	Kapuni River	Kapuni	Taranaki andesite	Q21/103888
136	Fishers	Pungarehu	Taranaki andesite	P20/881118
137	Burrows	Taupo	Tauhara dacite	U18/865759
138	Otukou	Otukou	Central andesite	T19/393379
139	Lunn Avenue	Auckland	Auckland basalt	R11/751799
140	Okete	Okete	Alexandra basalt	R14/790751
141	Jowseys	Te Mata, Raglan	Alexandra basalt	R15/777645
142	Te Pahu	Kaniwhaniwha	Alexandra basalt	S15/942638
143	Tuakau	Tuakau	Franklin basalt	
144A-D	Bombay (NC)	Bombay	Bombay basalt	
145	Smeeds	Pukekawa	Franklin basalt	R12/863329
146A-B	Lunn Av. (Winstones)	Auckland	Auckland basalt	R11/751779
147	Lunn Av. (Winstones PAP7)	Auckland	Auckland basalt	R11/751779
	Lunn Av. (Winstones PAP7)	Auckland	Auckland basalt	R11/751779
149-A-C	East Tamaki (Milburn)	East Tamaki, Auckland	Auckland basalt	R11/794718
150	East Tamaki (Stevenson)	East Tamaki, Auckland	Auckland basalt	R11/793712
151	Waikato River		Rhyolite, andesite	
152	Waikato River (Stevensons)	Mercer	Rhyolite, andesite	S12/929315
153A-C	Waikato River (Stevensons)	Mercer	Rhyolite, andesite	S12/929315
154A-B	Waikato River	Puni	Rhyolite, andesite	R12/740340
155	Waikato River	Tuakau	Rhyolite, andesite	R12/825325
156	Waikato River (King)	Tuakau		R12/825325
157	Waikato River (Stevensons)	Mercer	Pumice	S12/929315
158	Kaipara Harbour (Sea Tow)	North Head, Kaipara Harbour	Marine quartz sand	Q9/15-36-
159A-B	Mt Rex Silica	Helensville	Quartz sand	Q10/399051
160	Kaipara Hb (Atlas Consold)	Kaipara Harbour	Marine quartz sand	Q9/15-36-?
161	Kaipara Harbour (Sea Tow)	North Head, Kaipara Harbour	Marine quartz sand	Q9/15-36-
162	Pakiri Beach (McCallum)	Pakiri Beach	Marine quartz sand	R9/658494
163	Hunua (Winstones PAP7)	Hunua Gorge	Greywacke	R12/876563
164	Hunua (Winstones	Hunua Gorge	Greywacke	R12/876563
165A-C	Stevensons	Drury	Greywacke	R12/866507
166A-C	Dannevirke	Dannevirke	Greywacke	U23/776051?
167	Otaika (Winstones)	Otaika, Whangarei	Greywacke	Q7/284033

Figure F1: Results of testing Egmont andesite from Taranaki by ASTM C289.

