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FINAL REPORT

S51: Suitability of the Use of Recycled Aggregate in Concrete (2020–21)

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Summary

This report presents the findings of a literature review and the results of engagement with the concrete and material recycling industries on the utilisation of recycled aggregates in concrete. The focus of the literature review and the survey was the application of recycled aggregates in non-structural concrete in terms of aggregate properties and concrete performance in order to potentially justify the use of these recycled aggregates in future Queensland Department of Transport and Main Roads (TMR) projects. Data from overseas has been included to support or complement the Australian findings.

The scope of the Year 1 project was to investigate the viability of three recycled aggregate materials for use in non-structural concrete applications, which are broadly consistent with TMR Normal (N)-Class concrete applications up to 40 MPa characteristic compressive strength. These materials were recycled concrete aggregate (RCA), recycled crushed glass (RCG) and reclaimed aggregate (RA). RCA is recovered from crushed recycled concrete and contains pieces of the old aggregate and mortar. RCG is produced from crushing recycled glass containers and other glass products. RA is reclaimed from fresh concrete by washing the cement paste out of the aggregate or by use of a chemical admixture to form balls of hardened cement paste and aggregate. The characteristic properties of each recycled aggregate vary considerably, particularly those of RCA, as the material properties vary according to the strength grade and the type of the natural aggregate used in the original concrete which has been crushed and recycled.

The scope of the Year 2 project was to extend the investigations of the Y1 project into an additional two recycled aggregate materials, including ferronickel slag (FNS) and power station bottom ash (BA). These materials were primarily considered as potential fine aggregate replacements. Both materials are by-products of heavy industrial processes which are ferronickel alloy manufacturing and coal-fired power generation, respectively. The physical and chemical properties of FNS may vary depending on the processing method used and the type of source ore used.

For both project Years, the performance of concrete mixes which utilise recycled aggregates to replace part, or all, of the natural aggregates was reviewed in terms of the workability of the fresh mix, the mechanical properties of the hardened concrete, and their durability and structural performance.

Review of the literature indicated that at high replacement levels (50% up to 100% by mass), coarse or fine RCA concrete is likely to develop reduced strength and durability properties compared to an equivalent mix produced using 100% natural aggregate (NA). However, at replacement levels of up to 30% of coarse RCA, the workability, strength and durability properties of RCA concrete are likely to be acceptable for non-structural concrete applications. At this replacement level, there is not expected to be an impact on the feasibility of target design lives of up to 50 years as specified for TMR N-Class concrete. The risks and

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benefits associated with RCA concrete at replacement levels of 30-50% are unclear and replacement levels up to 50% may be feasible if performance that meets TMR specifications can be demonstrated by further investigation or trials.

The use of fine RCA is generally not recommended due to issues associated with decreased workability and increased water demand, though some research indicates that there may be potential to include up to 30% fine RCA by mass in concrete.

Alkali-reactivity has been found to be increased for RCA in some circumstances but based on the literature reviewed it is likely that the TMR MRTS70 (11/2018) supplementary cementitious material (SCM) replacement levels (25% fly ash, or 60% blast furnace slag) will be sufficient to mitigate ASR in RCA concrete up to a 30% replacement level.

The presence of contaminants in RCA is likely to have a detrimental impact on RCA concrete properties, and it is recommended that contaminants such as wood, plastic, metal, brick etc. are maintained at a level of 1% or lower by volume. An upper limit on water absorption of 6% is recommended for coarse RCA to maintain the strength and durability of the RCA concrete.

The cost and availability of RCA have been identified as potential barriers for the implementation of RCA in concrete, and further industry engagement is recommended to gain a better understanding of these issues in the Queensland context. Such engagement should also focus on other identified barriers to implementation including perceived poor performance of RCA concrete. It is recommended that TMR collaborates with concrete producers to undertake field trials of coarse RCA in N-Class (non-structural) concrete with replacement levels ranging from 10% up to 30% by total mass of coarse aggregate.

Review of the literature indicates that fine RCG may be used to replace up to 30% of natural fine aggregate by total aggregate mass without having a detrimental impact on strength or durability properties. Reductions in strength, durability and workability properties have been observed in some cases for mixes using 40% or higher replacement levels. Allowance for 30% replacement of natural fine aggregate with fine RCG is in line with the current VicRoads specification Section 703. The use of coarse RCG in concrete is not recommended at this time, due to uncertainties associated with the strength and shape of the coarse RCG particles.

The ASR reactivity of glass is a key consideration, and it has been found that the current TMR SCM replacement levels of 25% fly ash and 60% GGBFS are likely to mitigate ASR in concrete using up to 30% (or greater) fine aggregate replacement using RCG.

The availability of good quality RCG (free of contaminants such as sugar, metal, paper etc.) may be an issue for the widespread uptake of RCG in concrete. Sugar should not be present in detectable levels, and other contaminants should be maintained below 2% by mass of RCG. The cost of RCG may impact upon its feasibility for concrete applications, since some degree of washing is generally required to remove contaminants including sugar. If more cost effective natural or manufactured sands are available this will reduce RCG feasibility. It is recommended that TMR trials the use of RCG at varying replacement percentages from 10% up to 30% prior to any large-scale implementation.

There has been limited research undertaken on concrete produced using coarse RA, though some positive results have been obtained for a study using up to 90% by mass replacement of natural coarse aggregate with coarse RA. Further research into appropriate replacement levels and available admixture products is recommended prior to a decision on an appropriate inclusion amount for coarse RA.

FNS appears to be a technically viable fine aggregate replacement material at levels up to 35% inclusion in TMR non-structural concrete. At this replacement level there are not expected to be any significant impacts on concrete strength or durability properties. Blends with manufactured sand and natural sand may be required to provide an adequate proportion of fines in the 0.6 mm – 0.3 mm fractions. Discussion and trialling with concrete producers are recommended to determine appropriate mix designs. The inclusion of at least 25% fly ash is required to mitigate ASR in FNS concrete.

At levels of up to 30% inclusion as fine aggregate replacement, BA appears to be a technically viable option for inclusion in TMR non-structural concrete. At this level there are not expected to be significant impacts on strength or durability properties. There is a significant volume of BA available, and discussion/trialling with concrete producers and BA suppliers is recommended to determine appropriate mix designs and identify any issues associated with concrete performance.

Overall, it is recommended that TMR considers allowing the following replacement levels of recycled aggregates in N-Class non-structural concrete up to 40 MPa characteristic compressive strength as per the S51 project scope, pending the positive outcomes of any field trials.

- up to 30% by combined coarse aggregate mass of coarse RCA.
- up to 30% by combined fine aggregate mass of fine RCG.
- up to 35% by combined fine aggregate mass of FNS.
- up to 30% by combined fine aggregate mass of BA.

The following specification requirements for acceptance of recycled aggregate for use in concrete are recommended. These limits are in addition to the standard aggregate requirements specified by AS 2758.1:2014 and any relevant additional requirements specified by MRTS70 (11/2018) for use in TMR N-Class concrete.

- Coarse RCA
  - Inclusion of contaminants such as brick, metals, plastics and other demolition wastes maintained at a level ≤1% by mass of the RCA product.
  - Water absorption ≤6%
  - Inclusion of SCMs required to manage ASR chosen from the following options:
    - ≥25% fly ash
    - ≥60% GGBFS
    - 25% fly ash combined with 4 to 8% silica fume (ternary blend).
- Fine RCG
  - Loss on Ignition (LOI) ≤2%
  - Contaminant materials (ceramics, metals, specialist glasses etc.) limited to ≤2% by weight of RCG
  - Limits on concentration of chemicals, heavy metals and other attributes as per Table 6.2 of MRTS36 (11/2020)
  - Inclusion of SCMs required to manage ASR chosen from the following options:
    - ≥25% fly ash; or
    - ≥60% GGBFS; or
    - 25% fly ash combined with 4 to 8% silica fume (ternary blend).
- Ferronickel slag
  - Iron unsoundness: negative
  - Inclusion of SCMs to manage ASR:
    - ≥25% fly ash
- Power station bottom ash
  - Water absorption ≤6%.
  - Inclusion of SCMs: ≥25% fly ash as a percentage of total cementitious materials.

Combinations of recycled aggregate inclusions have not been investigated as part of this project, and as such cannot be recommended without appropriate further investigation.

The inclusion of recycled aggregates in concrete has the potential to create environmental and social benefits through diversion of material from landfill and reduction of the need to produce new aggregate material. At the replacement levels recommended in this report, it appears likely that this can be achieved without penalty to the performance of the N-Class (non-structural) concrete applications in scope of this review.

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1 Introduction

1.1 Background

The use of recycled aggregates in concrete production in Australia has gained some attention since around the turn of the 21st century (HB 155:2002), though widespread adoption in concrete does not appear to have been realised. For example, recycled concrete aggregate (RCA) has been identified as suitable for partial replacement of up to 30% of natural aggregate in concrete for footpaths, kerbs, etc. (Cement Concrete & Aggregates Australia 2008), but industry take-up appears to be limited. Recycled materials that have been identified by TMR for potential use as aggregate in non-structural concrete include crushed recycled concrete (RCA), reclaimed aggregate (RA), recycled crushed glass (RCG), ferronickel slag (FNS), and power station bottom ash (BA).

During consultation associated with the updating of TMR Technical Specification MRTS70 (11/2018) (Concrete), allowance to use these aggregates was requested by industry, but it was not progressed because the influence of recycled aggregates on the performance of concrete was not clear, and the availability and quality of sources of recycled aggregate were also unknown. While there are potential economic and environmental benefits associated with the use of recycled aggregate in non-structural concrete, use of these materials needs to be supported by strong evidence of consistent performance and suitable concrete properties. Any assessment of the feasibility of using these aggregates must consider both the properties and consistency of recycled aggregates, and the plastic and mechanical properties of the resulting concrete mixes, including durability.

In the past decades, laboratory and field research on using RCA and RCG in concrete has been conducted in Australia and overseas, mainly in terms of the mechanical properties and the durability of low- to moderate-strength grades of concrete. Some laboratory trials have focused on the use of these materials in structural applications and higher strength grades, but most field research has been focused on non-structural applications. Varying partial replacement levels of RCA and RCG have been considered in the research, from 10% up to 100% replacement of natural coarse or fine aggregate. Less research has been conducted on RA, FNS and BA, but a limited number of studies exist.

This report presents the findings of a literature review and an industry survey focusing on the topic of the suitability of using RCA, RA, RCG, FNS and BA as aggregate for non-structural concrete, based on the Australian and worldwide experience. The focus of the review and survey is on the properties of the recycled aggregates, the plastic and mechanical properties of the produced concrete, and the durability of the concrete produced with these materials. The costs and availability of the recycled materials are also investigated, and recommendations are made for the adoption of recycled materials and appropriate controls.

1.2 Aims and Objectives

The aim of this project was to determine whether recycled aggregates including RCA, RCG, RA, FNS and BA can be accepted by TMR for use in non-structural concrete applications which typically have a design life of 50 years or less. The viability of these materials for the intended application has been investigated through a literature review and industry survey focusing on aggregate properties, concrete performance, concrete durability, availability, and cost of materials.

1.3 Scope

The scope of the Year 1 project included investigation of the following recycled aggregate materials:

1. Recycled concrete aggregate (RCA)

2. Aggregate reclaimed from a plastic concrete mix (RA), including:
    a. Treated by using conventional methods (aggregate washed out of plastic concrete mix)
    b. Recycled using alternative methods (hardened concrete mixture at a very young age recycled by crushing, or by plastic concrete treated by using admixtures, e.g. Re-Con Zero Evo, and dried)
3. Recycled crushed glass (RCG).

Further recycled aggregates considered as part of the Year 2 scope of works include:

1. Ferro nickel slag (FNS)
2. Power station bottom ash (BA).

The scope of application considered by this project is non-structural concrete. Discussions with TMR revealed that the MRTS70 (11/2018) specification does not provide specific requirements for non-structural concrete, but that Normal (N) class concrete is generally used for such applications. Considering this, the intended scope of application is to be non-structural uses of N-Class concrete (up to 40 MPa compressive strength) with a design life of up to 50 years, which typically does not see regular heavy vehicle traffic, or high volumes of light vehicles. The scope of application is expected to include:

- footpaths and bike paths
- driveways, boat ramps, car parking areas
- kerb and channel, and other surface drainage works, and erosion protection works.

National and international literature was reviewed as part of the literature survey. The industry survey was conducted nationally with a focus on Queensland aggregate suppliers and concrete producers. Research into structural concrete has been considered as part of the literature review where available, but research outcomes have been considered in the context of non-structural applications.

1.4 Glossary

A list of abbreviations and glossary items used in this report is detailed in Table 1.1.

Table 1.1: Glossary of terms used in report

Term	Description
AAR	Alkali-aggregate reaction
AMBT	Accelerated mortar bar test
ASR	Alkali-silica reaction
BA	Power station bottom ash
CCP	Coal combustion products
Coarse aggregate	Aggregate having a nominal size greater than or equal to 5 mm (as per AS 2758.1:2014)
CPT	Concrete prism test
FA	Fly ash
Fine aggregate	Aggregate having a nominal size less than 5 mm (as per AS 2758.1:2014)
FNS	Ferronickel slag
GGBFS	Ground granulated blast furnace slag
GP cement (OPC)	General Purpose cement. Also known as Ordinary Portland cement (OPC) overseas
ITZ	Interfacial transition zone – region of cement paste that exists around aggregate particles
NA	Natural aggregate (virgin aggregates from typical quarry sources)
RA	Reclaimed aggregate
RCG	Recycled crushed glass
RMC	Ready-mix concrete
SCM	Supplementary cementitious material, e.g. fly ash, silica fume
SLN	Société Le Nickel – ferronickel smelting company and producer of FNS

Term	Description
VPV	Volume of permeable voids (AS 1012.21:1999)

2 Literature Review

2.1 Background

A 'recycled aggregate' was historically defined by the Australian Standard for concrete aggregates as 'crushed concrete composed of aggregate fragments coated with cement paste or cement mortar' (AS 2758.1:1998), although this definition was removed from the current version of this standard (AS 2758.1:2014). A broader definition used in the UK and the European Union is 'aggregate resulting from the processing of inorganic or mineral material previously used in construction' (BS EN 16236:2018) which includes aggregate obtained from demolished masonry. These definitions exclude the sourcing of aggregate from non-construction sources, such as recycled crushed glass and are therefore not all-encompassing.

In general, the term 'recycled aggregate' can be considered to cover any recycled material which is used as a coarse or fine aggregate material in newly produced concrete. This includes recycled construction materials, as well as other commercially and industrially produced recycled materials such as crushed glass, blast furnace slag and power station bottom ash. As indicated in Section 1.3, the review incorporated in this report is focused on:

- recycled concrete aggregate (RCA), produced from crushing demolished or returned hardened concrete
- recycled crushed glass (RCG), produced from crushing glass materials, generally bottles and containers
- reclaimed aggregate (RA), produced from rejected or waste batches of plastic concrete using a special admixture or processing to remove mortar
- ferronickel slag (FNS) produced as a by-product of nickel smelting
- power station bottom ash (PSBA) produced as a by-product of coal-fired power station operation.

2.2 Standards And Guidelines

AS 2758.1:2014 specifies the requirements for the properties for concrete aggregates in Australia. It nominates testing methods and acceptance criteria for aggregate properties, including bulk density, particle density and water absorption, particle size grading and shape, and durability related issues. Further to AS 2758:2014, MRTS70 (11/2018) sets out the specific requirements for concrete used by TMR, including aggregate properties and relevant test methods. Some of the aggregate requirements specified by MRTS70 (11/2018) are more stringent than those specified by AS 2758.1:2014, and there are also some differences in the test methods applied. These are discussed under the respective Mechanical Properties headings for each recycled aggregate considered in this review.

To be accepted for use in TMR, Normal (N) class concrete, a prospective aggregate material shall comply with the requirements of AS 2758.1:2014 (as specified by MRTS70 (11/2018)). This includes a requirement for testing of concrete aggregate to be undertaken using the specified test methods. Where a material does not comply with the requirements of AS 2758.1:2014, there may be potential to develop special requirements (i.e. specification clauses within the Normal Class concrete section of MRTS70) based on proven performance or other demonstrated benefits. Sufficient basis must exist to substantiate the inclusion of such clauses, and it may not be possible to conclusively adopt a certain approach based on a literature review alone.

The Australian Standard for concrete aggregate (AS 2758.1:2014) does not specify any special requirements for recycled aggregate materials, which suggests that any recycled aggregate material should be capable of meeting the general requirements for concrete aggregate specified by the standard. Similarly, the TMR standard MRTS70 (11/2018) does not specify any special requirements. The use of recycled aggregates is currently not allowed for Special (S) class concrete, but no specific guidance is provided for N-Class concrete.

2.3 Recycled Concrete Aggregate (RCA)

2.3.1 Overview of RCA and State of Practice

The use of recycled materials in the building construction sector in Australia commenced in the late 1990s. Research undertaken by the CSIRO (Sagoe-Crentsil et al. 1996; Sagoe-Crentsil, Brown & Taylor 2001) showed that RCA had the potential to be used as a partial replacement (up to 30%) for coarse virgin material in concrete production for non-structural work. As a result, a *Guide to the use of recycled concrete and masonry materials* (HB 155:2002), was developed to 'provide engineers with fundamental product specification information and the tools required for conventional design with graded recycled construction and demolition waste material'.

HB 155:2002 provides some guidance on the application of RCA, including a recommendation for a limit of 30% coarse aggregate RCA substitution in concrete production, a 1% by mass limit on inclusion of contaminants in RCA (bricks, metal, wood etc.) and a limit of 6% water absorption for coarse RCA used in concrete production. The guide states that fine RCA is not suitable for concrete production due to its high water demand which results in reductions in workability, strength and durability properties. These limits have been considered during the review of literature, and generally appear to be reasonable.

Despite the advice of the CSIRO, there appears to have been limited uptake of RCA for new concrete applications in Australia. Internet searching reveals little publicly available evidence of concrete applications in Australia, though there is more significant coverage of RCA use in non-concrete pavement (e.g. road base) and backfill applications (Tam, Soomro & Evangelista 2018). The industry survey conducted as part of the NACoE S51 Y1 project received limited responses, but it was reported that concrete producers are reluctant to adopt RCA due to the variable high water absorption characteristics of RCA which result in variable water demand, and a need for higher cement content to maintain target strengths (personal communication with Queensland concrete producer 2020). Water absorption impacts on the plastic properties of concrete including slump and pumpability, and while the impacts of higher absorptions can be managed for a consistent material, the variability of RCA introduces extra complexity which is undesirable for concrete producers.

Coarse recycled concrete aggregate (coarse RCA) consists of particles of natural aggregate (NA) used in the original concrete, the old mortar attached to the NA and other debris such as brick pieces (Shayan & Xu 2003). It is angular in shape and coarse in texture. For a commercial coarse RCA, the amount of other debris is generally controlled, i.e. limited to 1 to 2% by weight, but the amount of attached mortar varies (Cement, Concrete and Aggregates Australia 2008, Shayan & Xu 2003). The smaller-sized fraction of a crushed RCA contains a greater proportion of mortar and a lower proportion of unfractured stone than the larger size fractions (Berndt 2009). The coarse particles may be of original shape or fractured, and the attached mortar may be present in various forms, such as a few spots of old paste on a particle, a thin film of old paste covering most of the particle surface, or a layer of old mortar 2–3 mm thick (Shayan & Xu 2003).

The crushed concrete fines (i.e., fine RCA) may also be used in concrete to replace natural sand (fine NA), although this approach was discouraged by HB 155:2002. The fine RCA consists of crushed coarse aggregate, sand particles with adhering hardened cement paste, or lumps of old mortar, and some fine particles which resemble crusher dust. These fine particles should not be an issue if maintained at similar levels to what is accepted for fine or manufactured NA (shall not exceed 5% or 20% respectively), which may be controlled through adherence to the deleterious fines index or maintaining the proportion of material below 75 µm at acceptable levels based on AS 2758.1:2014.

In summary, RCA consists of old NA (coarse aggregate from crushed concrete or collected from dumped construction waste), mortar attached to aggregate from crushed concrete, mortar separated from aggregate from crushed concrete, brick, and small amounts of other debris. The density of RCA is lower than that of comparable NA, due to the mortar and other debris having a lower density than the virgin stone. The water absorption of RCA is also higher compared to NA. These characteristics are discussed further in Section 2.3.2.

A concern for concrete made with RCA is the uncertainty around the actual content of 'real aggregate' in it, and the quality of the mortar attached to the aggregate, which depends on the grade of the demolished concrete and what level of deterioration that the demolished concrete has undergone. Current recommended tests for particle density, water absorption and abrasion value or crushing value have partly revealed these two issues, although the relationships between these results are not definite. This is not likely to be a critical issue at lower replacement levels of coarse RCA (below 50%).

2.3.2 Mechanical Properties of RCA and Comparison to Standards

CCAA (2008) states that coarse RCA (produced in Australia) has an average density of 2440–2460 kg/m^3 and water absorption of 4.5–5.4%, and fine RCA may have density of about 2320 kg/m^3 and water absorption of up to 6.2%. It is not clear, however, how much mortar may be present in an RCA since it is produced by crushing and screening to separate particles according to size, but processing is not intended to remove the attached mortar. The content of mortar attached to crushed concrete varies with the particle size of the RCA produced. For example, Etxeberria et al. (2007) noted that there was 20% and 40% mortar for the particle fractions of 10/25 mm and 4/10 mm, respectively. Knaack and Kurama (2013) investigated RCA supplied from 16 sources, which reportedly had mortar contents ranging from 18.8% to 63.6% by weight (Knaack 2013).

The physical and mechanical properties of RCA reported in the literature vary over a larger range than that stated above, because the content of the attached mortar significantly varies, and the type of NA present in demolished concrete varies depending on the local resources, such as gravel, crushed rocks of various mineral compositions, as well as the variation in the strength grade and deterioration of the demolished concrete. Furthermore, some researchers suggested that the impact force during the crushing process of the demolished concrete would generate cracks and fissures in the aggregate, leading to changes in properties including higher permeation and water absorption (Olorunsogo & Padayachee 2002, Tavakoli & Soroushian 1996).

A comparison of the key aggregate properties outlined in the standards relevant to TMR practice (MRTS70 (11/2018) and AS 2758.1:2014) and the typical properties of RCA reported in the literature is presented in Table 2.1. It must be recognised that most limits recommended by MRTS70 (11/2018) do not apply to N-Class concrete and are therefore not directly relevant to the applications included in the scope of this study. In these cases, the limits are provided for comparison purposes only. Some of the notable differences between the standard limits and reported properties of RCA include:

- The water absorption of RCA is generally higher than the 2.5% limit recommended by MRTS70 (11/2018) for S-class concrete. AS 2758.1:2014 notes that 'higher absorption values may be acceptable based on local performance records. Water absorption of up to 6% is specified by HB 155:2002 to be acceptable for coarse RCA, which appears to have been based on the typical values recorded for RCA products available at the time, and the observed performance of RCA field trials using these products. Fine RCA generally has higher water absorption than coarse RCA due to the greater presence of crushed mortar particles in fine RCA.
    - Water absorption impacts on the water demand of concrete mixes, and if not accounted for in mix design may lead to reduction in concrete strength or durability. This is discussed in Section 2.3.6.
    - Water absorption of RCA can be variable within a batch, leading to issues associated with mix control in concrete production.
- The Los Angeles (LA) value (AS 1141.23:2009) of the RCA is high due to the presence of attached mortar, i.e. the LA value may not represent that of the natural aggregate in the RCA. This is likely to be less critical for aggregates which are not used in pavement applications, since this property is most critical for aggregates which are exposed to abrasion.

- Values for Micro-Deval loss in the literature are generally above the 15% limit specified by TMR for S-class concrete (up to 32%). The literature generally refers to this test as being most relevant for aggregates used in concrete pavement design for which abrasion and skid resistance are of concern (Wu et al. 2018), and advice received from TMR supports this assertion. Consequently, this property of RCA is of lower importance for non-structural concrete used in applications such as footpaths as per the scope of this study.
- Wet and dry strength testing is conducted to verify the durability of an aggregate and its strength for inclusion in concrete (as per AS 1141.22:2019). For an RCA, a lower wet-to-dry strength ratio may partly be because of the crushing of the old mortar which is weaker when saturated.
  - RCAs from Queensland suppliers showed a wet/dry strength variation within the MRTS70 (11/2018) limits, but the wet strength was slightly below the limit of ≥110 kN for wet strength (Latter 2021b).
- RCA derived from concrete components exposed to aggressive environments, such as seawater and sulphate-contaminated soil, may contain chlorides and sulphates, which should be determined through testing. Stricter quality controls on RCA sources are an option for limiting the inclusion of concrete components which have been subject to high chloride/sulphate exposure, but this may introduce extra costs. Since the primary use of RCA in QLD is in non-concrete fill/road base applications, these contaminants are not of concern and current quality control processes are reportedly minimal.
- There is potential for crushed RCA to contain higher proportions of particles finer than 75 µm than the limits specified by MRTS70 (11/2018). Further processing such as washing is likely to reduce the portion of very fine material in coarse RCA.

Table 2.1: Standard limits for aggregate properties vs. reported RCA properties

Property	AS 2758.1:2014	MRTS70 (11/2018)	HB 155:2002	RCA (Coarse)	RCA (Fine)
Wet strength	• ≥ 50 kN for exposure class A1, A2 • ≥ 80 kN for exposure class B1, B2 • ≥ 100 kN for exposure class C	≥ 110 kN		93-111 kN (data from 4 QLD suppliers) (7)	
Wet/dry strength variation	• ≤ 45% for exposure class A1, A2 • ≤ 35% for exposure class B1, B2 • ≤ 25% for exposure class C	≤35%		• 31–34%	
Weak particles	• ≤ 0.5% (Coarse aggregate) • Other limits of weak particles may be adopted based on demonstration of successful performance.	≤ 0.5%			N/A
Water absorption (%)	No hard limit. Recommended that aggregates with high absorption are pre-wetted prior to mixing.	≤ 2.5%	• Class 1A: ≤ 6% • Class 1B: ≤ 8%	• 4.7-5.6% (Australia) (1, 2) • 3.9-7.7% (Overseas) (3, 4, 5, 6)	• 6.3% (Australia) (2) • 13.1% (8)
Degradation factor (fine aggregate)	≥ 60	≥ 50		N/A	

Property	AS 2758.1:2014	MRTS70 (11/2018)	HB 155:2002	RCA (Coarse)	RCA (Fine)
Density	≥ 2.1, < 3.2 t/m^3 (normal weight aggregate)	≥ 2.1, < 3.2 t/m^3	• Class 1A: ≥ 2100 kg/m^3 • Class 1B: ≥ 1800 kg/m^3	• 2394-2557 kg/m^3 (Australia) (1, 2) • 2210-2360 kg/m^3 (Overseas) (3, 4, 5, 6)	2330 kg/m^3 (2)
Chloride content	Combined chloride salt content < 0.04% in reinforced concrete Total water-soluble chloride salt content < 0.03%	Report content as per AS 2758.1		0.0016% (9) Limited data available Will be dependent on exposure and environmental conditions for demolished concrete.	
Sulphate content	Sulphate content of concrete mix ≤ 5% by mass of Portland cement	Report content as per AS 2758.1		0.0025% (9) Limited data Will be dependent on exposure and environmental conditions for demolished concrete.	
Deleterious fines index	≤ 150	≤ 150		N/A	No data
Micro-Deval loss	Not referenced	≤ 15%		• 20% (100% RCA) • 15% (60% RCA) (10) • 32% (11)	
Soundness	6% (Exposure class C)	6%			
Organic impurities	Colour shall not be darker than standard colour of reference solution. Performance of suspect fine aggregate may be verified by comparing performance to satisfactory fine aggregate.	Negative			
Sugar content	Test negative to presence of sugar based on AS 1141.35	As per AS 2758.1		Negative (Queensland sources) (7)	
Light particles	≤ 1% (fine) ≤ 3% for vesicular aggregates	≤ 1% (fine)		No data	No data
Material finer than 75 µm	• Coarse: ≤ 2% • Fine: ≤ 20%	• Coarse: ≤ 2% • Fine: ≤ 20%		2.6-4.5% (7)	
Material finer than 2 µm	≤ 1%	≤ 1% (fine)		No data	No data
Flakiness index	≤ 35%	≤ 30%		• 7-19% (Overseas) (11) • 4-10% (QLD) (7)	N/A
Los Angeles value	Maximum percent loss is 35% and 30% for concrete exposure Class A and B, and C, respectively.	Not referenced.		29-35% (42% outlier)	N/A

Property	AS 2758.1:2014	MRTS70 (11/2018)	HB 155:2002	RCA (Coarse)	RCA (Fine)
ASR reactivity	Test in accordance with AS 1141.60.1:2014. Testing to AS 1141.60.2:2014 may also be conducted to determine aggregate classification. SCMs are to be used to mitigate reactivity.	Where aggregate identified as having potential for AAR in accordance with AS 2758.1, treatment shall be in accordance with SA HB79:2015. Applies to N-Class concrete.		Likely to demonstrate greater reactivity than original NA.	

Note: Australia: RCA is not specified in AS 2758.1:2014. HB 155 (2002) specifies: Class 1A – well-graded RCA with no more than 0.5% brick content, Class 1B – RCA mixed with no more than 30% of crushed brick.

Sources: 1. Sagoe-Crentsil, Brown and Taylor (2001), 2. Shayan and Xu (2003), 3. Eguchi et al. (2007), 4. Limbachiya, Leelawat and Dhir (2000), 5. Ulloa et al. (2013), 6. Thomas et al. (2013), 7. Latter (2021b), 8. Evangelista and de Brito (2007), 9. Tam, Tam, and Le (2007), 10. Đokić et al. (2020), 11. Joseph et al. (2015).

2.3.3 Cost

Industry engagement was conducted by Senaratne et al. (2017) as to perceptions of the use of RCA concrete. It was found that high initial cost was a major concern amongst the building consultants and engineers who were consulted. This was reportedly linked to the need for additional admixtures and cement inclusion in RCA concrete compared to 100% NA concrete. No relative costs were provided however, which makes it difficult to assess the validity of the concerns raised by the study, but these concerns are consistent with industry expectations.

Cost was highlighted as a primary concern of the Queensland concrete producer contacted through the industry engagement portion of the S51 Y1 project. Concerns were highlighted regarding perceived increased concrete costs with the use of RCA related to the need for increased cement content and lower water/cement (w/c) ratios to meet target strengths relative to concrete manufactured with NA.

Wijayasundara et al. (2016) conducted a review of the economic viability of producing coarse RCA concrete in Australian ready-mix concrete (RMC) plants. It was noted that the business structure of RMC plants is also a likely contributor to the low uptake of RCA concrete, since the RMC industry generally achieves a lower profit margin (e.g. gross industry margin of 6.5% in 2013/14) compared to aggregate and cement producers. Vertical integration of concrete producers with hard rock quarries is another barrier to coarse RCA inclusion, since the business is founded upon purchase of aggregate product from the integrated quarry entity (Wijayasundara et al. 2016).

It was found that the largest additional cost associated with coarse RCA material supply was associated with pre-processing to remove reinforcement, crushed fines and sieving to achieve well-graded material (Wijayasundara et al. 2016). These costs were estimated based on the experience of a coarse RCA road base supplier. Costs associated with handling, quality control, and maintaining additional material stockpiles were also considered. A total additional cost of \$2.72/m^3 for supply of coarse RCA was estimated, compared to natural coarse aggregate.

The approach taken by Wijayasundara et al. (2016) assumed that additional cementitious material would be required to meet target concrete strength and durability properties. A 1% increase in the cost of the final product was estimated for each 6% increase in cement content. It was observed that the addition of fly ash and ground granulated blast furnace slag (GGBFS) in place of some additional Portland cement was more cost effective (a reduction of \$2.30/m^3 compared to use of additional cement at 30% coarse RCA replacement of NA), though a direct comparison with 100% NA concrete was not included (Wijayasundara, Mendis & Crawford 2018).

Following on from the previous study which focused on the production of RCA and RCA concrete, Wijayasundara, Mendis & Crawford (2018) conducted an economic analysis of the use of RCA concrete in residential structural applications in Australia. It was concluded that producing concrete using RCA would not be a financially attractive option for the concrete industry based on current material costs and practices, which would have a flow-on effect on the feasibility of RCA concrete in construction. It was highlighted that an indirect economic benefit could be realised if the environmental and social benefits of using RCA concrete are considered and allocated to the cost of RCA.

It appears that the use of RCA in concrete is unlikely to be economically attractive where more cost-effective NA sources exist. RCA requires additional processing compared to NA, which results in a more costly aggregate product. RCA also introduces additional costs associated with logistics and transport, and requires additional storage capacity at concrete plants, though these factors will generally apply to all recycled aggregate products.

The need for additional cement to meet target concrete properties may increase costs further. It appears likely that the use of fly ash and GGBFS in concrete could partially account for the volume of additional cementitious materials required, though some additional Portland cement is likely to be required. Further consultation with aggregate suppliers and concrete producers is recommended to determine the current economic scenario and refine understanding of RCA viability from an economic standpoint.

2.3.4 Availability

RCA is primarily used in non-concrete applications as a road base and pavement material in Australia and it is currently approved for use in this capacity by VicRoads and TMR (e.g., MRTS05 (07/21), VicRoads 812:2016). There are several suppliers of RCA in Queensland, which were detailed by the NACoE P94 project, though it was noted by this project that there are a limited number of suppliers compared to quarry sources, with most RCA suppliers concentrated in South-East Queensland (SEQ) (Latter 2021b). This may mean that the production of concrete using RCA will be less financially viable in areas away from SEQ.

Advice received as part of an external review of the S51 Year 1 draft report (unpublished) indicated that RCA from demolished sources was not readily available in SEQ for concrete production, and the material that was available was generally used for construction fill and drainage applications which required relatively less processing compared to that required for concrete applications. Demand for RCA in applications other than the production of new concrete outstrips supply, and there was reportedly limited interest shown by waste operators in supplying RCA for new concrete applications.

Further consultation with Queensland suppliers of RCA with the aim of determining what volume of material is available for concrete production and whether there is a reliable supply of the material available may be beneficial but based on the external review of the S51 Y1 draft report this approach may not yield positive outcomes. Consideration should be given to the quality of the available material and whether it will meet requirements for limits on contaminants such as brick and metals, and other parameters including water absorption and density. The availability of RCA which is suitable to produce new concrete appears to be a limiting factor on the viability of pursuing RCA concrete as a recycled aggregate source for new concrete production.

2.3.5 Performance of RCA Concrete

RCA is different to natural aggregate in that hardened cement mortar attached to the surface of aggregate particles results in a lower density and higher porosity. The former is due to the lower volume of NA in the RCA which consequently leads to a lower modulus of elasticity and reduced strength of the concrete.

The focus of investigations into the influence of RCA on concrete performance has been on compressive strength, tensile strength, flexural strength, modulus of elasticity, shrinkage, creep, and durability issues and the structural behaviour of the reinforced concrete made with RCA. The main research topics include the quantity of the RCA as a replacement of natural aggregate, the concrete strength grade (represented by the quantity of cement and the water to cement ratio (w/c) in the concrete mix), and the inclusion of supplementary cementitious materials (SCMs) as binder to improve concrete properties.

Fresh properties

The workability of RCA concrete is heavily influenced by the presence of old mortar in coarse and fine RCA, which results in higher water absorption. This leads to a need to increase the water content of RCA concrete to maintain target workability for a given mix design (Etxeberria et al. 2007, Kurda, de Brito & Silvestre 2017). The shape and texture of RCA will also impact on workability and concrete crushing processes should be designed to create particles which comply with particle shape requirements such as the flakiness index (MRTS70 (11/2018).

Etxeberria et al. (2007) note that the inclusion of coarse RCA in a mix will typically need 5% more water to be included in a mix to achieve comparable workability with a 100% NA concrete. Given that fine RCA generally exhibits higher water absorption than coarse RCA, it may also have a higher water demand, which will also vary with material shape and grading. Kisku et al. (2017) note that the inclusion of up to 20% of fine RCA in concrete will have no major effect on the workability of concrete, but higher inclusions will have a significant impact.

The heterogenous nature of typical mixed-source RCA means that there may be significant variability in aggregate properties from one batch to another. This is evidenced by the variability in key properties such as water absorption and flakiness which was discussed in Section 2.3.5. This makes it more difficult to implement a consistent mix design, as an overestimation of aggregate water absorption could lead to an overinflated w/c ratio, while an underestimation could result in a loss of workability which requires additional input of superplasticiser to address. The assumption of high RCA absorption would also require higher cement input, leading to more costly concrete mixes (Joseph et al. 2015).

Aggregate pre-wetting has been suggested as a mitigation for the higher water absorption of RCA (Joseph et al. 2015). Current industry practice is to pre-saturate coarse aggregates before mixing, which may be an effective method for RCA, but the high water absorption may also lead to higher early concrete shrinkage as the aggregate loses water (Xiao, Lu & Ying 2013).

If RCA is unsaturated, the continuing water absorption of the RCA will likely cause slump loss. Yang, Chung and Ashour (2008) showed that concrete containing a high volume of RCA and/or high absorption RCA exhibited significantly more slump loss. The slump decreases with the increase in total water absorption of aggregate mixes. It is likely that this slump loss can be managed through use of superplasticisers, although variability will still result due to the variation in water absorption of the RCA. Testing is recommended to determine appropriate levels of superplasticiser inclusion, and whether increased dose rates (compared to standard practice) are required.

Workability is generally managed within appropriate limits by using superplasticiser admixtures (Matias et al. 2014; Wijayasundara, Mendis, & Crawford 2018, Rao, Bhattacharyya, & Barai 2011,). One study showed that the inclusion of superplasticiser as 0.5% of cement content would result in slumps within ±10 mm of the target slump (80 mm) for concrete at coarse RCA replacement levels up to 100% (Matias et al. 2014). This study also notes that the addition of superplasticiser may enhance the compactness of a mix, improving strength properties. It can be concluded that the use of superplasticisers is standard practice for concrete production, but it is possible that higher dose rates will be required for concrete produced using RCA.

Strength properties

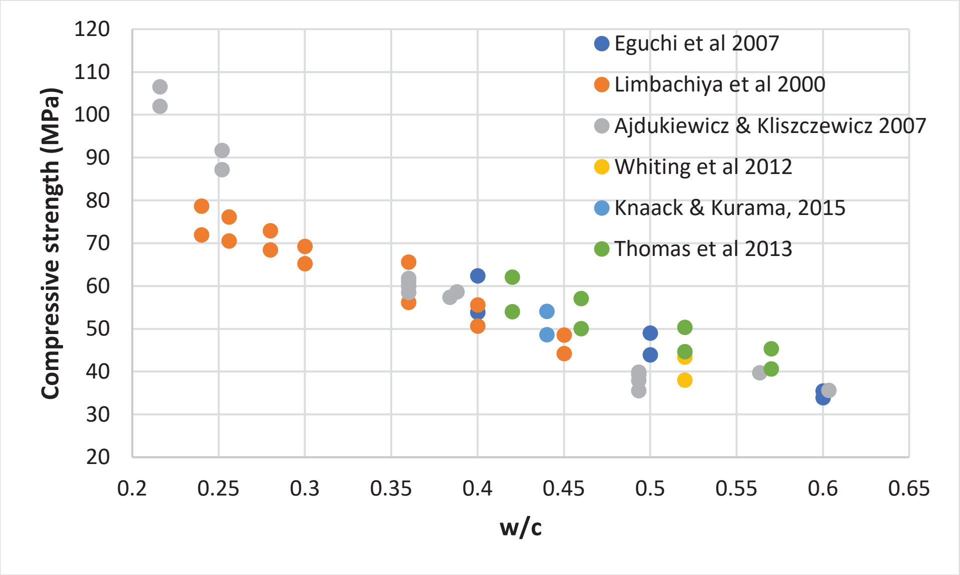
The relatively low content of NA in RCA concrete and the fact that the RCA has higher water absorption than NA, leads to the conclusion that concrete made by replacing NA with RCA would have a lower resistance against stress, i.e. a lower compressive strength and modulus of elasticity for concrete with the same total cementitious content and the same water to cement ratio. The development of strength with concrete age, which depends on the hydration of the cementitious materials in concrete, would be similar for both the NA and RCA concrete based on the same strength grade.