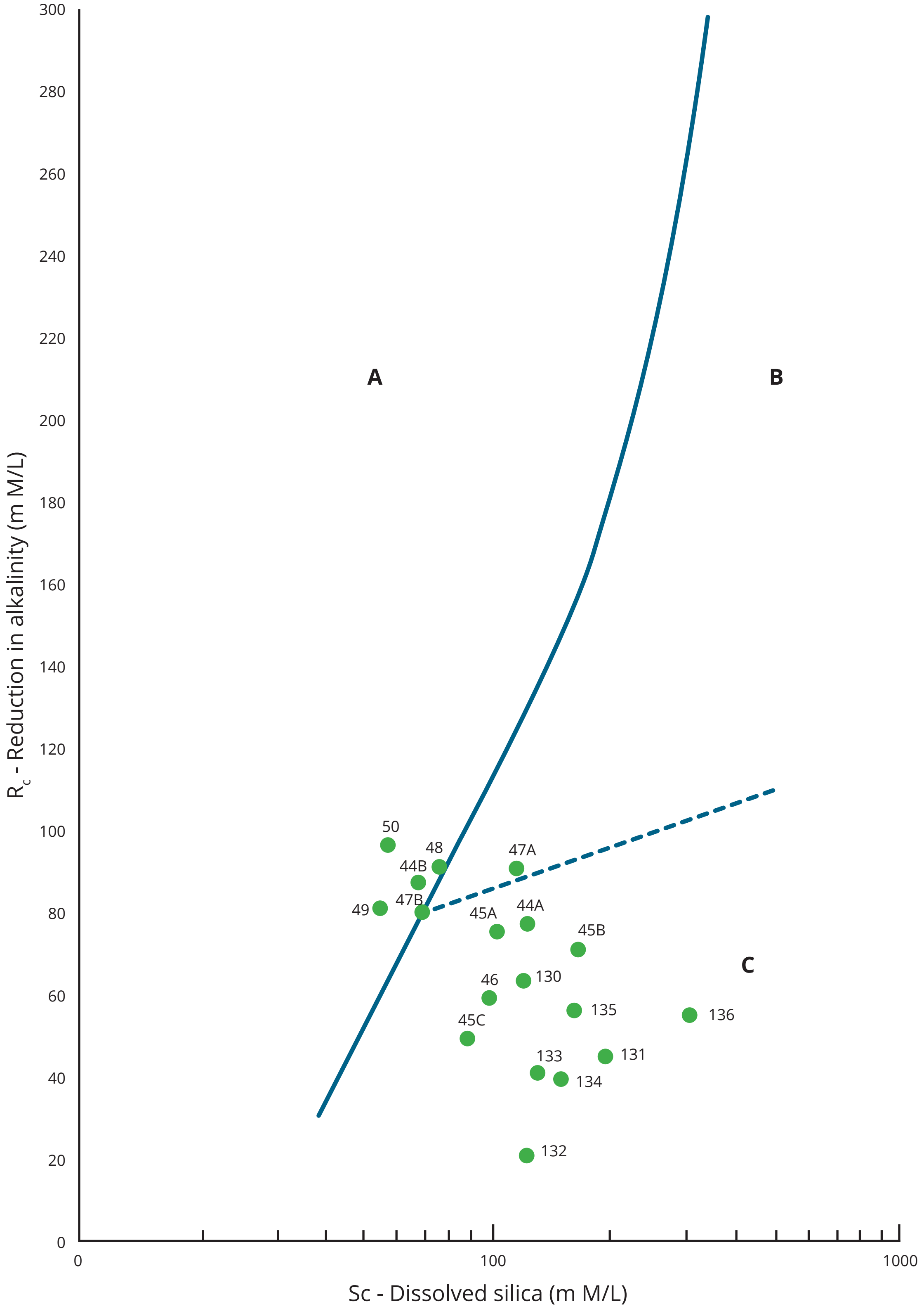


Figure F2: Results of testing basalt by ASTM C289.