A key factor influencing concrete strength is the mass ratio of water to cement (w/c ratio). The experimental data reported in several papers representing concrete of high to moderate grade indicates that the dependence of the compressive strength on a w/c ratio of 100% RCA concrete is largely the same as that of the control concrete containing no RCA, though there are discrepancies at very low w/c, i.e. below 0.30 (Figure 2.1).

Another key influence on the strength and stress resistance properties of concrete is the development of the bond between aggregate and the cement paste. This can also be referred to as the strength of the interfacial transfer zone (ITZ) between the aggregate and cement paste (Scrivener, Crumbie, & Laugesen 2004). The presence of old mortar in coarse RCA has been seen to result in a weaker ITZ in coarse RCA concrete compared to 100% NA concrete which is influenced by the water demand of the old mortar (Kisku et al. 2017). This effect contributes to the relatively lower compressive strength which is generally observed for concrete produced using coarse RCA (Kisku et al. 2017).

Figure 2.1 demonstrates that, if the NA was replaced with 100% coarse RCA, the compressive strength of the resultant concrete would reduce by 10%, or that, in order to obtain the same strength level, the w/c ratio of the coarse RCA concrete would need to be lowered by around 0.03 to 0.05 points.

Figure 2.1: Relationship between compressive strength and the water to cement ratio (w/c)

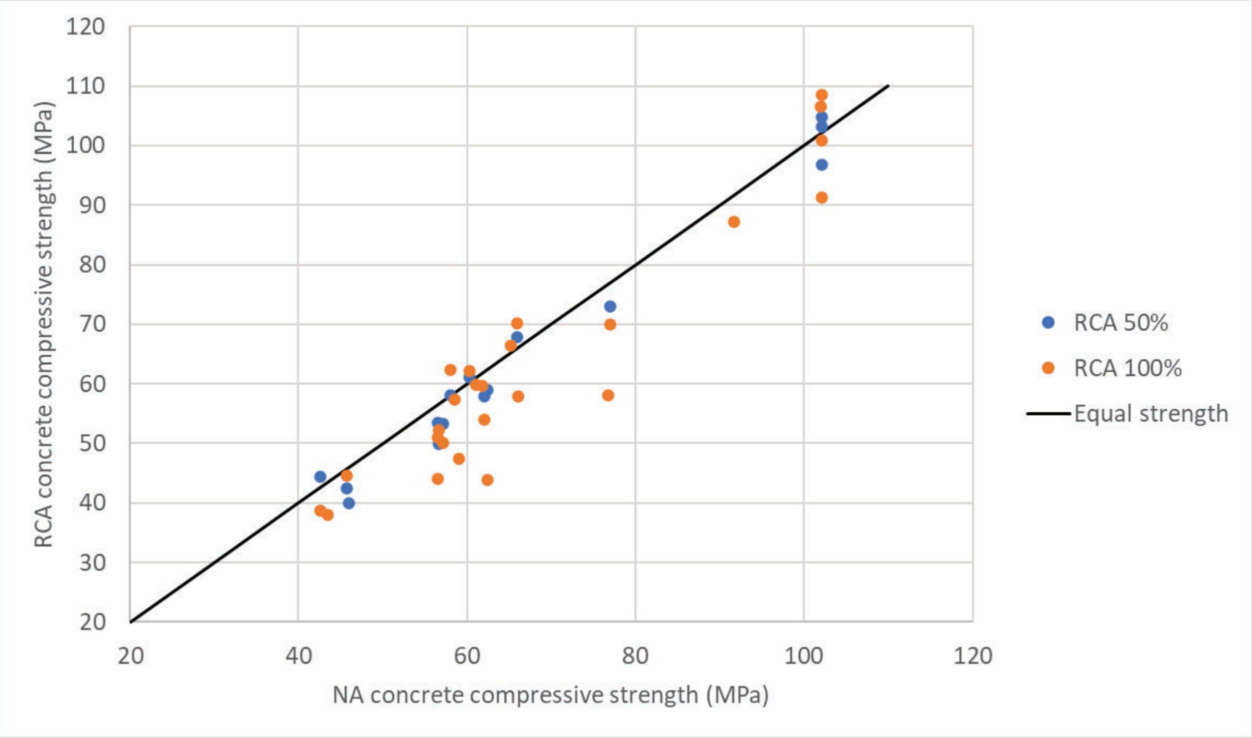


Note: Two identically coloured markers for each w/c: the upper marker and lower markers represent the control concrete and 100% RCA concrete, respectively.

Research results have demonstrated that replacing NA by coarse RCA, up to a level of 30%, would not adversely affect the performance of higher-grade concrete. Limbachiya, Leelawat, and Dhir (2000), who investigated concretes with design grade of 50 to 70 MPa, demonstrated that up to 30% coarse RCA would have no effect on concrete strength; thereafter, a gradual reduction in strength would occur as the RCA content increased. Similar results were returned by Thomas et al. (2013) who studied the strength of RCA concrete as influenced by its w/c ratio, RCA to NA replacement ratio and the age for concrete strength grades between 25 and 50 MPa. At replacement levels greater than 20% (50% was the next highest level tested), the compressive strength of the RCA concrete was lower than that of the control concrete, especially for higher-grade concrete (within the range of the study).

Figure 2.2 shows the compressive strength of high-volume RCA concrete (50%, 100%) and that of the control concrete reported in several research papers. Without adjustment of total cementitious materials content in concrete, the concrete made by replacing NA with 100% RCA showed a reduction in concrete strength by 5–15%. For characteristic strengths above 60 MPa, the loss is less than 10%, for characteristic strengths below 40 MPa, the loss grows to around 15%. The strength loss for 50% RCA concrete is less than 10% for most of the results shown in this Figure.

Figure 2.2: Compressive strength of structural concrete containing high volumes of RCA compared with the control concrete



Sources: Ajdukiewicz and Kliszczewicz (2007), Andreu and Miren (2014), Brandes and Kurama (2019), Eguchi et al. (2007), Etxeberria et al. (2007), Knaack and Kurama (2015), Kurda, de Brito and Silvestre (2019), Limbachiya et al. (2000), Shaikh and Nguyen (2013), Shayan and Xu (2003), Thomas et al. (2013), Whiting, McCarthy and Lume (2012).

The variation in the strength and other properties of coarse RCA concrete can be partly attributed to the properties of the demolished concrete, including the strength grade, aggregates used (type of rock, particle size and texture) and extent of deterioration. These factors will generally vary between different experimental studies on RCA, which leads to the variance of results seen in the literature for strength and durability properties. Depending on the sources of commercial coarse RCA, similar variance and heterogeneity may exist within batches of RCA and between different batches.

Investigating the strengths of concrete made from coarse RCAs obtained from field-demolished concrete, Tavakoli and Soroushian (1996) produced concrete that achieved a similar strength compared to the original concrete (without changing the cement content and w/c). It was noted that knowing the properties of the original concrete would help to produce RCA concrete with realistically targeted quality. Several other investigations on the relationship between the old (demolished) concrete and new (RCA) concrete reached the same conclusion (Ajdukiewicz & Kliszczewicz 2007; Hansen & Narud 1983;; Tabsh & Abdelfatah 2009).

Several researchers also tested the influence of fine RCA replacing sand on concrete properties. In general, fine RCA has a high content of old cement paste which has a higher angularity than NA sand as well as a higher water absorption. Slightly lower compressive strength for RCA concrete containing a high volume of fine RCA (50% or higher) was reported by Ajdukiewicz and Kliszczewicz (2007).and Shayan and Xu (2003) Evangelista and de Brito (2007) showed that up to 30% of fine RCA (water absorption 13.1%) replacing sand would not jeopardise concrete strength. Their data indicated that, by further increasing the replacement level, the adverse effect was still not great, e.g. at 100%, the reduction in strength was about 8%. It was suggested that the water saturation degree of the RCA was an influencing factor because the water at the surface of the

RCA would weaken the bond between the cement and RCA. This level of water absorption may also compromise concrete durability, potentially worsening transport properties as discussed in Section 2.3.6.

The flexural strength and tensile strength of concrete have been reported to be impacted to a lesser degree by RCA in some circumstances, though several studies have reported a decreasing trend with increasing RCA content (Kisku et al. 2017). Some experimental results did however show that RCA concrete had a higher splitting tensile strength and flexural strength for the same compressive strength (Etxeberria et al. 2007; Tavakoli & Soroushian 1996). It generally appears that the expected relationships between compressive strength and tensile/flexural strength (as per AS 3600:2018) are not voided by the inclusion of coarse RCA, meaning that coarse RCA concrete may be suitable for structural use in some capacity, but this is beyond the scope of this project.

Limbachiya, Leelawat, and Dhir (2000) showed that the inclusion of coarse RCA at up to a 100% replacement level had no appreciable impact on flexural strength for concretes ranging from 50–70 MPa target compressive strength. Similar results were obtained by Ahmed (2012), who found that flexural strength development in 50% coarse RCA concrete was initially below that of the reference concrete, but reached parity with the reference concrete at 91 days. Flexural strength results obtained by Rao, Bhattacharyya, and Barai (2011) showed a slight decrease in flexural strength at 100% coarse RCA replacement compared to the control mix (4.97 vs. 5.23 MPa), though this was not considered significant by the authors.

Considering tensile strength, Ahmed (2012) found that 50% coarse RCA concrete produced a slightly reduced tensile strength at 91 days compared to the control (4.3 vs. 4.8 MPa), though the 25% RCA mix produced a higher tensile strength at 5.1 MPa. Similarly, a small reduction in split tensile strength for 100% RCA concrete compared to control concrete (2.03 MPa vs. 2.67 MPa) was observed by Rao, Bhattacharyya, and Barai (2011). Similar tensile strength results for the reference mix and 100% RCA mixes were also observed by Manzi, Mazzotti, and Bignozzi (2013), with all mixes producing results in the range of 3–4 MPa (0.48 w/c ratio, 350 kg/m^3 cement content).

The experimental data and conclusions of published research on the strength properties of RCA concrete can be summarised as follows:

- The quality of RCA, in terms of the original concrete grade (and condition state), plays an important role; RCA derived from higher-grade concrete with no significant deterioration would produce higher-strength RCA concrete, and vice versa.
- Replacing NA with up to 30% coarse RCA (mass to mass) for concrete with strength grades up to 40 MPa would not adversely affect concrete compressive strength. This conclusion also applies for concrete strength grades greater than 50 MPa.
- In most cases, compressive strength reduction due to the inclusion of 50% coarse RCA would be lower than 10%, compared to the strength of an equivalent 100% NA concrete (same w/c ratio, cementitious contents). It is likely that lesser reductions in strength could be achieved if coarse RCA is sourced from high-grade concrete (e.g. 50 MPa and higher), but the source of RCA would need to be tightly controlled to achieve this in practice.
- At coarse RCA replacement levels higher than 50%, the concrete compressive strength will be relatively lower compared to an equivalent concrete produced using 100% NA; at a replacement level of 100%, a 10% to 15% compressive strength reduction would be expected.
- At replacement levels up to 100% of coarse or fine RCA, small reductions in flexural and tensile strength may be experienced, although several studies have shown insignificant reductions or even improvements for these properties where RCA is used.

Deformation properties – modulus of elasticity

Silva, de Brito and Dhir (2016) conducted a literature review on the influence of coarse and fine RCA on the modulus of elasticity and strength of concrete based on studies conducted between 1973 and 2015. It was found that the main parameters which impacted on the strength and deformation properties of RCA concrete were the RCA replacement level, the quality of the original material, the mixing procedure and the incorporation of supplementary cementitious materials. In terms of RCA-NA replacement ratio, it was found that up to 30% inclusion of coarse RCA would not significantly influence the modulus of elasticity but at 100% coarse RCA the modulus of elasticity may fall by as much as 20% to 40%. It was also found that fine RCA has a relatively more deleterious effect on the elastic modulus compared to coarse RCA, due to the higher mortar content of fine RCA (which is more susceptible to deformation). The development of the modulus of elasticity of RCA concrete was considered as parallel to that of NA concrete. Thomas et al. (2013) also concluded that the influence of RCA on the modulus of elasticity was significantly higher than that on the compressive strength.

The relationship between the modulus of elasticity and compressive strength of concretes (with coarse RCA content 0–100%) was reviewed based on experimental results reported by Ajdukiewicz and Kliszczewicz (2007), Andreu and Miren (2014), Brandes and Kurama (2019), Eguchi et al. (2007), Etxeberria et al. (2007), and Knaack and Kurama (2015). The modulus of elasticity generally increased with an increase in the compressive strength (which is to be expected), and the ratio of modulus of elasticity to the strength of concrete was somewhat lower for the RCA concrete. There was a large scatter in the data, though this is not uncommon for the estimation of the concrete elastic modulus. The data reviewed demonstrates that the variation between results obtained by different studies is significant compared to the relative variation between properties due to the incorporation of RCA at different replacement levels. This phenomenon can be attributed to the differences in the type (i.e. mineral or rock) of the original aggregate, the original concrete strength grade of the RCA, and the quality of the RCA determined by properties such as the amount of adhered mortar and percentage of deleterious materials.

The quality of RCA, particularly the amount of mortar attached to the aggregate (which determines the effective quantity of NA in the RCA concrete), plays an important role in the modulus of elasticity. For example, the RCA used by Eguchi et al. (2007) had a high recovery ratio, i.e. it contained a low quantity of mortar, and the concrete produced from it showed a low ratio of modulus of elasticity to strength (i.e. a close relationship). Some researchers considered that the contribution of the fraction of the adhering mortar to resist stress could be taken as zero, as expressed in the equations for calculating concrete creep proposed by Fan, Xiao and Tam (2013).

The approach of considering old mortar to have no influence on modulus of elasticity may underestimate the elastic modulus of RCA concrete if the original concrete contained a strong mortar component. This was evidenced by the experimental results obtained by Andreu and Miren (2014) who found that coarse RCA from recycled concretes with higher strength grades resulted in a higher modulus of elasticity when used as a 100% coarse aggregate replacement in new RCA concrete. The use of 100 MPa crushed concrete as coarse RCA resulted in an RCA concrete elastic modulus of 91% of that achieved for the reference concrete, compared with 74% of the reference concrete elastic modulus for the RCA concrete when 40 MPa crushed concrete was used as coarse RCA.

Experimental data on using fine RCA to replace sand (Ajdukiewicz & Kliszczewicz 2007; Evangelista & de Brito 2007) showed that, at a high content of fine RCA, the modulus of elasticity of concrete reduced. It has been reported that fine RCA has a greater impact on the modulus of elasticity of concrete compared to coarse RCA. This is thought to be due to the higher content of old mortar in the fine fraction of RCA, which has a lower capacity to resist deformation compared to natural sand (Silva, De Brito, & Dhir 2016). As for coarse RCA, studies have shown that at replacement levels up to 30% the impact of fine RCA on modulus of elasticity is insignificant (Evangelista & de Brito 2007; Silva, De Brito, & Dhir 2016)

There are three key factors influencing the modulus of elasticity of concrete: the modulus of elasticity of the original NA (depending on rock type) of the RCA particles, the volume content of the attached mortar in the RCA, and the modulus of elasticity of the mortar attached to the RCA particles (depending on the w/c ratio of the original concrete). However, the mortar content of the RCA was rarely reported in the published literature.

Deformation properties – shrinkage

Both shrinkage and creep are measures of the deformation of concrete with time. These properties are determined at a constant temperature condition, e.g. 23 °C, as set out in AS 1012.13:2015 and AS 1012.16:1996, respectively. Considering shrinkage, there are considered to be three main types (autogenous, plastic, and drying), but since drying shrinkage generally dominates the overall shrinkage experienced in ordinary-strength concrete (less than 60 MPa strength) (Šahinagić-Isović, Markovski, & Ćećez 2012), it has formed the focus of most RCA shrinkage research, and subsequently forms the focus of the review presented in this report. In general, the drying shrinkage of concrete made with coarse RCA was higher compared with that of NA concrete under the same drying conditions (Fan et al. 2013, Kurda et al. 2019; Limbachiya et al. 2000), which is discussed in this section.

Drying shrinkage is generally considered to be restricted by coarse aggregates in concrete, and it has been postulated that aggregates with a higher elastic modulus have greater ability to resist shrinkage (Zhang, Zakaria, & Hama 2013). Furthermore, the loss of water from RCA which generally has high water absorption may also contribute to shrinkage, though this effect is likely to be small compared to the shrinkage of the cement matrix (Xiao, Lu, & Ying 2013). The relatively low elastic modulus of RCA (due to the old mortar content) may have a more significant impact on shrinkage, however.

For quality control, VicRoads specifies that the concrete shrinkage measured according to AS 1012.13 is not to exceed 500 micro-strain (με) and 750 με after 21 days and 56 days of drying respectively for concrete of less than 60 MPa strength (VicRoads 610.07:2020). There is no such requirement in Queensland, but TMR advice indicates that shrinkage measured in practice is typically well below the VicRoads specified limits. In practice, drying shrinkage will occur until the moisture in the concrete pore structure is in equilibrium with that of the environment.

Sagoe-Crentsil et al. (2001) observed that the drying shrinkage of RCA concrete (25 MPa grade) made with commercially-graded RCA followed the same trend as the NA concrete, i.e. it stabilised after 91 days of drying. Their experimental data demonstrated that the drying shrinkage of concrete (25 MPa grade) made with 100% commercially-graded RCA (~700 με) was about 30% higher than that of the NA concrete at 56 days (~500 με) which is considered as in agreement to the conclusion made by other researchers. Similarly, Eguchi et al. (2007), who studied the properties of RCA concretes with 28-day compressive strengths ranging from 32 to 62 MPa, showed that the shrinkage of concretes made from 100% RCA was about 20–30% higher than that of their NA counterparts. Shayan and Xu (2003) reported that, compared with NA concrete (50 MPa grade), the shrinkage of concrete made from 100% coarse RCA increased by 40%, but remained within the AS 3600 limits at 56 days at 0.075%. Ahmed (2012) reported that the 56-day shrinkage (AS 1012.13) of 50% coarse RCA concrete was ~600 με compared to ~420 με for 25% coarse RCA and ~320 με for the reference 100% NA mix.

The experimental results by Babu et al. (2015) on concrete (75 MPa grade) showed that the shrinkage of concrete with up to 30% RCA was effectively the same as that of the control concrete, but a further increase in RCA content (increments of 10% up to 100%), led to a steady increase in the shrinkage such that the 100% RCA concrete doubled the shrinkage compared to the control. Similarly, Kurda et al. (2019) showed the shrinkage of concrete (50 to 70 MPa grade) with 100% coarse RCA (fine NA sand) (~320 μ/m at 56 days) was 1.5 to 2.2 times that of the NA concrete (~180 μ/m at 56 days), and slightly more if the sand in the concrete was replaced with fine RCA (natural coarse aggregate) (~350 μ/m at 56 days). Kurda, de Brito, and Silvestre (2019) attributed the higher shrinkage of RCA concrete to the high water absorption of the old mortar component, and subsequent higher water demand of the RCA concrete.

The use of fly ash as an SCM has been reported to be effective to mitigate increased shrinkage in RCA concrete, with Kou et al. (2007) reporting that 25–35% inclusion of fly ash by weight of cement decreased the drying shrinkage and creep of RCA concrete at replacement levels up to 100%. This conclusion was also made by Limbachiya, Meddah, and Ouchagour (2012) who stated that 30% replacement of cement by weight with fly ash was sufficient to reduce shrinkage to levels similar to that observed for the control concrete. These results indicate that the standard TMR practice of incorporating fly ash in concrete would likely limit the shrinkage of concrete produced using coarse or fine RCA, although some increases in shrinkage can be expected relative to standard 100% NA TMR concrete which incorporates fly ash.

As discussed in this section, the replacement of NA with RCA leads to increased shrinkage where no additional mitigations are applied. The reported shrinkage value of RCA in comparison with NA concrete varies significantly, especially for high replacement percentages. For example, the shrinkage increase ranged between 40% to 200% for 100% coarse or fine RCA–NA replacement. For studies where shrinkage values were included, it was shown that drying shrinkage of concrete produced using coarse NA at up to 100% replacement produced 56-day shrinkage values close to the VicRoads 610.07:2020 limit of 750 με. In practice, this increased shrinkage could potentially be controlled using increased reinforcement, lower w/c ratios, or the creation of more tightly spaced control joints

Most of the research work indicates that up to 30% replacement of coarse or fine RCA would not induce any significant increase in shrinkage. Since shrinkage cracking is undesirable for concrete applications where the final appearance of the concrete is important, use of coarse or fine RCA beyond a 30% replacement ratio is not recommended in these scenarios unless an appropriate controlling method can be implemented.

Deformation properties – creep

Creep is the time-dependent deformation of concrete under sustained load which occurs simultaneously with shrinkage. A basic premise behind concrete creep is that, under sustained load, the calcium silicate hydrate (C-S-H) making up the bulk of hydrated cement matrix undergoes polymerisation (Hewlett 2003), and some microcracking occurs during this process. It has also been suggested that internal water migration under sustained load may also be responsible for concrete creep (Lopez, Kahn & Kurtis 2008).

Compared with NA concrete, RCA concrete has a lower portion of rock aggregate due to the inclusion of old mortar which may contribute to higher creep in service. It has been observed that the creep of RCA concrete is significantly higher than that of NA concrete (Fan et al. 2013; Limbachiya et al. 2000).

Knaack and Kurama (2015) studied the shrinkage and creep of normal-strength concrete (about 45 MPa) with 50% and 100% NA replaced by RCA. Both the creep and shrinkage strains increased with the increases in the replacement ratio, e.g. compared with NA concrete the relative strains after 7.5 months were 1.31 and 1.61 for the replacement ratios of 50% and 100% respectively. Much larger creep of RCA concrete relative to NA concrete was also reported. For example, the experimental results of Fan et al. (2013) showed that the relative creep (after about 200 days) of the RCA concrete (cement 417 kg/m^3, w/c 0.49 for the control concrete) increased by 30%, 77% and 100% with RCA–NA replacement of 33%, 66% and 100%, respectively. The scattering of the data is most likely to be due to the quality of the RCA in terms of amount of the attached mortar as well as the stiffness of the natural aggregate in the RCA particles.

Lye et al. (2016) reviewed the literature published from 1984 to 2014 and summarised the published data for establishing the creep of concrete composed of coarse RCA and recycled masonry aggregate (RMA), and with characteristic strength range of 14.5–63 MPa. The findings demonstrated that the average creep coefficient increased by about 15% as the content of RCA in the total aggregate increased from 0 to 30%, but it only further increased by 15% as the RAC content increased from 30% to 100%, and the relative creep coefficient of 100% RCA concrete was about 31% ±10%. The creep coefficient of 100% RCA concrete reported by Knaack and Kurama (2015) was about 20%, which was within the range of the findings of Lye et al. (2016).

In summary, the creep of concrete made with RCA replacing NA increased with an increase in replacement level; however, the relationship was not linear, being less advanced at high replacement levels. As for the shrinkage, the reported data for creep varied over a very large range. The quality of the RCA was the main reason for the increased creep in RCA concrete. In addition, the strength grade of the original concrete in the RCA, which determines the creep of the old mortar attached to the RCA during the process, may play an important role. While creep does not undergo significant increases past a certain replacement level, the high levels of creep shown by RCA concrete at the initial low replacement levels (up to 30%) indicates that RCA concrete may not be suitable for structural applications. Creep is likely to be of low concern for most non-structural applications, however, so further works to investigate mitigation of creep in RCA concrete are only recommended if the use of RCA in structural concrete is thought to be desirable.

2.3.6 Durability of RCA Concrete

Overview

Deterioration of concrete can occur when the material is exposed to an aggressive environment, such as sulphate and acidic soil, exposure to physical abrasion which could be due to the brushing effect of debris carried in water, or due to the internal chemical reactions such as alkali-aggregate reaction (AAR). Among these, the sulphate resistance depends mainly on the cement composition (Xu, Shayan & Baburamani 1998). Since it is not a property related to the aggregate it will not be discussed further in this report.

The main durability concerns for concrete are related to steel reinforcement corrosion, degradation of the cement mortar matrix and chemical expansion of the concrete. Reinforcement corrosion is related to the pore structure of the concrete matrix and changes in the electrochemical condition of concrete due to exposure to environmental conditions. The main durability controls are achieved by determining chloride ion penetrability or chloride diffusion coefficient, the rate of water absorption or sorptivity, air permeability, the rate of carbonation, and abrasion. These are related to the porosity of the cement paste and the cement-aggregate interface as well as the quality of the hardened cement matrix. Chemical expansion of the concrete will generally be related to alkali-silica reaction (ASR) in Australian concrete, which is primarily influenced by the reactive potential of the coarse and fine aggregates used in concrete. ASR may be controlled by limiting the alkalis available in the cement. The current TMR MRTS70 (11/2018) specifies that SCMs (fly ash, blast furnace slag, amorphous silica) are to be used in conjunction with low-alkali cement (maximum 0.6% NA_2O equivalent content) to mitigate ASR reactivity in produced concrete.

In general, RCA concrete has shown poorer behaviour for the transport properties (i.e. gas permeability, ion diffusion coefficient) compared to NA concrete. This is believed to be due to the higher water absorption and permeability provided by the old mortar attached to the RCA, e.g. chloride diffusion and carbonation rate of RCA concrete is higher than that of NA concrete (Limbachiya, Meddah & Ouchagour 2012; Olorunsogo and Padayachee 2002; Xiao, Lu & Ying 2013) suggested that cracks and fissures may be created in RCA particles during crushing, which would allow relatively easy gas permeation, ion diffusion and water absorption in the RCA concrete.

The following subsections are focused on ion diffusion with reaction (chloride diffusion), gas permeability and water penetrability, and gas permeation with chemical reaction (carbonation) and ASR.

Permeation

The permeability of RCA concrete has been shown to increase with increasing levels of RCA addition, which can be attributed to the porosity of the RCA including the attached old mortar (Amorim, de Brito, & Evangelista 2012). This is evidenced by increases in water absorption of concrete which incorporates RCA, linked to the higher water absorption of the RCA compared to NA.

Thomas et al. (2013) determined water absorption rate, carbonation rate (BS EN 13295:2004), water penetration depth under pressure (BS EN 12390-8:2009) and oxygen gas permeability, as well as the strength of concretes made with RCA and NA. It was concluded that the durability of the concretes made with coarse RCA, particularly at replacement levels above 50%, was worse than that of the NA concretes made with the same w/c ratio; however, the difference between the RCA and the NA concretes decreased for higher-grade concretes with lower w/c ratios. This indicates that the high porosity of RCA is counteracted by the reduction of available water in low w/c concretes. Shaikh and Nguyen (2013) showed that the water absorption and sorptivity rate of the coarse RCA concrete (40 MPa grade) was high, particularly for the concrete containing 50% RCA, but all mixes (0, 25, 50% coarse RCA) remained below the sorptivity limit of 0.2 mm/min proposed by Papworth and Grace (1985).

In a review by Xiao et al. (2013), the water penetration depth under pressure and gas permeability (determined by oxygen) of concrete increased exponentially with the increase in w/c ratio (or decrease in concrete compressive strength) and increased significantly with increases in RCA content. However, at w/c's below 0.50 or compressive strengths above 50 MPa, the RCA concrete at replacement levels up to 100% RCA showed only marginal adverse effects on these properties.

Amorim et al. (2012) tested the water absorption rate of concrete (28-day strength 50 MPa) containing 0–100% RCA replacing NA and cured at various conditions (outdoor, laboratory dry, laboratory humid, water). The results demonstrated that the water absorption rate of the RCA concrete was high compared to the NA concrete, with the worst case occurring for the 100% RCA concrete cured under outdoor conditions, being 57% higher than its NA counterpart. Experimental results by Evangelista and de Brito (2010), who examined the influence of replacing natural sand by fine RCA on concrete durability, showed that the water absorption rate increased by 34% and 70% at the fine RCA-sand replacement ratios of 30% and 100% respectively, compared with the control concrete.

The variation in test results can be attributed to the porosity of the concrete containing RCA. Firstly, the higher water absorption of the RCA led to more water being added to the concrete, which could result in a higher effective water-to-cement ratio, thus a more porous cement matrix. Secondly, for the semi-dry concrete, the pores in the old mortar attached to the RCA particles opened to allow more gas or water flow in the permeability tests. These observations lead to the conclusion that the permeability of RCA concrete increases as the replacement level increases. Direct relationships between permeability and concrete durability were generally not found in the RCA literature, but higher permeability and water absorption may leave concrete more susceptible to moisture ingress and chemical attack.

Based on the literature reviewed, it could be concluded that up to 25% replacement of coarse RCA would not have a significant impact on water absorption or sorptivity, but 50% or greater replacement may result in undesirable levels of permeability for reinforced concrete applications. Less details on fine RCA were found in the literature, although experimental results by Evangelista and de Brito (2010) showed relatively high sorptivity even at 30% replacement using fine RCA, although appropriate limits for sorptivity are unclear. For non-structural concrete, specifying additional concrete cover may be an option to account for the increased permeability of RCA concrete in more aggressive environments.

Chemical attack – carbonation

Carbonation of concrete is due to the diffusion of carbon-dioxide gas (CO_2) into concrete which reacts with cement hydration products (calcium hydroxide (CH) and C-S-H) to form calcium carbonates. The reaction reduces the pH value of concrete from 12.5–13 or higher for newly-made concrete to below 9.5 (the equilibrium pH of $CaCO_3$ solution) at which point the reinforcing steel in the concrete loses its passivation and start to corrode if moisture and oxygen are available.

The resistance of a concrete to carbonation depends on the cement content in the concrete, which determines the ability of concrete to absorb the CO_2, and the open pore porosity of the concrete, which is closely related to the w/c ratio of the concrete. This leads to the observation that carbonation resistance increases with the reduction of the w/c ratio. It has also been shown that for concrete with a lower w/c ratio (i.e. less than 0.45) and high cement contents, the presence of RCA has less of an impact on carbonation resistance, though this may not be economically viable for non-structural applications.

It has been theorised that the decrease in carbonation resistance observed for RCA concrete is linked to an increase in water absorption and permeability which is linked to the presence of old mortar in the coarse RCA (Silva et al. 2015). This is supported by the observations of an increase in carbonation depth relative to increasing coarse RCA content. There have been some limited observations of slightly decreased carbonation depth resulting from the addition of coarse RCA (Xiao et al. 2012), which was attributed to the effect of a higher mortar content in the RCA concrete compared to the control mix. Similarly, Shayan and Xu (2003) found that the 100% RCA concrete (w/c 0.35, 28-day strength 65 MPa) had the same resistance to carbonation as the control concrete, though the low w/c ratio and high cement content are likely to contribute to this outcome.

Furthermore, fine RCA has been shown to be relatively more detrimental to carbonation depth, resulting in higher increases in depth relative to coarse RCA. This is thought to be linked to the higher water absorption of fine RCA, which in turn is linked to the higher mortar content of fine RCA compared to coarse RCA. As for coarse RCA, carbonation depths for fine RCA concrete are similar to control mixes where a high cement content and low w/c ratio are constant between the mixes (Shayan & Xu 2003).

Sagoe-Crentsil et al. (2001), who examined the water absorption rate and carbonation rate, showed that there was little difference in these properties between RCA concrete and NA concrete having a cement content of 242 kg/m^3 (25 MPa target strength), although the coarse RCA concrete showed the highest carbonation depth (28 mm vs. 25 mm for the NA concrete). Silva et al. (2015) reviewed the effect of incorporating RCA on the carbonation behaviour of concrete. They found that carbonation depths increased with increasing replacement levels when recycled aggregate concrete mixes were made using a similar mix design to that of the control NA concrete.

Amorim et al. (2012), showed that the carbonation depth of concrete (tested at 5% CO_2, 60% relative humidity) increased with an increase in RCA content, but the difference was not large, e.g. for water-cured specimens, the carbonation depth was about 8.5, 9.5, 10.1 and 10.5 mm for concrete (w/c 0.43, 28-day strength 50 MPa) with RCA content 0%, 20%, 50% and 100% respectively. The difference in the carbonation depth of concrete with 50% RCA and 100% RCA was only marginal.

It may be noted that the concrete carbonation rate, or the carbonation depth after a certain exposure time tested in an accelerated condition, e.g. 4 or 5% CO_2 (note: current atmospheric CO_2 is about 400 ppm, or 0.04% (Earth Systems Research Laboratories 2020), was significantly influenced by the moisture condition of the concrete specimens during testing. The moisture content in concrete depends more on the w/c ratio and cement content than coarse aggregate type. For a concrete of grade 45 MPa and above (cement content about 400 kg/m^3 or higher, and a w/c ratio about 0.45 or lower), the carbonation of RCA concrete would not be significantly different from that of NA concrete.

It can be concluded that the inclusion of RCA in concrete is likely to result in decreased carbonation resistance. 100% coarse RCA concrete may increase carbonation depths twofold compared to NA concrete, while the inclusion of 100% fine RCA may result in up to an eight-fold increase in carbonation depth. Where corrosion of reinforcement is a concern, it is recommended the coarse RCA content is limited to 50% or lower.

Chemical attack – chloride diffusion

Chloride diffusion into concrete is a major concern regarding reinforcing steel corrosion because the physio-chemical potential of steel changes drastically in the presence of chloride which will promote steel corrosion. Chloride ions (from seawater, ground water, or airborne) penetrate concrete with a higher chloride diffusion coefficient more quickly and will progress through the cover concrete of the component to reach the steel. This property depends on the permeable pore content in concrete, which is the path for the ingress of chloride ions, and the cementitious materials (cement and supplementary cementing materials, such as fly ash, slag and silica fume), which have the capacity to absorb the ions as well as modifying concrete pore structure to slow down the process.

Shayan and Xu (2003) showed the chloride penetration depth in concrete (60 MPa grade) after immersion in a 5% NaCl for 112 days was 5.5, 7.5 and 5.2 mm in the 100% coarse RCA concrete, 100% coarse RCA with half the sand replaced by fine RCA, and the control concrete. It was concluded that fine RCA should be used with caution, whilst the effect of coarse RCA was insignificant. Little detail is provided on the reasons for the negative impact of fine RCA on chloride penetration in the paper, though it could be theorised that since the fine RCA fraction is likely to contain a higher proportion of fine crushed old mortar, it results in greater permeability and lower resistance to chloride ingress.

The more significant influence of coarse RCA was demonstrated on concrete designed for low- to moderate-strength grade. Shaikh and Nguyen (2013) tested the influence of 25% and 50% coarse RCA (RCA from a water recycling plant in Perth, WA) on the durability of a concrete (40 MPa grade). The chloride penetrability at 91 days curing time (ASTM C1202:2012) of the 50% RCA concrete was found to be considerably higher (~5800 coulombs) while the penetrability of the 25% RCA concrete was almost the same as the control concrete (~4200 coulombs). It should be noted that both the control concrete and the 50% RCA concrete fell into the 'high' chloride ion penetration classification (poorest electrical resistance) (> 4000 coulombs passed) based on the ASTM C1202:1997 limits (Shayan & Xu 2003). For comparison, the ASTM C1202:1997 results presented by Shayan and Xu (2003) ranged from 258 coulombs for the control mix to 1163 coulombs for the mix using coarse and fine RCA, though these mixes were 60 MPa grade.

In a study of the durability of concrete influenced by fine RCA (replacing sand), Evangelista and de Brito (2010) showed that the chloride penetration depth of the concrete containing 30% and 100% fine RCA as sand increased respectively by 12% and 34%.

In summary, the difference in the chloride penetrability between coarse RCA and NA concrete was not large, and the diffusion coefficient of concrete made with coarse RCA would be in the acceptable range as that of the NA concrete. Using fine RCA to replace sand appears to reduce chloride resistance more significantly, and its use should be limited unless measures are taken to mitigate the adverse effects.

Physical attack

Abrasion of concrete occurs at different rates depending on where the concrete is located and the purpose of the concrete surface. For footpaths and roadways, abrasion may occur due to foot traffic and vehicular traffic, or for applications in contact with water. Contact with hard particles and debris in the flow may also lead to abrasion over time.

The resistance of concrete to abrasion is mainly related to its strength, represented by its w/c ratio and cement content, and hardness of the fine aggregate and its bond to cement paste. The quality of coarse aggregate in terms of resistance to polishing may also be of concern for surfaces where skid resistance is a key parameter (i.e. roadways).

Sagoe-Crentsil et al. (2001) tested a 25 MPa grade concrete and concluded that using RCA (100%) as coarse aggregate led to a reduction in abrasion resistance of about 12% compared to the reference concrete made with basaltic aggregate. This was equivalent to a 212 mm^2 loss of material for the coarse RCA concrete compared to ~175 mm^2 loss for the 100% NA concrete. The test was conducted using an abrasive disk of white fused aluminium oxide, though no reference is made to whether a standard test method was used.

Limbachiya et al. (2000) studied the abrasion depth of moderate- and high-strength concretes (50–70 MPa) containing up to 100% RCA as coarse aggregate. The experimental data demonstrated that the abrasion depth of the RCA concrete increased with the RCA content, e.g. the abrasion depth of the 100% RCA was higher by 8 to 14% compared to that of the control concrete which was a difference of 0.03–0.04 mm. The authors concluded that this was not a significant difference in abrasion depth.

Based on the literature reviewed it can be concluded that the use of coarse or fine RCA is likely to have a relatively small negative impact on the abrasion resistance of concrete. However, definitive conclusions are difficult, since in most cases the literature reviewed does not appear to have used standard test methods, and the appropriate limits for the test methods used are unclear. The likely impact on abrasion resistance resulting from inclusion of RCA is unlikely to have a significant impact on the non-structural concrete applications in scope of this review.

While generally not crucial for the intended uses included in scope of this review, the lower values recorded for RCA for parameters such as Los Angeles value, or Micro-Deval value (as discussed in Section 2.3.2) may reduce the polishing resistance of coarse RCA, consequently reducing the viability of RCA for use in concrete roadway materials (subjected to regular/constant vehicular traffic).

Alkali-Silica Reaction (ASR)

Depending on the aggregate source originally used, demolished concrete may contain reactive aggregates which will facilitate the development of alkali-silica reaction (ASR) in concretes produced using the RCA if appropriate preventative measures are not taken. Commercially produced RCA is likely to include aggregates from a mix of different sources, which means that it is hard to control the nature of the source material. Considering the reactive nature of certain NAs found in Queensland, it is reasonably likely that locally sourced RCA will have some level of alkali-reactive potential.

Varying results have been obtained by authors who have investigated the alkali-reactivity of RCA obtained from demolished structures which underwent ASR deterioration during operation. A review conducted by Barreto Santos et al. (2020) indicated that one author found that the newly produced concrete experienced limited ASR expansion, while another found that new concrete containing RCA from a 30-year-old bridge underwent deleterious expansion at unacceptable levels. The differing results highlight the variability of behaviour between different RCAs and mean that it is hard to make a definitive conclusion on the reactivity of RCA if the aggregate origin of the original concrete is not known.

Shehata et al. (2010) reviewed several cases where RCA produced from the ASR-affected concrete was used and high expansion was detected in the new concrete. These authors conducted experiments to compare RCA produced from an ASR-affected concrete (using a highly-reactive siliceous limestone as aggregate after 12 years exposure) with the original ASR reactive aggregate by conducting accelerated mortar bar (AMBT) (ASTM C1260-01:2001) and accelerated prism testing (CSA A23.2-14A:1977). The concrete was produced using 100% RCA for both the coarse and fine aggregate fractions. They found that 50% by weight replacement of cement with fly ash was sufficient to protect against expansion in concrete using the reactive NA, but over 70% replacement would be needed to achieve the same results in the RCA concrete (which is not feasible for general concrete production). Further tests showed that a ternary blend consisting of 5% silica fume and 25% fly ash replacement by weight of cement was effective in keeping the 100% RCA concrete below the 1-year prism test (in accordance with CSA A23.2-14A:1977 – reportedly similar to AS 1141.60.2:2014 (Sirivivatnanon et al. 2019)) expansion limit.