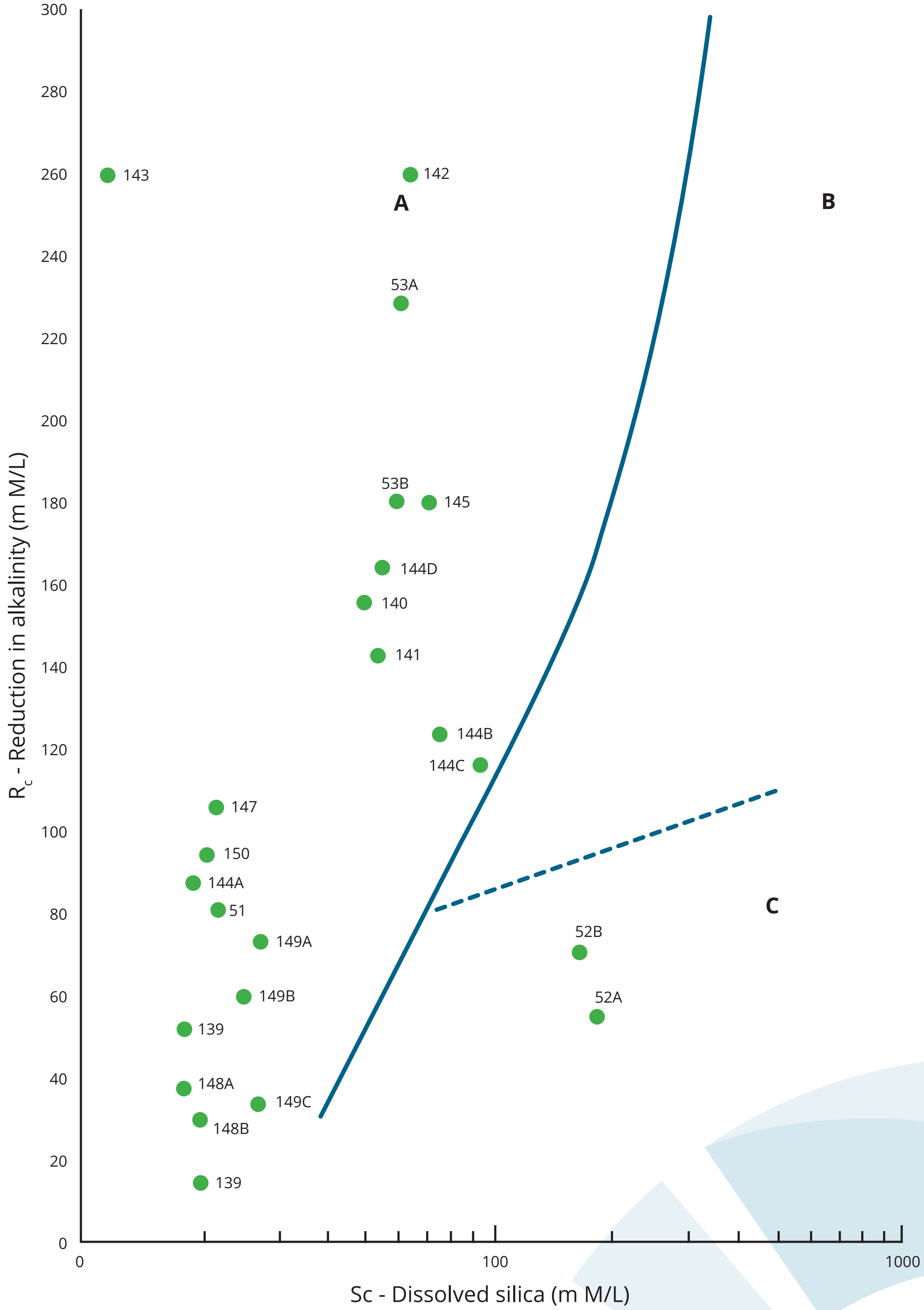


Figure F3: Results of testing andesite from the central North Island by ASTM C289.

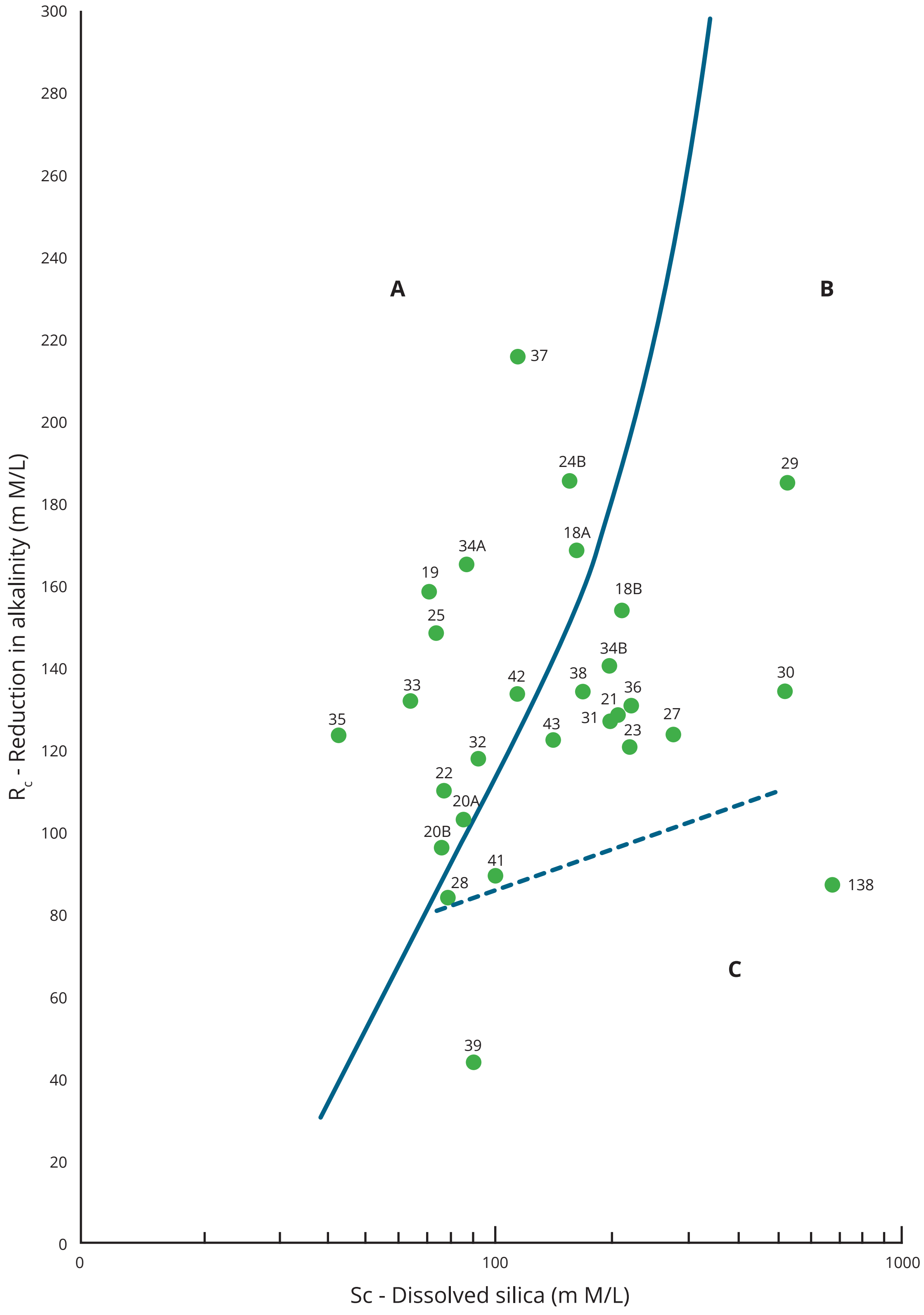


Figure F4: Results of testing andesite from the eastern region of the North Island by ASTM C289.