It was suggested that the alkalis contained in the RCA (which may be in the attached mortar and in the ASR gel) would have contributed to the total alkali in the new concrete, evidenced by the fact that the concrete specimens made using water-washed RCA showed less expansion (washing does not appear to be standard practice) (Shehata et al. 2010). The crushing of RCA may also lead to exposure of fresh faces of the original NA, which may increase the reactive potential of the RCA if un-reacted material is exposed. Conversely, if an RCA has undergone ASR, and fresh faces are not exposed, it is likely that the reactivity of the RCA will be reduced compared to that of the original aggregate, particularly if most or all of the reactive material has been consumed (Tanner et al. 2015).

Adams and Ideker (2020) investigated the effectiveness of SCMs for mitigating ASR in RCA concrete using modified 14-day AMBTs according to ASTM C1260-14:2019 and ASTM C1567-13:2019 (aggregate washing procedure was altered due to concerns that washing RCA may result in reduced expansions due to alkali being washed away). It was found that the level of fly ash required to mitigate ASR was higher in 100% RCA concrete (40% fly ash replacement of Portland cement by weight) compared to 100% NA concrete made using similar coarse aggregates to that used to produce the recycled concrete (20% fly ash replacement of Portland cement by weight). Furthermore, it was found that a ternary blend of 10% metakaolin, 25% fly ash and 65% Portland cement was most effective for ASR mitigation, reducing expansion below the 0.1% limit at 14 days for all mixes tested. The concrete prism test (ASTM C1293:2020) is recommended by Adams and Ideker (2020) to be undertaken to correlate results with mortar bar testing methods.

It should be noted that, in the accelerated ASR tests, the aggregates were crushed and sieved to grade. This process could have removed a large part of the reacted layer (composed with ASR gels, which were soft) over the reacted aggregate. Thus, the part of the RCA tested was essentially the same aggregate as that obtained from the quarry. Shayan and Xu (2003) tested the AAR reactivity of RCA concrete by using both the accelerated mortar bar (AMBT) and concrete prisms (CPT) tests (using the as-received RCA supplied by a recycling plant in Melbourne). The accelerated mortar bar test (AMBT) was conducted based on a test procedure developed by Shayan et al. (1988), which was reportedly the basis for the RMS T363:2012 test method. No details are provided regarding the origin of the concrete prism test method.

The testing conducted by Shayan and Xu (2003) showed that, although the RCA was classified as 'reactive' by the accelerated mortar bar test, it was classified as 'non-reactive' by the concrete prism test. Shayan and Xu (2003) advises that the RCA material was of basalt/dolerite origin, which has been known to fail the AMBT, but still perform satisfactorily in concrete. This indicates that the different results obtained by the two test methods may be primarily due to an aggregate-specific issue rather than an issue associated with the general detection of ASR-potential in RCA. The concrete prism test (AS 1141.60.2:2014) has been shown to be generally more reliable than the Australian Standard AMBT (AS 1141.60.1:2014) for correctly identifying non-reactive aggregates (Sirivivatnanon, Mohammadi, & South 2016), but requires a significantly longer test period (1–2 years for CPT vs. 10–21 days for AMBT).

Stark (1996) investigated the alkali-reactivity of coarse RCA obtained from the crushing of cylinders cast using known reactive NA (rhyolitic to andesitic composition). As a part of this research, cements with various alkali contents (0.5% up to 1.0%) as well as mixes with and without fly ash replacement of cement were investigated. It was concluded that the use of 20% low-lime fly ash (Class F based on ASTM, containing less than 5.47% calcium oxide) was sufficient to mitigate ASR expansion in the concrete produced using coarse RCA at replacement levels up to 100% of coarse NA. Similarly, Li and Gress (2006) showed that 25% by weight replacement of cement using fly ash was appropriate to reduce expansion to acceptable levels based on the AMBT (as per ASTM C1260-05) and CPT (as per ASTM C1293) results for RCA concrete produced using blue rock aggregate. Stark (1996) also specifies that low alkali cement should be used to prevent the occurrence of ASR expansion (0.5–0.6% equivalent NA_2O content), which is standard TMR practice (MRTS70 (11/2018)).

Based on the reviewed literature, it can be summarised that the reactivity of RCA concretes is dependent on several factors, including:

- the reactivity of the original NA used
- the degree of crushing that took place in the preparation of the RCA (influencing whether new faces of the RCA were exposed)
- amount of reactive material consumed prior to recycling of RCA
- proportion of RCA replacement relative to known non-reactive NA
- use of SCMs to mitigate expansive potential.

The incorporation of SCMs in concrete with the aim of improving durability, controlling ASR and decreasing the environmental impacts of concrete production is part of standard TMR practice (MRTS70 (11/2018). The mix proportions currently approved by TMR are detailed in Table 2.2. These are compared to mix proportions used by various researchers which were shown to protect against ASR expansion in RCA concrete.

Table 2.2: SCM mix proportions approved by TMR vs. mix proportions in the literature

MRTS70 (11/2018)	Sheheta et al. (2010)	Adams and Ideker (2020)	Barreto Santos et al. (2020), Li and Gress (2006), Stark (1996)
65–75% GP cement, 25–35% fly ash	70% cement, 25% Intermediate calcium fly ash (CI-LA), 5% silica fume	10% metakaolin, 25% class F fly ash, 65% Portland cement	25% class F fly ash, 75% Portland cement
30–40% GP cement, 60–70% GGBFS	30% cement, 70% intermediate calcium fly ash (CI-LA)	40% Class F fly ash, 60% Portland cement	45% GP cement, 55% GGBFS

MRTS70 (11/2018)	Sheheta et al. (2010)	Adams and Ideker (2020)	Barreto Santos et al. (2020), Li and Gress (2006), Stark (1996)
50–55% GP cement, 25-30% fly ash, 20–25% GGBFS			
65–71% GP cement, 25–31% fly ash, 4–8% amorphous silica			

In practice, the variability of ASR potential exhibited by RCA (due to its heterogeneity) appears to be a good match to TMR's current policy of using SCMs in all produced concrete to mitigate ASR reactivity. It is hard to make a definitive call on whether a certain RCA will be reactive or not since the RCA may be a mix of demolished concrete which has used various NA types. The reactivity of a given batch of RCA may be investigated using either the mortar bar test (AS 1141.60.1:2014) or concrete prism test (AS 1141.60.2:2014) if there are concerns over the alkali-reactivity of the aggregate but given the variability of the material and differences between sources, the results obtained from testing of one batch may not be applicable to following batches.

The required SCM replacement level to protect against ASR development in RCA concretes will vary depending on the reactivity of the RCA material and the replacement ratio of RCA to known non-reactive NA. Research conducted to date indicates that the current TMR replacement levels recommended by MRTS70 (11/2018) are likely to be capable of mitigating ASR expansion in coarse or fine RCA concrete (RCA sourced from Australian recycled concrete) at replacement levels of up to 100%. A ternary blend including at least 25% fly ash and 5% silica fume is likely to be most effective, but binary blends including at least 25% fly ash or 55% GGBFS are also likely to produce acceptable results. Further confidence may be had in the effectiveness of the recommended SCM replacement levels if coarse or fine RCA content is limited to lower proportions of total aggregate content such as 30-50% as has been recommended based on some other performance and durability properties.

2.3.7 Improving Properties of Concrete Incorporating Recycled Aggregate

Review of the literature has shown that concrete produced using high levels of coarse or fine RCA as a NA replacement will have relatively poorer strength and durability properties when compared to an equivalent 100% NA concrete (i.e. same target strength, w/c ratio, cement contents). For use of 100% coarse RCA, this generally translates to a 10–15% reduction in the achieved compressive strength for the produced concrete. Durability transport properties including carbonation and chloride diffusion resistance are also impacted by the use of coarse and fine RCA.

Review of the literature has shown that mix modifications, including the use of lower w/c ratios and more cementitious material (binder), may be necessary at high coarse and fine RCA replacement levels (i.e. 50% and higher replacement levels) to meet target strength and durability properties. Several other approaches to improving these properties such as modifying the concrete mixing procedure, using admixtures or using SCMs in concrete production have also been investigated either individually or in varying combinations. Some of these approaches are in line with those currently adopted by TMR and concrete producers, which will be discussed in this Section.

It has been shown that acceptable concrete strength and durability properties can be achieved without mix modifications for concrete produced using 30% or lower replacement of NA using coarse or fine RCA. For non-structural applications, reduction in target compressive strength at higher RCA replacement levels may be acceptable for certain applications such as footpaths. Reduction in strength may be less acceptable for applications which might undergo concentrated loadings, such as kerb and guttering.

Supplementary Cementitious Materials (SCMs)

The use of SCMs including fly ash (FA), ground-granulated blast furnace slag (GGBFS) and silica fume (SF) is standard practice for TMR concrete production as detailed by MRTS70 (11/2018). Use of these SCMs as a partial replacement of Portland cement has benefits such as increasing strength properties, improving resistance to chloride penetration, sulphate attack, and mitigation of alkali-reactivity in aggregates. The use of SCMs to mitigate ASR in RCA concrete is discussed in Section 2.3.6, with the conclusion made that current TMR SCM replacement levels are likely to be appropriate to mitigate ASR in concrete with up to 30% (or higher) inclusion of coarse or fine RCA.

Among the SCMs, fly ash consists of mainly spherical particles, which, when incorporated into concrete, help to improve the workability of cement paste. Kurda et al. (2019) demonstrated that partial replacement of ordinary Portland cement (OPC, or GP cement) with FA in RCA concrete reduced the amount of mixing water required to achieve fresh mix workability, which in effect reduced the w/c ratio of the mix (30% or 60% replacement of cement). The incorporation of fly ash also led to reduced shrinkage for mixes containing 50% or 100% replacement of coarse or fine RCA.

Kou and Poon (2013) conducted a study on the long-term (up to 10 years) mechanical and durability properties of concretes with 0%, 50% and 100% RCA, with fly ash (FA) as part of the cementitious materials. It was found that after 10 years field monitoring, the concrete mix that used 25% fly ash and 50% coarse RCA produced the highest measured compressive strength amongst all mixes tested (Kurda, de Brito, & Silvestre 2017). This mix also showed improved resistance to chloride penetration.

Berndt (2009) investigated the combined effect of partial cement replacement with GGBFS and use of RCA to improve the sustainability of concrete. It was shown that the binder containing 50% GGBFS had good results in terms of mechanical properties and durability for both NA concrete and RCA concrete, with the target compressive strength at 28 days (40 MPa) being achieved for mixes using up to 100% coarse RCA. This effect is reportedly linked to the ability of GGBFS to strengthen the interfacial zone between aggregate and cement paste (Berndt 2009). Splitting tensile strength was also reported to be improved by slag inclusion for the RCA mixes.

It should be noted that the inclusion of SCMs in concrete can allow for more rapid carbonation of concrete through reducing the content of calcium hydroxide in the concrete pore solution. This should not be an issue if the MRTS70 (11/2018) substitution levels are adopted, but caution should be applied if carbonation is a concern for the intended concrete application.

It can be concluded that the use of SCMs in line with the MRTS70 (11/2018) guidelines (25% replacement of cement with FA, or 50–60% replacement with GGBFS) is likely to improve the workability, strength, and durability properties to acceptable levels for concrete with up to 30% coarse aggregate replacement using coarse RCA. The literature also indicates that the inclusion of FA or GGBFS at these levels will be sufficient to mitigate ASR in RCA concrete, as discussed in Section 2.3.6.

It is recognised that current TMR practice is to include SCMs in all concrete production, meaning that it is unlikely that concrete properties matching those of an equivalent TMR NA concrete could be achieved by using standard MRTS70 (11/2018) SCM inclusion levels in RCA concrete. Notwithstanding this, the inclusion of SCMs in RCA concrete does not present any significant disadvantage and is likely to improve concrete properties to a level on par with an equivalent 100% NA, 100% GP cement concrete. This should be considered when reviewing the suitability of RCA concrete for a given application (e.g. likely exposure conditions).

Changing w/c ratio and cement content

To achieve the same design strength for RCA concrete (50–70 MPa), Limbachiya et al. (2000) changed mix proportions by slightly reducing w/c, increasing the cement content, or both, compared to a control NA concrete. However, it was found that, for 50% or more inclusion of RCA in concrete, reducing w/c would make it difficult to achieve the desired fresh properties. This difficulty can be overcome by using high range water-reducing admixtures (HRWRA), potentially at higher dosage rates compared to current practice. Thomas et al. (2013) proposed that, for concrete strength levels of up to 48 MPa, a reduction in w/c by 0.10 would be necessary if the durability properties of 100% RCA concrete were to be increased to be comparable with 100% NA concrete.

Etxeberria et al. (2007) reported that to achieve the same workability and strength as conventional concrete of moderate strength (30–45 MPa), RCA concrete made with 50% or 100% coarse RCA would require 4–10% lower effective w/c and 5–10% more cement. It was highlighted that use of up to 25% coarse RCA in moderate strength concrete would be unlikely to have an appreciable impact on strength and workability properties compared to a reference mix.

While it has been shown to be a technically viable option, reduction in w/c ratio and the use of additional cement in mixes is unlikely to be a cost effective or environmentally favourable solution to meeting target strength and durability properties for RCA concrete. Increases in admixture dosage may be viable, but this alone may not be sufficient to guarantee acceptable performance in 100% RCA concrete.

Pre-saturation of aggregate and mixing adjustments

Another approach used in the published investigations to improve concrete properties was the pre-absorption of RCA. The high water-absorption of RCA made it necessary to add extra water when producing a fresh mix with a desirable workability. It was found that the pre-saturation degree of the RCA and the amount of the added water could be adjusted to improve the concrete strength.

Tam, Gao and Tam (2005) proposed a 'two-stage mixing approach' to improve the strength of RCA concrete:

- Stage 1: dry mixing aggregates for 1 minute, and adding half of the mixing water and further mixing for 1 minute
- Stage 2: placing in cement and mixing for half a minute, then adding the rest of the mixing water, and mixing for 2 minutes.

It was shown that for concrete containing up to 30% RCA the two-stage mixing led to strength improvement which increased with the increase in the replacement ratio, and that the micro-cracks surrounding the RCA particles produced using this method were better filled by new hydration products. They considered that the improvement was related to a low w/c slurry coating the RCA in Stage 1, which filled up cracks and pores. Similarly, Andreu and Miren (2014) suggested the residual water absorption of RCA in fresh mixes could reduce the (effective) w/c in the cement-aggregate interfacial transition zone and improve its strength.

Babu et al. (2015) showed that the two-stage mixing (different from the procedure used by Tam et al. (2005) in that the NA was the last to be placed in the mixer) could be applied to produce higher strength concrete with high RCA content concrete, e.g. 60% RCA for up to 75 MPa, and 100% RCA for up to 70 MPa. The studies of the effect of the two-stage mixing procedure reported by Tam et al. (2005) and Babu et al. (2015) seemed to coincide with the effect of using unsaturated RCA for mixing.

Poon, Shui and Lam (2004) showed that using water-saturated RCA led to a reduction in the strength and that more water bleeding occurred in the fresh mix. It was proposed that the degree of water saturation of the RCA prior to mixing should not be higher than 50%. Likewise, Andreu and Miren (2014) used RCA pre-wetted to about 80% saturation, which was considered to be beneficial for developing a good interfacial zone between the RCA grain and the cement paste.

Methods which attempt to achieve a specific degree of aggregate saturation are likely to have limited accuracy in field applications, and it may not be viable to rely on the outcomes of these methods to deliver improvements in RCA concrete properties. Similarly, it may not be viable or effective to apply two-stage mixing approaches as described in this section to commercial concrete production, despite the reported effectiveness of the methods in a laboratory setting.

Notwithstanding the above discussion, there is scope for concrete producers to adjust certain parameters such as the amount of added water in a mix to account for the properties of RCA. There may be potential for mix process adjustments to contribute to the increased feasibility of higher coarse or fine RCA replacement levels (50% or greater), though consultation with concrete producers to determine what is reasonably viable is recommended.

2.3.8 Field Trials

HB 155:2002 contains some details of RCA concrete field trials which have occurred around Australia. These trials have typically been limited to footpath or parking/driveway applications. Most trials detailed in the guide have used 100% coarse RCA as a replacement for natural coarse aggregate. Some of the trial footpaths were in potentially aggressive environments near the ocean, though not subject to direct water spray or immersion.

HB 155:2002 stated that there appeared to be no evidence of apparent failure that was directly attributable to material performance for the trials studied. At the time of publication, most trials had been in place for 2–3 years, apart from two trials which had been in operation for 5 and 8 years respectively.

Some results for concrete strength and durability properties were reported for certain trials. Considering fresh properties, the slump achieved for the coarse RCA mixes was either within tolerance of the target or lower as was the case for one study where a slump of 75 mm was achieved for the RCA concrete vs. 115 mm for NA concrete. This was reportedly not an issue for placing and finishing of the concrete.

For compressive strength, results obtained were similar to those in the literature for laboratory studies. Target strengths ranged from 15 MPa up to 40 MPa (for a kerb pour). In all cases, the reference NA concrete achieved a higher compressive strength at 28 days (up to 20% higher), but the RCA mix met the 28-day target characteristic strength. No significant shrinkage cracking was observed for the in-service concrete

The longer duration trials had experienced some degradation of the footpath concrete, with the occurrence of edge cracking and spalling at joints leading to a need for a limited number of sections to be replaced. The damage observed may have been contributed to by inadequate concrete strength or improper curing, but no definitive conclusion is made by the literature. Internet searching did not yield any further details on the trials presented in the guide, so the current performance of the footpaths is unknown.

Some overseas field trials of RCA concrete have also taken place. One such coarse RCA trial took place in Canada, where a sidewalk was cast using segments mixed with differing replacement levels of RCA (0%, 30%, 50%, 70%, 100%) to allow for comparison of performance (Ahmed et al. 2020). Demolished concrete was retrieved from landfill, and large sized impurities (e.g. metal, wood, organics) were manually removed. The demolished concrete was then crushed to coarse grade between 5-20 mm. Concrete mixtures were then developed based on CSA A23.1/23.2 with the required replacement level of coarse RCA substituted for the coarse aggregate. These mixtures contained partial replacement of cement by weight with fly ash, as well as water reducer and air entrainer admixtures.

Compressive strength was tested on laboratory cylinders at 7, 28, 56 and 120 days. The 0% and 100% coarse RCA mixes reached the 32 MPa target strength at 28 days, while the other mixes did not, which is inconsistent with most of the published literature which identifies higher levels of coarse RCA replacement to be detrimental to compressive strength. Rebound hammer testing was also conducted to measure the strength of the sidewalk after time in service. No serviceability issues (cracking, wear and tear, need for maintenance) had developed in five years of sidewalk operation up to the time that the research paper was written. Monitoring will continue, but based on the observations published, the trial was a success.

The results of the field trials discussed in this section indicated that coarse RCA concrete at replacement levels up to 100% may be suitable for applications such as footpaths, driveways and parking areas. The durability of the RCA concrete for a design life of up to 50 years as specified by MRTS70 (11/2018) for normal class concrete is uncertain, and for the longer-term trials which had been in place for up to 8 years, some concrete degradation was observed which could have been linked to insufficient concrete strength or improper curing. It is unclear whether the use of RCA contributed to the development of these defects, though considering the results of laboratory studies it is a distinct possibility. A present-day review of the field trials reported on in the 2002 guide would likely provide insight into the long-term durability and performance of coarse RCA concrete, though the logistics of such an exercise may be complicated. Considering the short duration of most of the trials reported on, and the fact that no more recent information on field trials in Australia could be sourced, the suitability of higher RCA replacement levels beyond the 30% of coarse RCA previously recommended by HB 155:2002 cannot be guaranteed.