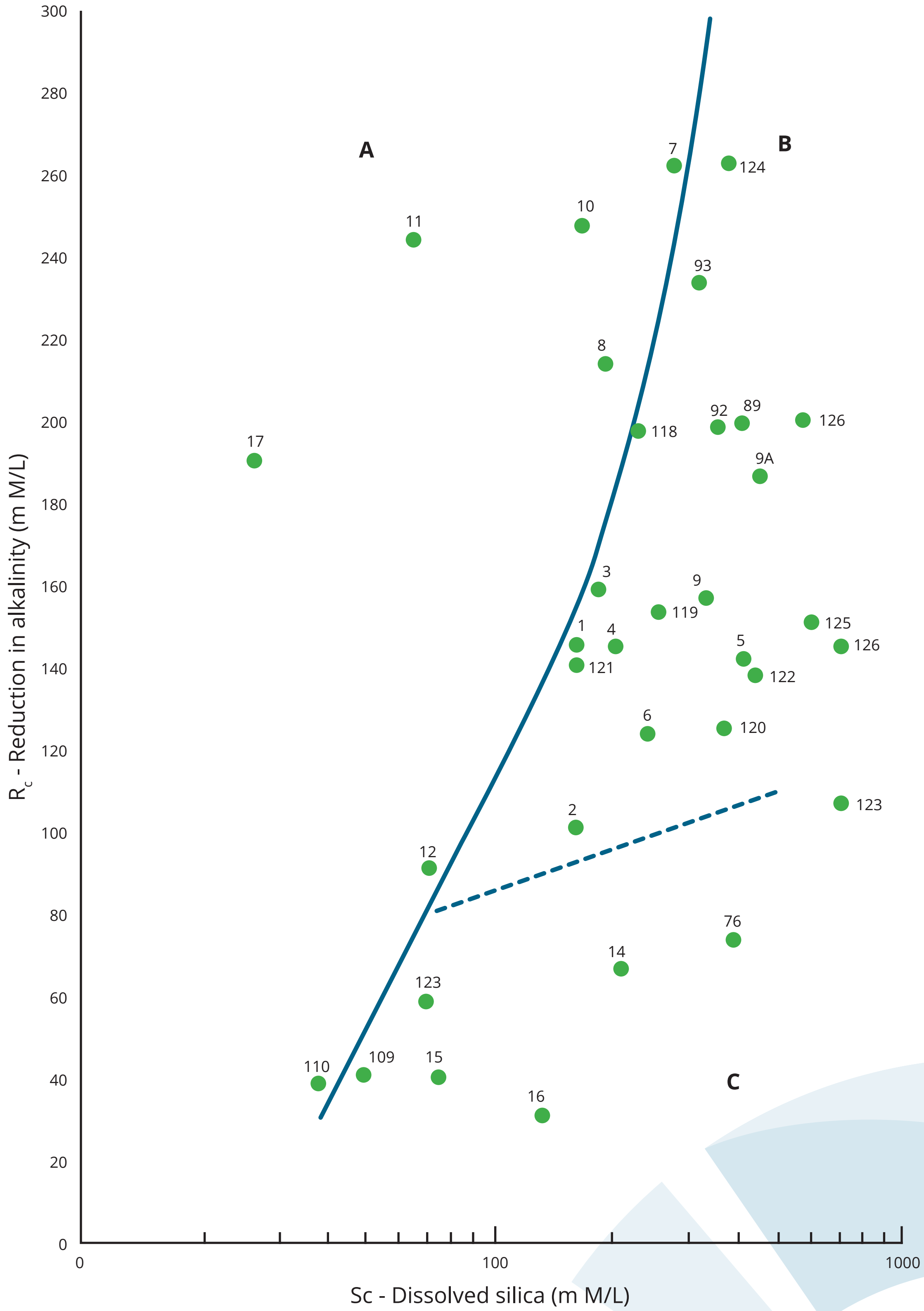
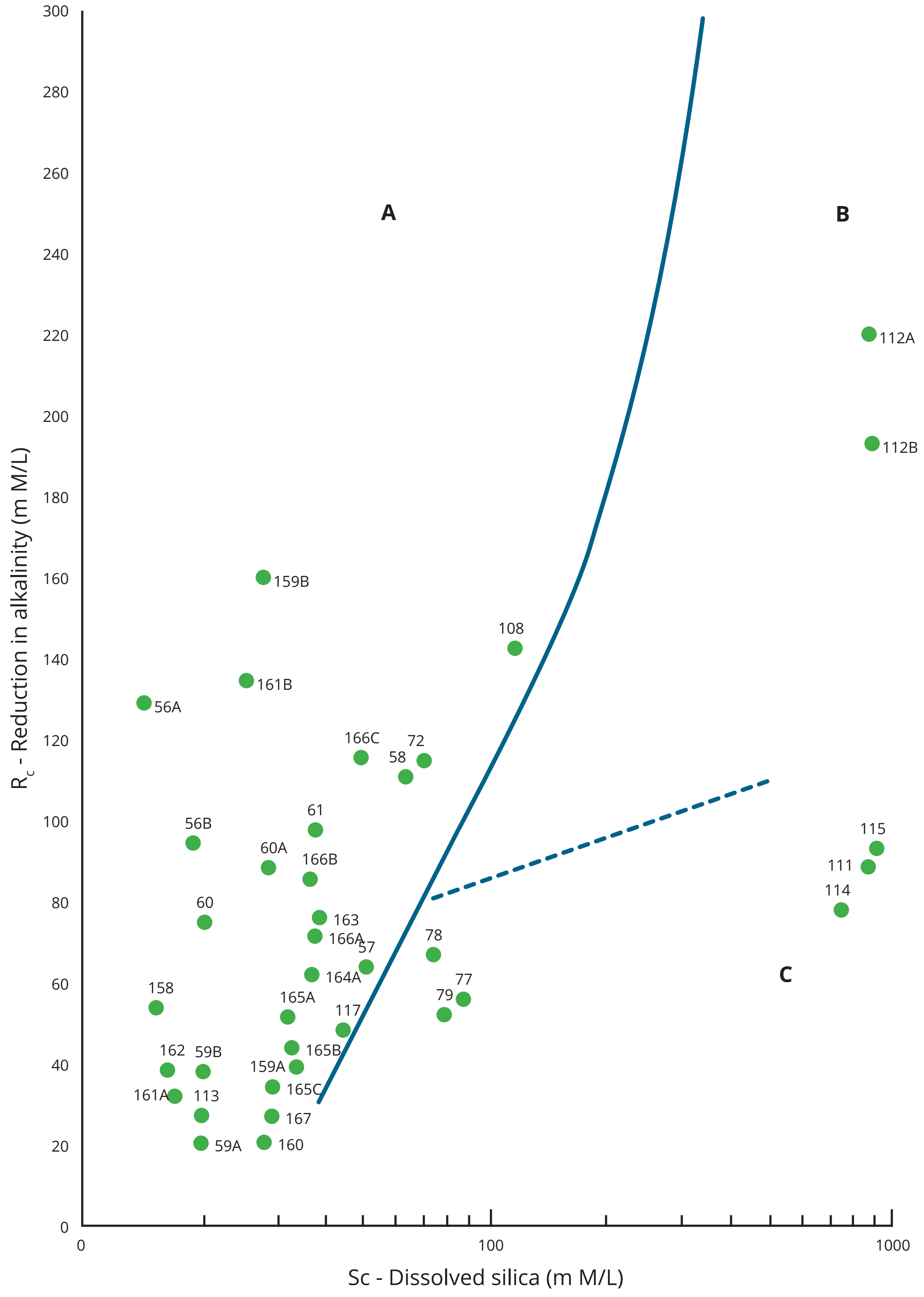


Figure F5: Results of testing rhyolite, dacite and some alluvial materials containing these rock types by ASTM C289.



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