2.4 Recycled Crushed Glass (RCG)

2.4.1 Overview of RCG and Current State of Practice

Recycling of glass material is primarily fed by kerbside collection of household glass waste, as well as commercial and industrial glass recycling. In some cases, recycled glass is crushed by councils that collect kerbside recycling (e.g. Lismore City Council, Cairns Regional Council). There are also companies which receive and crush recycled glass to be sold (e.g. Enviro Sand Brisbane). Current known uses for RCG in infrastructure include as up to a 100% replacement of NA for pipe bedding and as a partial replacement of NA in road base, asphalt and concrete aggregate.

Most of the recycled glass in Australia is sourced from various types of drink containers which are made from soda-lime glass. This material is primarily used for drink containers and window glass and constitutes around 90% of glass manufactured (Robertson 2005). Other more specialised types of glass include fused silica (used for scientific equipment), Pyrex (borosilicate glass) and others such as technological and windscreen glasses which make up a relatively small portion of the glass supply. These materials may be present in small concentrations in the supply of RCG as discussed in Section 2.4.3.

To be accepted for use in TMR N-Class concrete, RCG must comply with the requirements of AS 2758.1:2014 with respect to aggregate properties and MRTS70 (11/2018) for the properties of the produced concrete. Comparison of RCG aggregate to the relevant standards is discussed in Section 2.4.2.

The coarse part of crushed glass is largely composed of flat pieces which may not comply with the particle shape requirement; as a result, the quantity of this material as a percentage of the total aggregate should be limited. To enable its use as coarse aggregate, the flakiness (refers to thin particles which have a least dimension less than 0.6 of the mean of the small sieve size passing and largest sieve size retained) of the crushed glass should be determined, and the amount incorporated should be such that the combined aggregate would satisfy requirements for particle shape. MRTS70 (11/2018) allows a maximum 30% flakiness index for coarse aggregate, which is conducted on individual aggregate products. This means that coarse RCG would be unlikely to comply with the flakiness limits. There are also some concerns linked to the strength properties of recycled coarse glass which are discussed in Section 2.4.2.

Glass is formed by heating the raw materials at a high temperature until they develop a state of fluidity and then cooling the materials down and passing through a viscous stage to become an amorphous solid. Under highly alkaline conditions, glass may react with alkalis to form alkali-silicate gels in a similar way as that occurring in natural aggregates susceptible to alkali-silica-reaction (ASR). The most widely available soda-lime glass has been identified by the literature as the least reactive type of glass, but nevertheless soda-lime glass has been shown by many studies to have expansive potential, which is discussed in Section 2.4.7, together with mitigation strategies.

VicRoads currently allows for 30% replacement of fine aggregate by recycled glass sand in non-structural concrete (VicRoads 703:2018). TfNSW allows for 15% replacement in non-structural concrete (email communication with S Henwood Jan 2021). The basis upon which these decisions were made is unclear, but considering the literature reviewed in the following sections, these replacement percentages appear to be reasonably appropriate for non-structural concrete.

Actual VicRoads practice to date appears to have been more conservative, with a June 2020 field trial conducted by Hanson in conjunction with VicRoads using 10% recycled glass fines in pavement concrete (Green Roads 2020). Mixes using both washed and unwashed glass aggregate were trialled, and it was noted that a 'little more' water was required at the time of pouring, though no quantitative data is provided. It was noted that the performance of the concrete would be reviewed in six months' time from June 2020, so further reporting of project outcomes may be available soon, though it is unclear whether the results of this review will be published.

A trial of 15% fine RCG aggregate replacement in pavement concrete on the Woolgoolga-Ballina section of the Pacific Highway upgrade took place in 2019. RCG was sourced from Lismore City Council and washed prior to blending with natural sand. The project was identified as a success with regard to the final concrete product, though limited details are available regarding target strength or durability factors.

Recycled crushed glass is currently established as a partial replacement for natural road base material in Australia, and it has been noted that much of the crushed recycled glass available in Queensland is currently used in this manner (Latter 2021a). Furthermore, a primary supplier in Brisbane has previously indicated that most of their crushed glass is supplied to a council, though it is unclear how this reflects upon other suppliers. The feasibility of RCG inclusion in concrete will be impacted upon by the proximity of RCG suppliers to the site where concrete is to be mixed and poured.

2.4.2 Mechanical Properties of RCG Aggregate

Crushed glass has uniform physical properties across most sources in terms of density (around 2500 kg/m^3) and water absorption. While the water absorption of glass is negligible, the water absorption of recycled glass can be higher due to the presence of fissures in glass particles which may form during crushing. Glass is a hard material, having a modulus of elasticity comparable, or higher than, normal natural aggregate. The presence of contaminants in RCG such as plastics, paper, metals etc. may impact upon the physical properties of the combined product, and the presence of these materials should be limited to maintain acceptable RCG material properties as discussed in Section 2.4.3.

A full comparison of typical RCG properties reported by the literature to the MRTS70 (11/2018) and AS 2758.1:2014 aggregate properties is shown in Table 2.3. For several of the aggregate properties such as deleterious fines index and wet strength, no data was found in the literature for RCG. Further research may be required to verify these properties. Chloride content and sulphate content are expected to be negligible for soda-lime RCG based on the standard chemical makeup of the glass. It must be recognised that most limits recommended by MRTS70 (11/2018) do not apply to N-Class concrete and are therefore not directly relevant to the applications included in the scope of this study. In these cases, the limits are provided for comparison purposes only.

It is known that flaky and elongated particles in concrete aggregate are undesirable due to several factors including weakness when load is applied to the flat side of the particle, lower workability and a poorer bond between the aggregate and cement paste, which subsequently reduces compressive strength, flexural strength and durability (Raju 2014). Coarse particles of recycled glass (> 5 mm) would therefore be assessed as unsuitable for use in TMR concrete if considering the particle shape requirements of the flakiness index. Fine recycled glass (size < 5 mm) is generally crushed to size, which reduces the prevalence of 'flaky' or elongated particles. Flakiness is not directly measurable for size fractions which pass the 4.75 mm sieve, but poorly shaped and flaky fine particles may increase water demand and decrease workability, and such materials should be treated with caution.

Research on using recycled glass as coarse aggregate found that concrete strength typically decreases with increasing particle size of the glass, which was explained in terms of the cement paste binding poorly to the smooth surface of the glass particles, and the tendency of the coarse recycled glass to crumble under applied force (high friability) (Berry, Stephens & Cross 2011; Polley, Cramer & de la Cruz 1998). The friability of large RCG particles was explained as being caused by the crushing impact in the recycling process that leaves internal cracks in the glass (Cassar & Camilleri 2012).

The grading of RCG sourced from Queensland suppliers is shown in Figure 2.3. Eight of the nine sources tested fell primarily within the limits specified by AS 2758.1 for manufactured fine aggregates, except for one coarse graded RCG aggregate. Grading slightly coarser than the AS2758.1 recommendation can also be observed for two sources (Source 3, Type 2; Source 2, Type 1). Grading of aggregate may be improved by blending with appropriate natural sand.

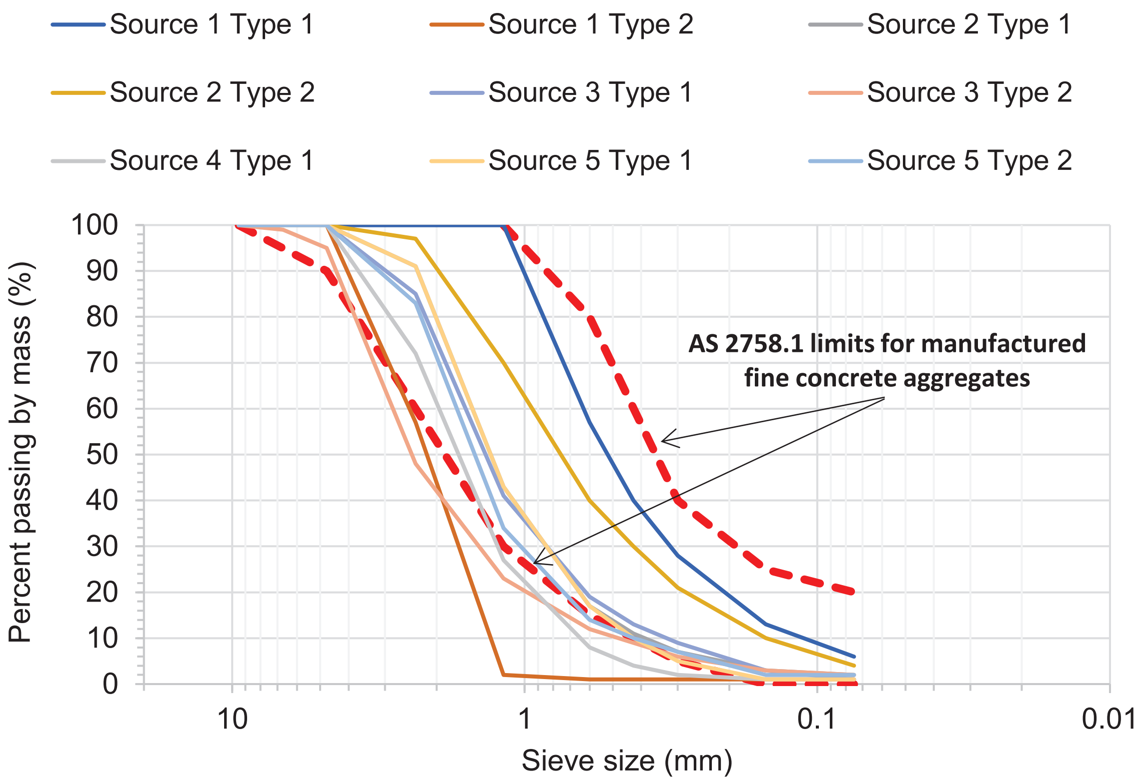
Table 2.3: Reported properties of RCG aggregate vs. standard values

Property	AS 2758.1:2014	MRTS70 (Nov 2018)	HB 155 – 2002	Recycled glass (coarse)	Recycled glass (fine)
Aggregate grading			As per AS 2758.1	See Figure 2.3	
Density	≥ 2.1, < 3.2 t/m^3 (normal weight aggregate)	≥ 2.1, < 3.2 t/m^3	> 1.8 t/m^3	2.15–2.65 t/m^{3-} (soda-lime glass)[1]	
Water absorption	No hard limit. Advises that the water absorption of lightweight, vesicular or recycled aggregates can exceed 2% considerably without affecting many of the properties of concrete made using such aggregates. Recommended that such aggregates are pre-wetted prior to mixing.	≤ 2.5%		0–0.48% (1)	
Degradation factor (fine aggregate)	≥ 60	≥ 50		N/A	No data
Flakiness index	≤ 35%	≤ 30%		84.4–94.7% (2)	N/A
Deleterious fines index	≤ 150	≤ 150		No data	No data
Material finer than 75 µm	• Coarse: ≤ 2% • Fine: ≤ 20% (manufactured fines)	• Coarse: ≤ 2% • Fine: ≤ 20% (manufactured fines)		No data	< 10% (3)
Particle shape	Particle shape in coarse aggregate: the proportion of misshapen particles retained on the 9.50 mm test sieve shall not exceed 10% when determined according to AS 1141.14:2007 using a 3:1 ratio, or the flakiness index shall not exceed 35% when determined according to AS 1141.15:1999. The tests can be performed using a 2:1 ratio when the available aggregate does not comply with the requirements.	As per supply agreement otherwise Table B1 and B2 of AS 2758.1:2014			
Los Angeles value	Maximum percent loss is 35% and 30% for concrete exposure Class A and B, and C, respectively.	Not referenced.		24–52% (1) Reference states coarse glass unsuitable for concrete.	N/A
Wet strength	• ≥ 50 kN for exposure class A1, A2 • ≥ 80 kN for exposure class B1, B2	≥ 110 kN		No data	N/A

Property	AS 2758.1:2014	MRTS70 (Nov 2018)	HB 155 – 2002	Recycled glass (coarse)	Recycled glass (fine)
	• ≥ 100 kN for exposure class C				
Wet/dry strength variation	• ≤ 45% for exposure class A1, A2 • ≤ 35% for exposure class B1, B2 • ≤ 25% for exposure class C	≤ 35%		No data	N/A
Impurities (weak particles) % in dry mass	≤ 0.5% (coarse aggregate) Other limits of weak particles may be adopted based on demonstration of successful performance.	≤ 0.5%		Coarse glass likely to be friable (1). Likely to exceed weak particle limit.	N/A
Organic content	Colour shall not be darker than standard colour of reference solution. Performance of suspect fine aggregate may be verified by comparing performance to satisfactory fine aggregate.	Negative		Varies depending on source. See Section 2.4.3.	
Chloride content	Combined chloride salt content < 0.04% in reinforced concrete Total water-soluble chloride salt content < 0.03%	Report content as per AS 2758.1		No data	
Sulphate content	Sulphate content of concrete mix ≤ 5% by mass of Portland cement	Report content as per AS 2758.1		No data	
Sugar content	Test negative to presence of sugar based on AS 1141.35	As per AS 2758.1		Varies depending on source. See Section 2.4.3.	
ASR reactivity		Where aggregate is identified as having potential for AAR in accordance with AS 2758.1, treatment shall be in accordance with SA HB79.		Generally regarded as alkali-reactive. See Section 2.4.7.	

1. Obe et al. (2018).
2. Mohajerani et al. (2017).
3. Latter (2021a).

Figure 2.3: Particle size distributions of crushed recycled glass materials from multiple sources



Source: Latter (2021a).

2.4.3 Contamination of Aggregate

Contamination of RCG may occur due to the inclusion of undesirable materials (e.g. metals, plastics, ceramics, glass other than soda-lime glass) in crushed glass, sugar residues from food products, or heavy metals and other chemicals from specialised glasses which may be harmful to the environment if leaching occurs.

Many Queensland councils specify that only container glass is to be recycled through kerbside collection (e.g. Townsville City Council n.d.). This does not guarantee that recycled glass will not contain specialised glasses or other materials however, and the contamination of recycled glass with ceramics, stoneware and Pyrex has been noted as an issue in Victoria (Victorian Parliamentary Budget Office 2019).

Chemical testing conducted as part of NACoE P76 to determine the composition of glass aggregate streams found that eight of the nine aggregates studied had individual compositions of < 0.1% for materials including metal, ceramics and paper (one outlier had 0.26% paper and 0.16% vegetable matter). It must be noted however that the chemical testing uses a very small sample to determine the composition of the material, which may not be entirely representative of the bulk material available (personal communication with L Latter, February 2021).

The recycled glass aggregates also underwent chemical analysis, which included detection of heavy metals, leachates, and foreign materials. Two aggregates had lead content above the NSW EPA specified maximum (100 mg/kg), with lead contents of 120 mg/kg and 2000 mg/kg respectively. This may indicate that the glass includes comparatively higher proportions of glass other than container glass (e.g. lightbulbs, electronic glass). This indicates that contamination of glass is to be expected if there is not a rigorous framework to regulate the supply and ensure that RCG is fit for use in concrete. All foreign materials, including metal, rubber, plastic, ceramics etc. were found to be below 0.3% by mass of the aggregates tested. This is below the limit of 2% foreign materials specified by VicRoads for recycled glass aggregate. The basis of the VicRoads guideline is unclear, but it appears to be low enough to have a negligible impact on concrete properties. The TMR standard for recycled glass use in pavement applications (MRTS36 11/20) specifies a set of limits on chemical and heavy metal contamination, and it is expected that these are appropriate to manage risks from a health and safety and environmental perspective.

Sugar may be present in RCG due to food residues or other sources. Since sugar is a natural set retarder, this may impact on the set time of concrete produced using RCG. While this phenomenon has been identified by several authors as a potential issue (Obe et al. 2018; Tamanna, Tuladhar, & Sivakugan 2020), there appears to have been little qualitative research conducted which considers the impacts of sugar content as a measurable percentage of RCG aggregate. For studies where sugar content of unwashed recycled glass has been tested, the sugar content has been below the detectable limits for the test conducted (AS 1141.35) (Copland, Robertson, & Slaughter 2009, Tamanna, Tuladhar, & Sivakugan 2020). Sugar testing to AS 1141.35 was undertaken on Queensland recycled glass aggregate as part of the NACoE P76 project, finding that sugar was detected in a measurable quantity for three of the nine aggregates tested (Latter 2021a). No explanation was given for the differences in test results, and no concentrations of glass contamination were provided. A detectable amount of sugar is 1 part per 1000 parts of aggregate (AS 1141.35-2019).

One explanation offered for the absence of sugar in tested RCG aggregate is fortuitous washing of the outdoor aggregate stockpile by rain suggested by Tamanna, Tuladhar, & Sivakugan (2020). There may also be differences in how recycled glass streams are processed between different suppliers which may impact on sugar contamination. Further investigation would be required to identify whether these processes do have a significant impact on the sugar content of recycled glass. Washing has been identified as a recommended practice for RCG aggregate by some authors (Du & Tan 2014), but advice from a Brisbane-based RCG supplier (personal communication with L Latter February 2021) indicates that this is may not be a cost-effective practice.

Washed glass has been used in Australian trials to date, including the NSW Pacific Highway Upgrade trial where washed RCG was used, and the trial by Hanson in Victoria, where both washed and unwashed RCG was used in separate mixes for comparison purposes. No details have been found in the literature regarding the costs associated with washing RCG aggregate, however. Further industry consultation is recommended to determine whether washing of RCG prior to use in concrete is economically viable.

2.4.4 Cost

There is limited real-world data available in the literature regarding the cost of producing, distributing, and producing concrete from RCG aggregate. Some figures were presented by Flanders (2019) for the cost of crushing fine RCG aggregate, but these were based only on the cost of electricity for the crushing machines (50 cents per tonne crushed). Other costs such as collection, transport and further processing of RCG (e.g. washing, blending) were not considered. Production costs of RCG and non-RCG mixes were reported to be within \$5/m^3 of each other for the Cairns field trial reported by Flanders (2019), though no absolute values are provided.

A review produced by Southern Sydney Regional Organisation of Councils (2020) identified that the current cost of processing and producing RCG (\$60–100/tonne) is higher than the cost of natural sand production (\$30–40/tonne). It is stated that the cost of producing RCG needs to be considered against the cost of sending glass to landfill (\$250/tonne), but without a direct incentive it is likely to be unrealistic for concrete producers to adopt the higher-priced RCG. It is more attractive however for local government to make use of RCG in-house, due to the introduction of measures such as the waste levee on material sent to landfill in Queensland (\$75/tonne) (Flanders 2019).

It is stated in the final report for the Pacific Highway Upgrade trial (Lendlease 2020) that the use of RCG in concrete is generally not cost competitive compared to manufactured or natural sand due to the costs associated with sourcing, transporting, washing, processing and blending the product prior to batching. It is stated that demand and efficiency of scale can reduce the price of RCG aggregate, but no quantitative or qualitative data is provided meaning that it is difficult to assess the validity of these statements.

A review of the economics of including RCG in road materials (excluding concrete) conducted for NACoE P76 (Latter 2021a) identified that the cost of including RCG in asphalt mixtures overseas can either reduce life cycle costs or maintain cost neutrality with natural materials. Latter (2021a) established that RCG for road pavement applications is cost competitive if there are large haulage distances associated with NA compared to the RCG material. RCG prices are likely to be uncompetitive at distances 15 km or less from NA quarries, and more competitive where distances to the NA quarry are more than 65 km. This does not take into account costs specific to concrete production such as blending or washing of aggregate.

Based on the limited information that is currently available in the literature, it can be concluded that RCG aggregate is likely to be either of similar cost or more expensive than NA for use in concrete depending on factors including haulage distances for NA, and the degree of processing which is required to prepare the RCG for concrete production.

2.4.5 Availability

Most literature indicates that there is a large supply of waste glass available in Australia and Queensland, though it is unclear how much of this supply is either suitable or available for concrete production. One study found that approximately 850,000 tonnes of glass are consumed in Australia each year, with 350,000 tonnes recovered for recycling (Austroads 2009 via Latter 2021a). Publicly available data from the Alex Fraser Group indicates that 150,000 tonnes of fine RCG (unsuitable for recycling into containers) is being produced in Melbourne each year. In SEQ, advice from Enviro Sand indicates that 50–60,000 tonnes per year of RCG are available for infrastructure applications (personal communication).

The grading data obtained from NACoE P76 for Queensland RCG suppliers shows that most available product falls under the classification of fine aggregate, with one exception that was crushed as coarse aggregate. This may indicate that fine RCG is most readily available in Queensland, although no production volumes or other quantitative measures of availability are provided in the literature.

The use of RCG for relatively low risk applications such as road base or pipe bedding appears to be the primary application at the present time. There is also a push to use higher volumes of recycled glass for higher value applications including the production of new glass containers. Further consultation with Queensland RCG suppliers is recommended to determine what volume of material is available for concrete production.

2.4.6 Performance of RCG Concrete

Fresh properties

There is some contention in the literature as to whether glass increases or decreases the workability of concrete. Some authors have reported increases, suggesting that the smooth nature of glass results in weaker cohesion with the cement paste (Ali and Al-Tersawy 2012), while others have reported decreases which are postulated to be due to the angular nature of the glass material (Adaway and Wang 2015; Tamanna, Tuladhar & Sivakugan 2020). It appears likely that both effects exist to varying degrees and will act in opposition to each other (Du & Tan 2014). The literature generally shows that workability is not greatly impacted for mixes containing up to 20% coarse or fine RCG aggregate and may be acceptable for mixes containing up to 40% fine RCG aggregate (Obe et al. 2018).

Serpa, de Brito and Pontes (2015) studied the mechanical properties of moderate-strength concrete (44 MPa) incorporating glass as partial replacement of coarse aggregate only, fine aggregate only, and both coarse and fine aggregate. Using up to 20% replacement of natural sand with fine RCG resulted in slightly decreased workability but using coarse RCG as a 20% replacement coarse aggregate led to slightly increased workability compared to a 100% NA reference mix.

Tamanna, Tuladhar and Sivakugan (2020) tested the properties of concrete (design grade 32 MPa, 0.53 w/c ratio) containing 20%, 40% and 60% of fine RCG to replace sand for Cairns Regional Council. The slump of the fresh mix decreased as the replacement level increased, and it was reported that the 60% RCG mix was difficult to spread and screed when used as part of a trial footpath pour (Flanders 2019). In this field trial, the 40% RCG mix also showed decreased workability compared to the reference mix but was within ±15 mm of the target slump.

Relatively similar results were obtained by Adaway and Wang (2015), who produced test mixes using RCG sourced from a Sydney supplier. Using fine RCG (100% passing the 1.18 mm sieve) to replace sand led to a decreased workability. This was explained by the fact that the angular nature of the glass particles meant that more cement paste was needed to coat the surface; however, this trend was not established at replacement levels up to 25%.

Strength and deformation properties

There is not a clear consensus in the literature as to whether the replacement of fine aggregate with RCG sand at levels up to 100% will improve or reduce concrete strength properties, though most studies indicate that some reduction in final compressive, tensile and flexural strength occurs relative to control mixes for replacement levels above 20–30%. The research available for replacement of coarse glass is more limited, but it is generally shown that up to 20% replacement can be achieved without significant reduction in strength properties. Considering elastic modulus, it has been shown that an increase or parity with the reference mix can be expected with the inclusion of fine RCG if a constant w/c ratio is maintained (Obe et al. 2018).

Serpa, de Brito and Pontes (2015) found that the compressive strength of concrete containing coarse or fine glass was somewhat lower than that of the control concrete, and it was more affected by the fine glass than by the coarse glass, but there was no noticeable influence on the modulus of elasticity. They concluded that it is feasible to use recycled glass aggregate (both coarse and fine) up to a 20% replacement ratio of either coarse or fine aggregate in concrete.

Dumitru et al. (2013) undertook a trial of recycled glass in a footpath at the Boral quarry south of Sydney. A mix was developed using fine RCG as a 45% replacement of natural sand with a target compressive strength of 35 MPa. The target compressive strength was met and exceeded by the RCG concrete and the control concrete at 28 days. The flexural strength of the RCG mixes did not reach the 4.5 MPa target at 28 days but reached up to 6 MPa at 91 days. Shrinkage was below 600 microstrain at 56 days, which was lower than that measured for the control concrete.

Tamanna, Tuladhar and Sivakugan (2020) found that compressive strength of concrete slightly increased at a replacement level of 20%. At 40% and 60% replacement levels, the 28 days compressive strength slightly decreased compared with that of the control. The lower strength for the 60% glass sand concrete was considered to be due to weaker interfacial transition zone (ITZ) development in the RGS concrete. Negligible differences in concrete density were measured for the control and recycled glass mixes, with hardened densities ranging from 2396 kg/m^3 for the control down to 2361 kg/m^3 for the 60% RGS concrete. Relatively similar results were obtained by Adaway and Wang (2015) who found that the compressive strength of test mixes (target 40 MPa) slightly increased at a replacement level up to 30% but significantly decreased at 40% (16% loss of compressive strength relative to control). They suggested that the optimum replacement level was 30%.

Du and Tan (2014) investigated the effect of recycled fine glass to replace sand at levels of 0%, 25%, 50%, 75% and 100% on three grades of concrete (30, 45 and 60 MPa). There were no significant observed influences on the fresh or mechanical properties, including strength and modulus of elasticity. The drying shrinkage was lower for the high-volume glass concrete. The mixes produced by Du and Tan (2014) included a mix containing 30% fly ash, and a mix containing 60% GGBFS replacement of cement respectively. The waste glass bottles were soaked in water for one day prior to crushing and mixing with the aim being to remove impurities. Du and Tan (2014) noted that compressive strength for all recycled glass mixes was comparable to the reference mix at 7 and 28 days, with increased compressive strength compared to the reference mix observed at 90 days for all glass mixes.

Ali and Al-Tersawy (2012) studied self-compacting concretes made with different cement contents (350, 400 and 450 kg/m^3) and incorporated with up to 50% sand replaced by fine glass. They showed that, as the glass content increased, the compressive, splitting tensile and flexural strengths and the modulus of elasticity decreased. The decrease in mechanical properties was explained in terms of the high smoothness of the glass particles which reduced the adhesion between the cement paste and glass.

Mardani-Aghabaglou, Tuyan and Ramyar (2015) reported a slight reduction in the strength of concrete (50 MPa) containing up to 60% of fine glass as sand and that the compressive and splitting tensile strength reduced by about 8% compared to the control mix.

2.4.7 Durability of RCG Concrete

Permeation

Glass is hard and non-porous, which means that RCG aggregate does not increase the porosity of concrete, and aspects of durability which closely relate to the porosity, including the resistance to chloride penetration, carbonation and permeability are not significantly impacted (Obe et al. 2018).

Mardani-Aghabaglou et al. (2015) demonstrated that concretes of three grades incorporating up to 60% of fine glass as sand had improved durability when tested for water absorption, depth of water penetration under pressure, sorptivity and chloride penetrability, despite a slightly decreased strength with an increase in glass content. Since recycled glass sand has near-zero water absorption, the water absorption of concrete generally decreases as the proportion of glass sand as fine aggregate increases (Obe et al. 2018).

Chemical attack

Several researchers have demonstrated that the chloride resistance and sulphate resistance of fine RCG concrete (up to 100% replacement of NA) is either increased or remains constant relative to 100% NA reference mixes (Obe et al. 2018). The tests conducted by Du and Tan (2014) found that concrete with 0% and 25% fine RCG was of moderate chloride permeability based on ASTM C1202:2012 (> 2000 coulombs passed) while concrete with 50% and 75% fine RCG showed low chloride permeability (< 2000 coulombs passed).

Tamanna et al. (2020), who tested properties of concrete containing 20%, 40% and 60% finely crushed glass (mixed coloured soda-lime glass supplied by Cairns Regional Council and crushed in the laboratory) to replace sand, showed that there was some improvement in concrete resistance to chloride ion penetration (determined at 56 days of concrete age as per ASTM C1202:2019) as the proportion of glass as sand in concrete increased, with a 32% increase in resistance over the control concrete (~4200 coulombs charge passed, high) at 56 days for 40% recycled glass sand concrete (~3000 coulombs passed).

Chloride permeability results for concrete produced using GP cement generally range between 1000 coulombs passed (w/c ratio below 0.4), and 4000 coulombs passed (w/c ratio above 0.6). Acceptable limits for specific applications are not specified by the test method, but certain sources state that low (1000–1500 coulombs passed) permeability is preferred for structural applications such as bridge decks (which is achieved using SCMs) (e.g. Detwiler (2019)). The results returned for the control and RCG concretes tested by Tamanna et al. (2020) are in the medium-high range and would be typical of concrete intended for shorter design lives and/or more benign exposure conditions.

Several researchers have found that the inclusion of glass as fine aggregate in concrete increases the carbonation depth relative to concrete cast using control mixes (0% glass sand) (Obe et al. 2018). A 7% increase was observed at 20% replacement of fine aggregate with RGS by De Castro and De Brito (2013) (0.58 w/c ratio), and a 33% increase was observed for 25% replacement with RGS by Ling and Poon (2012) (0.48 w/c ratio). Furthermore, a 100% increase in depth was observed by Dhir et al. (2005). It is speculated that some portion of the calcium hydroxide in concrete is consumed by the fine RGS particles leading to an increase in carbonation depth, but Obe et al. (2018) state that further research is required to verify this hypothesis. Where carbonation depth is a concern, mitigative measures may be required such as lower w/c ratios.

Based on the research conducted to date, it can be concluded that the inclusion of fine RCG in concrete at replacement levels up to 100% will not have a harmful impact on the chloride or sulphate resistance of produced concrete. Conversely, there are some concerns related to the carbonation resistance of RCG concrete, but there have been a limited number of studies which have tested this parameter. Based on currently available literature, the detrimental impact of fine RCG is likely to be limited up to 20% replacement. It is unclear however whether an unacceptable reduction in carbonation resistance could result from combination of RCG with SCMs such as fly ash which are also known to reduce carbonation resistance.

Physical attack

It has been reported that the abrasion resistance of concrete produced using fine RCG aggregate at replacement ratios up to 100% decreases compared to reference mixes which use 100% fine NA if a constant w/c ratio is maintained (Obe et al. 2018). This is thought to be due to inferior bond development between the glass and cement paste matrix (Rajabipour, Maraghechi & Fischer 2010), which is in line with the conclusions of several other studies regarding inferior strength development in RCG concrete. Serpa, de Brito and Pontes (2015) found that the abrasion resistance of concrete containing up to 20% fine RCG was virtually unchanged (testing to DIN 52108 – Deutches Institut fur Normung 2010) compared to a 100% NA reference mix, indicating that abrasion may be less of an issue at lower replacement levels.

The field trial undertaken by Dumitru et al. (2013) using 45% fine RCG as a natural sand replacement found lower abrasion resistance for the RCG mix compared to the control concrete. It was observed that wheels and windborne grit from the Boral quarry were accelerating abrasion. Coarse aggregate had been exposed after 1 year of the trial, after which abrasion slowed.

Alkali-Silica Reaction (ASR)

It was identified earlier in the review that the majority of recycled glass available in Australia is sourced from the recycling of soda-lime container glass through kerbside recycling and bulk container recovery. Therefore, the reactive potential of soda-lime glass is of greatest concern to the production of concrete using recycled glass.

The silica in soda-lime glass is amorphous which will chemically react with alkali, meaning that ASR is a concern for the use of RCG aggregate. Alkali contained in glass is also of a concern in terms of ASR in concrete, because a concrete pore solution is highly alkaline and will dissolve glass to some extent, and also release alkali contained in the glass into the concrete over time, which in turn increases glass dissolution (Dhir, Dyer & Tang 2009; Shi 2009). Shi (2009) indicates that a pH value below 12 in the concrete pore solution is required to prevent dissolution of glass, and states that the addition of silica fume or metakaolin is the most effective for achieving this, but do not recommend any particular mix design.

Jin, Meyer and Baxter (2000) studied the expansion of glass subject to ASR reactivity testing (14-day AMBT as per ASTM C1260). They summarised the effect of glass type and colour on the potential for ASR. Fused silica (amorphous silica oxide) was the most reactive glass followed by Pyrex (borosilicate) and soda-lime glass; clear glass caused the most expansion, amber glass (purportedly due to the presence of Fe_2O_3) was considerably less reactive, and green glass (purportedly due to the presence of Cr_2O_3) appeared to be not reactive. It was considered that the chemicals used to colour glass have potential to counteract the ASR reaction, but this is debatable, given that other research has found differing trends regarding the reactivity of the various colours of glass (Dhir, Dyer & Tang 2009).

Given the potentially undesirable reactive potential of fused silica, borosilicate and other glass types, the percentage composition of these glasses within RCG aggregate should be limited. 10% replacement of natural sand using fine crushed Pyrex and fused silica produced expansions multiple times those produced for 10% replacement using fine crushed soda-lime glass in the experiments conducted by Jin, Meyer and Baxter (2000). The critical replacement level of these glasses for deleterious expansion in concrete is not clear, but it could be reasonably hypothesised that at low proportions of an RCG stream (e.g. < 10%), any increased expansive potential could be offset by the addition of appropriate SCMs as per Table 2.4.

The ASR expansion caused by glass is contributed to by the size of the glass and the amount of glass used as aggregate. Research (Shayan & Xu 2004) has shown that glass particles of about 1.5 mm in size at 100% replacement of fine aggregate caused excessive (greater than 0.1% at 21 days) ASR expansion in laboratory tests (0.8% at 21 days as per the ASTM C1260 mortar bar test), whereas particles smaller than 0.25 mm did not cause the expansion. This study used Geelong GP cement (0.62% Na_2O content) and no additional SCMs. The research summarised by Obe et al. (2018) is generally in agreement with these observations, showing that the highest level of expansion has been observed for particles of between 1 and 3 mm diameter. It has been speculated that particles in this size range are expansive due to the presence of microcracking on the glass surface, which provides locations for expansive material to occur (Dhir et al. 2009). As shown in Section 2.4.2, particle sizing for Queensland RCG generally has between 10% and 80% of particles that are 1 mm or greater in size, meaning that reactive potential exists.

Most research has shown that fine glass in concrete contributed to deleterious expansion (Dhir, Dyer & Tang 2009, Obe et al. 2018), but others demonstrated that glass is innocuous. Du and Tan (2014) tested brown-coloured fine RCG as a partial or total replacement of sand (0%, 25%, 50%, 75%, 100% replacement with 0.6% alkali content OPC, no other SCMs) and found that the test mixes showed innocuous ASR expansion (according to the 14–28 days mortar bar test as per ASTM C1567). The test was extended out to 49 days, where the largest expansion was observed for the control mix (0% replacement), and the lowest within the 100% glass replacement mix, which indicates that the expansion observed is likely not due to the glass inclusion.

Du and Tan (2014) hypothesised that the low expansion observed may be due to a lack of microcracks in the glass particles. This is in line with the findings of other works which have observed that ASR reactions occur within glass microcracks rather than at the interface with the concrete solution (Dhir, Dyer & Tang 2009, Maraghechi 2014; Shi 2009). The conditions for the formation and presence of microcracks in glass aggregate particles are unclear, however, with some hypotheses including that the cracks are formed due to certain manufacturing processes, or that the cracks can be formed during the processing of waste glass into aggregate (Maraghechi 2014). Without understanding the specific processes behind the formation of these cracks, it is reasonable to automatically assume that RCG is reactive unless it can be proven otherwise.

The majority of studies completed on the expansion of recycled glass concrete have used AMBT methods (ASTM C1260, ASTM C1293, RILEM AAR-2). One notable exception to this was Dhir, Dyer and Tang (2009) which used the concrete prism test BS 812-123:1990. This study found that crushed soda-lime glass used as a 100% fine aggregate replacement was classified as expansive, with expansion exceeding 0.2% over a 1-year period for amber and green glass. Clear (flint) glass did not exceed the 0.2% expansion limit, which is in opposition to most other studies where clear glass has been shown to be most expansive (Obe et al. 2018). An identical test was also conducted for crushed soda-lime glass as a 50% replacement of fine aggregate, which was classified as non-expansive for all glass colours. Similar expansion results for the different colours of glass were obtained by Dhir, Dyer and Tang (2009) using the ASTM mortar bar method. This led to the conclusion that the expansive potential was not solely explained by the glass colouring, or the chemicals used in the colouring process.

Dhir, Dyer and Tang (2009) presented a theoretical mechanism for the development of ASR in concrete containing fine glass aggregate. In a high pH environment, glass undergoes a leaching process whereby glass is converted into a gel-like network which allows for the ingress of water and subsequent swelling. For glass, a gradual dissolution of silica occurs which leads to a loss of gel and subsequent formation of calcium silica hydrate gel (CSH), which is a pozzolanic reaction. These reactions are likely to occur at different rates, and smaller glass particles will cease leaching sooner than larger particles, leading to a faster dissolution of available silica for the smaller particles. Expansion of the recycled glass concrete is thought to cease once alkalis have been absorbed into hydration products. The formation of CSH gel at the interface between the glass and concrete pore solution was also observed by Maraghechi (2014) through chemical analysis.

Dhir, Dyer and Tang (2009) found that the inclusion of 50% ground granulated blast furnace slag (GGBFS) as a cement replacement reduced the expansion of 100% recycled glass mixes below the 0.2% expansion limit at 1 year. 10% inclusion of metakaolin as a cement replacement was similarly effective in reducing the expansion. The cement used in all mixes was Portland cement with 0.70% Na_2O equivalent. These results are encouraging, given that 60–70% GGBFS is currently specified as an acceptable cement replacement by MRTS70 (11/2018). Dhir, Dyer and Tang (2009) consider that the function of GGBFS and metakaolin as an alkali sink prevents the expansion of the recycled glass concrete, and that fly ash and silica fume would have a similar effect (though no replacement levels were recommended).

Comparison of the Australian Standard AMBT (AS 1141.60.1:2014) and CPT (AS 1141.60.2:2014) has found that the AMBT is generally reliable for the detection of alkali-reactive aggregates, though it may overestimate the reactivity of certain aggregates when compared to observed field performance, with the CPT found to be more accurate in this regard (Rocker et al. 2015; Sirivivatnanon et al. 2019). The AMBT also provides a more conservative estimate of the amount of SCM required to mitigate ASR (Sirivivatnanon et al. 2019). It can be concluded that the shorter duration AMBT could be used to assess the reactivity of concrete mixes containing RCG and SCM(s), but if the mix was to fail the AMBT by a small margin, a less conservative result could be obtained through use of the CPT, which is likely to be more accurate.

Obe et al. (2018) conducted a review of work to date which investigated the ASR potential of recycled glass aggregate as part of a larger review on the feasibility of recycled glass as a concrete aggregate. Obe et al. (2018) concluded that the replacement levels for pozzolanic SCMs detailed in Table 2.4 are appropriate to maintain ASR expansion below the limits specified by the mortar bar test ASTM C1260 for fine aggregate replacement using crushed glass up to 100%. As previously discussed, the mortar bar test may not be entirely reliable for the detection of ASR potential, but the levels indicated below are in agreement with the recommendations of Dhir, Dyer & Tang (2009) which were based on the concrete prism test. The levels indicated by Obe et al. (2018) are compared to the TMR MRTS70 (11/2018) guidelines, showing good agreement for fly ash and GGBFS.

Table 2.4: Recommended SCM replacement levels to counteract ASR in concrete using fine glass as aggregate

	Cement replacement level	
SCM	Obe et al. (2018)	TMR MRTS70 (11/2018)
Fly ash	≥ 25%	25–35%
Metakaolin	≥ 10%	N/A
Ground granulated blast furnace slag	≥ 60%	60–70%
Silica fume	≥ 10%	4–8% in combination with 25–31% fly ash

2.4.8 Field Trials

Australian field trials to date have primarily used fine RCG as a partial sand replacement (up to 60%) in concrete used for footpaths. One of the trials reviewed used coarse RCG as a partial coarse aggregate replacement.

A trial of RCG in a footpath was conducted in Cairns, using 40% and 60% replacement of natural fine aggregate using fine RCG in an N32 grade concrete mix (Flanders 2019). Both 'standard' N32 (as per FNQROC S7 Concrete Works, tested in accordance with AS 1012.2:2014) and TMR (as per MRTS70 (11/2018)) mix designs were adopted, with the TMR mixes including Sika Retarder N at dosage of 50–100 ml/100 kg (which appeared to be the only difference between the two mixes (Tamanna, Tuladhar & Sivakugan 2020)). It was found the 'standard' mix design with 40% replacement exceeded the specified 28-day characteristic strength (32 MPa), reaching 33 MPa at 28 days. The TMR mix design at 40% replacement achieved 28 MPa at 28 days, while the 60% RCG mix achieved 25 MPa at 28 days, both lower than the specified characteristic strength. It was noted that the workability of the 60% fine RCG mix was undesirable, and the finished surface was rough in appearance. Workability was reportedly acceptable for the 40% fine RCG mixes. Delayed setting was reported for the 40% replacement TMR mix, with a set time of around 7 hours observed (Tamanna, Tuladhar & Sivakugan 2020).

A footpath trial at the Boral quarry site in South Sydney used 45% fine RCG as a fine aggregate replacement and found that compressive strength at 28 days met the 35 MPa target, though flexural strength did not meet the 4.5 MPa target at 28 days. Flexural strength did however reach up to 6 MPa at 90 days. A reduction in abrasion resistance was reported, though it was reported that abrasion resistance increased once the coarse aggregate was exposed (Dumitru et al. 2013).

A notable exception to the footpath trend is the trial undertaken by Lend Lease on the Woolgoolga to Ballina section of the Pacific Highway upgrade, where up to 15% of sand was replaced by fine RCG for the concrete roadway. The fine RCG was sourced from the Lismore City Council materials recovery facility and underwent washing (single wash) in an onsite batch plant prior to use in concrete. The final report for this project concluded that the 10% and 15% fine RCG mixes tested showed no significant change in water demand, 7/28 day compressive and flexural strengths, set time or abrasion resistance. Slight decreases were observed for plastic and hardened density. A significant reduction in drying shrinkage was also observed with increasing RCG content.

An RCG footpath trial in St Lucia, Brisbane was undertaken by UQ in 2018. This trial used 60% fine RCG and 25% coarse RCG and had a specified characteristic compressive strength of 32 MPa. It was noted that the required finish was difficult to achieve due to the high maximum aggregate size. Monitoring for this trial is set to continue for five years (unpublished report).

Field trials in Cairns and South Sydney have shown that it may be practical to increase the replacement level of fine RCG to 40% for footpath applications (above the 30% limit recommended based on previous experience). It should be noted that the 40% fine RCG mix that used TMR mix design in the Cairns field trial did not meet the required 28-day characteristic compressive strength, which was speculated to be linked to the presence of Sika Retarder-N (50–100 ml/100 kg), but further work is required to definitively identify the cause and whether there is a barrier to use of the higher RCG replacement percentage. Higher replacement levels of RCG generally lead to reductions in workability, which may lead to difficulties in achieving the required finish.

2.5 Reclaimed Aggregate (RA)

2.5.1 Overview of RA and Current State of Practice

RA is produced from fresh concrete which has not been used at the jobsite and is returned to a concrete batching plant. It is produced by separating the aggregates from the water-cement slurry through either water washing or use of chemical admixtures which absorb the free water in the mix (Gunasekara et al. 2020, Kazaz, Ulubeyli & Atici 2018)

Water washing is the oldest technology which has been used for the separation of aggregates. RA produced by this method in principle contains the coarse aggregate and sand in the original mix; however, these materials may become segregated during the washing process and should be checked for uniformity in particle grading before use. Developments in recovery technology, e.g. the recovery system of CONSEP®5000 (WAM Group 2018), which transports the washed aggregate by using a spiral shaft without separating the aggregate into different size groups, improves the homogeneity of the collected aggregate as well as recovery efficiency. Depending on the effectiveness of washing, the RA produced may have physical properties similar to the original aggregate. It is however likely that the aggregate produced will contain some fraction of cement mortar similar to that included in RCA, which will impact upon the physical properties.

Another technique is to treat the fresh mix with admixtures, e.g. Re-Con Zero Evo (Mapei 2019), or Cycrete (Gunasekara et al. 2020), which are hygroscopic materials that stabilise the fresh mix and absorb the free water. In this case, the material produced primarily consists of the original aggregate and hydrated cement paste. Once the RA material has been produced through use of the chemical admixtures, it needs to be spread out and moved at least once prior to stockpiling. Ferrari and Brocchi (2012) advises that piles of RA should be moved with a grab within the first 24 hours to break the bonds of hydrated cement paste. For Re-Con Zero Evo, a maximum handling time of 5 hours at 25 °C (reducing to 2.5 hours at 40 °C) is specified for the produced RA, stating that the produced material must be moved with a mechanical shovel or similar within this timeframe.

Returned or over-ordered concrete may also be allowed to harden and crushed by the concrete producer, resulting in an RCA product (Section 2.3). This approach is not discussed further in this section since the expected performance and durability of RCA have been discussed in the previous Section.

2.5.2 Mechanical Properties of RA

Limited research has been conducted on the mechanical properties of RA, but it appears to have similar properties to RCA in terms of density and water absorption. This is likely due to the presence of cement mortar in the aggregate material which is porous and increases the water absorption. The data reported by Gunasekara et al. (2020) indicated a water absorption close to 7% which is well above the MRTS70 (11/2018) guideline, but roughly in line with water absorption limits recommended by HB-155:2002, and typical results for RCA (Section 2.3.2). Water absorption of this magnitude may have an impact on concrete durability, though this has not been widely investigated in the available literature for RA (see Section 2.6.6).

Aggregate grading has been found to be an issue when using reclaimed aggregates, with grading for the RA tested by Gunasekara et al. (2020) falling outside of the target curve. Concrete produced using 100% RA did not meet the target compressive strength at 28 days. It was found that blending coarse RA with suitable natural coarse aggregate could produce suitable grading, and concrete that met the required compressive strength. This was achievable with the use of up to 91% coarse RA content by volume blended with natural basalt aggregate.

Limited information has been found on the other aggregate properties specified by AS 2758.1:2014 and MSTS70:2018. From the available data, produced RA appears to meet the requirements for Los Angeles value, but may not meet the Micro-Deval test limits. Based on this, RA may not be suitable for applications where abrasion and aggregate polishing are specific concerns, such as road pavements.

Table 2.5: Standard limits for aggregate properties vs. reported RA properties

Property	AS 2758.1:2014	MRTS70 (11/2018)	HB 155:2002	RA (coarse)	RA (fine)
Wet strength	• ≥ 50 kN for exposure class A1, A2 • ≥ 80 kN for exposure class B1, B2 ≥ 100 kN for exposure class C	≥ 110 kN		No data	No data

Property	AS 2758.1:2014	MRTS70 (11/2018)	HB 155:2002	RA (coarse)	RA (fine)
Wet/dry strength variation	• ≤ 45% for exposure class A1, A2 • ≤ 35% for exposure class B1, B2 ≤ 25% for exposure class C	≤ 35%		No data	No data
Weak particles	≤ 0.5% (coarse aggregate) • Other limits of weak particles may be adopted based on demonstration of successful performance.	≤ 0.5%		No data	No data
Water absorption (%)	No hard limit. Recommended that aggregates with high absorption are pre-wetted prior to mixing.	≤ 2.5%	• Class 1A: ≤ 6% • Class 1B: ≤ 8%	• 6.9% (25 MPa source) (2) • 6.6% (40 MPa source) (2) • 2.8% (1)	• 3.5% (1)
Degradation factor (fine aggregate)	≥ 60	≥ 50		No data	No data
Density	≥ 2.1, < 3.2 t/m³ (normal weight aggregate)	≥ 2.1, < 3.2 t/m³	• Class 1A: ≥ 2100 kg/m³ • Class 1B: ≥ 1800 kg/m³	No data	No data
Chloride content	Combined chloride salt content < 0.04% in reinforced concrete Total water-soluble chloride salt content < 0.03%	Report content as per AS 2758.1		0.008% (1)	
Sulphate content	Sulphate content of concrete mix ≤ 5% by mass of Portland cement	Report content as per AS 2758.1		0.35% (1)	
Deleterious fines index	≤ 150	≤ 150		No data	No data
Micro-Deval loss	Not referenced	≤15%		• 31% (4/10 mm) (1) • 21% (10/20 mm) (1)	
Soundness	6% (Exposure class C)	6%		No data	No data
Organic impurities	Colour shall not be darker than standard colour of reference solution. Performance of suspect fine aggregate may be verified by comparing performance to satisfactory fine aggregate.	Negative		Negative	
Sugar content	Test negative to presence of sugar based on AS 1141.35	As per AS 2758.1		No data	No data
Light particles	≤ 1% (fine) ≤ 3% for vesicular aggregates	≤ 1% (fine)		No data	No data
Material finer than 75 µm	• Coarse: ≤ 2% Fine: ≤ 20%	• Coarse: ≤ 2% • Fine: ≤ 20%		No data	No data
Material finer than 2 µm	≤ 1%	≤ 1% (fine)		No data	No data
Flakiness index	≤ 35%	≤ 30%		No data	No data

Property	AS 2758.1:2014	MRTS70 (11/2018)	HB 155:2002	RA (coarse)	RA (fine)
Los Angeles value	Maximum percent loss is 35% and 30% for concrete exposure Class A and B, and C, respectively.	Not referenced.		• 22.4% (25 MPa source) (2) • 25.7% (40 MPa source) (2) • 26.0% (1)	
ASR reactivity	Test in accordance with AS 1141.60.1:2014. Testing to AS 1141.60.2:2014 may also be conducted to determine aggregate classification. SCMs are to be used to mitigate reactivity.	Where aggregate is identified as having potential for AAR in accordance with AS 2758.1, treatment shall be in accordance with SA HB79.		Likely to display similar reactivity to original aggregate, though this has not been verified.	

1. Ferrari and Brocchi (2012).
2. Gunasekara et al. (2020).

2.5.3 Cost

The cost of RA is linked to the cost of the reclaiming equipment, admixtures and the need for any process adjustments to incorporate RA in concrete production. It has been identified that the use and viability of concrete reclaimers is limited by 'high cost of treatment' and lack of space in concrete plants for the reclaimer equipment (Tam & Tam 2007). The actual costs associated with RA production in Australia are not clear.

Tam and Tam (2007) reviewed the economic viability of RA for concrete batching plants in Hong Kong. It was identified that in Hong Kong over-ordering of concrete resulted in an average of 15,659 tonnes of concrete waste to landfill each year, and that the cost of this was roughly HK$125 (AU$20) per tonne, with increases in this price predicted due to increased levies on dumping to landfill. The authors stated that reclaiming aggregate from concrete waste would be more cost effective than dumping the material, with an annual saving of AU$325,000 expected for each batching plant. A review of economic viability for fresh concrete waste retainer technology has also been conducted for Turkey, which concluded that in the most likely scenario of available material, equipment costs and savings, it would take a maximum of 8 years for a plant to recoup the investment in the aggregate reclaimer technology (Kazaz, Ulubeyli & Atici 2018).

The above estimates assume that the reclaimed aggregate will be suitable for use in new construction, and it is unclear to what degree this is true. Depending on the product produced it is possible that RA will be suitable as a partial replacement of natural aggregate only. It is also unclear to what degree the Hong Kong or Turkish observations might apply to Australia in terms of cost of landfilling over-ordered concrete.

A figure of $10/tonne of concrete was quoted for the Mapei Re-Con Zero Evo admixture product at a conference attended by TMR (personal communication with W. Roberts March 2021). The Mapei product has the benefit of not requiring the purchase of special equipment, since the admixture is added directly to the concrete mixer truck containing returned concrete. This means that there are minimal upfront costs, compared to other methods which require investment in specialised machinery.

There is some reason to believe that Australian and Queensland concrete producers could extract a cost benefit from the use of RA given that aggregate material is diverted away from disposal (which costs money) and back into concrete usage. This does not appear to have been directly quantified in the literature, and further industry engagement is recommended to investigate whether the benefits of reclaiming aggregate outweigh the cost to concrete producers.

2.5.4 Availability

The availability of reclaimed aggregate depends primarily on the amount of leftover or over-ordered concrete that is produced by concrete plants. Significant estimates of the amount of over-ordered concrete have been made, e.g. £400 million worth of concrete dumped per year in the UK due to inaccurate orders by construction sites (Kazaz, Ulubeyli & Atici 2018). Another estimate was that 20–80 tonnes of concrete waste is produced by UK concrete plants per month. Over-ordering cannot be relied upon to provide a constant supply of reclaimed aggregate however, since it is dependent on the variability and accuracy of concrete ordering processes.

To make the use of RA viable, it is likely that concrete plants would need to be able to incorporate the use of the material into standard batching and concrete production whenever a suitable amount of the material was available. It may also be viable to stockpile the produced RA for use in certain concrete if it not acceptable to simply incorporate into everyday concrete production. The viability of each approach will largely depend on the preference of the concrete producer.

2.5.5 Performance of RA Concrete

There have been several studies which have reviewed the properties of concrete produced using coarse RA which was in turn produced by several different chemical admixture products that were added to fresh concrete. RA is generally recommended to be used as a partial replacement of natural aggregate, and replacement levels in the literature have varied from 30% up to 90% (Ferrari & Brocchi 2012, Gunasekara et al. 2020).

Fresh properties

Workability has not been reported to be critically impacted by the use of RA, but Gunasekara et al. (2020) report that increased superplasticiser was required to maintain an 80–100 mm slump for the mixes tested (3.6 kg/m^3 for 100% NA vs. 4.4 kg/m^3 for 90% RA concrete, target 40 MPa). This indicates a reduction in workability for the RA concrete, which is manageable using superplasticiser. This is not an unexpected result, since the RA tested had a similar water absorption to RCA (6.85%), which may have reduced the free water in the mix.

Strength and deformation properties

The strength properties of concrete have been found to be not greatly impacted by the use of coarse RA, provided that suitable aggregate grading is achieved through blending with coarse NA (Gunasekara et al. 2020). Up to 90% replacement of coarse NA with coarse RA produced using a hygroscopic chemical admixture (Cycrete) has been found to have a negligible impact on 28-day compressive strength for 40 MPa mixes. Similar results were obtained for 30% replacement of coarse NA with coarse RA produced using a superabsorbent polymer and set accelerator to reclaim aggregate from fresh concrete. Ferrari and Brocchi (2012) reported a negligible change in 28-day compressive strength (36 vs. 36.1 MPa) compared to a control mix.

Splitting tensile and flexural strength were also tested by Gunasekara et al. (2020) and it was found that the splitting tensile strength of the 40 MPa coarse RA concrete reached 4.07 MPa at 90 days. This was marginally higher than the tensile strength estimated by the AS 3600:2018 relationships between compressive and tensile strength. A similar result was returned for flexural strength, with 4.55 MPa reached at 90 days. Elastic modulus was found to be within ±20% of the predicted elastic modulus for 40 MPa concrete at 7, 28 and 90 days. Regarding shrinkage of produced concrete, Gunasekara et al. (2020) measured a drying shrinkage of 505 microstrain at 90 days for the 90% coarse RA mix. It was concluded that the coarse RA showed strength and deformation behaviour comparable to that of concrete produced using 100% NA.

Only two studies could be found in the literature where the mechanical properties of concrete produced using RA from use of chemical admixtures were investigated. The concrete produced using coarse RA as a partial replacement of NA up to 90% by volume showed comparable properties to control mixes for compressive, tensile and flexural strength, as well as elastic modulus. The results should be taken with some caution as it is unsure whether the use of different commercial admixtures or varying preparation processes might impact on the strength and deformation properties of the produced concrete. Coarse RA appears suitable to be used as up to a 90% replacement of coarse NA, but further testing is recommended if a different aggregate reclaiming admixture is to be used.

2.5.6 Durability of RA Concrete

The durability of concrete produced using RA produced with chemical admixtures has been reviewed by Gunasekara et al. (2020) for replacement of up to 90% of natural aggregate. No long-term durability studies were found in the literature. It was found that the water absorption of the 90% coarse RA concrete was ~7% after 7-days but reduced to 3.7% at 90 days which is slightly above the limit for low permeable concrete. This author also tested the average volume of permeable voids (AVPV) and found that a similar pattern existed whereby AVPV was 14.7% at 7 days (poor quality), and 8.7% at 90 days, which classifies the concrete as good quality. Ferrari and Brocchi (2012) also found that chloride resistance was not significantly impacted for a concrete produced using 30% coarse RA.

The authors conclude that the durability of concrete would not be significantly impacted by the use of coarse RA as a replacement for natural coarse aggregate. The tests carried out were limited compared to the scope of testing which has been completed for other recycled aggregates such as resistance to chemical attack, abrasion and other permeability tests. Further testing on recycled aggregate in line with expected usage is recommended to verify expected long-term durability behaviour.

2.6 Ferronickel Slag (FNS)

2.6.1 Overview of FNS and Current State of Practice

Ferronickel slag (FNS) is a type of furnace slag produced as a by-product from the manufacture of ferronickel alloy (Nguyen et al. 2019). In production, FNS forms from the smelting of source ore to obtain nickel alloy (Saha & Sarker 2017). The material remaining once the alloy has been extracted is the FNS. This molten FNS is then removed from the furnace and granulated via either cooling in water or air (Nguyen et al. 2019).

For this process, it is of note that the ferronickel alloy and therefore the FNS can be produced from differing processing methods, including from a variety of ores such as laterite (Nguyen et al. 2019) pentlandite, pyrrhotite, garnierite, millerite, or niccolite (Saha & Sarker 2017). This information is important as significant variation in the physical and chemical properties of the resultant FNS can occur depending on the source ore type, and the smelting and cooling methods adopted (Saha & Sarker 2017). The review presented in this report focuses on a single source of FNS, and comparison between material produced from different ore sources is not discussed.

In the ferronickel smelting industry, it is of interest to find valuable, sustainable uses for FNS, as sizeable amounts of the slag are produced for each tonne of extracted ferronickel alloy. For perspective, one of the largest producers of ferronickel alloy, and therefore FNS, is the Société Le Nickel (SLN) in New Caledonia. SLN produces approximately 12–14 tonnes of FNS for each tonne of nickel and SLN currently hosts 25 million tonnes of stockpiled FNS (Saha, Khan & Sarker 2018). If FNS was to be adopted by Australian concrete producers, the SLN stockpile would provide the primary source of supply, with material shipped from New Caledonia to Australian ports (e.g. Brisbane, Sydney). The product produced by this company therefore forms the focus of the review presented in this report.

FNS is used as a fine aggregate substitute for concrete production in New Caledonia, as well as road base, fill and drainage material. FNS has also found use as a land reclaiming material, and as an SCM which is produced after grinding. Laboratory testing of FNS properties and properties of produced concrete has been undertaken in Australia by Curtin University and University of New South Wales (UNSW).

Investigating the viability of the use of FNS in concrete, as either a fine aggregate replacement or SCM, has potential to achieve sustainable and eco-friendly outcomes on two fronts; reduction of FNS stockpiles, and reduction of natural material use in concrete mixes. Consequently, a body of research has developed which is focused on the investigation of the FNS by-product produced by SLN, which is reviewed in this report, focusing on fine aggregate applications.

2.6.2 Mechanical Properties of FNS Aggregate

In understanding the extent to which FNS is suitable as a concrete aggregate in TMR's concrete mixes, the current requirements for these applications (outlined by MRTS70 (11/2018) and corresponding Australian standards, primarily AS 2758.1:2014) are considered.

The current TMR specification, MRTS70, for concrete highlights that for concrete aggregate used in special class concrete, *recycled concrete aggregates, and synthetic and slag aggregates shall not be used* (MRTS70 11/2018) This requirement does not apply to normal class concrete, but it is important to compare the reported performance properties of FNS in concrete against the existing criteria to verify whether slags such as FNS would be suitable for use in the concrete applications in scope of this review (Section 1.3).

For concrete aggregates, the relevant criteria are summarised in Table 2.6, alongside average FNS research values as reported by CQT Services (2018).

Table 2.6: Standard limits for aggregate properties vs. reported FNS properties[1]

Item	Test method	Description	Average FNS result[2]	Acceptance criteria	Criteria reference
Bulk density	AS 1141.4	Compacted (t/m^3)	1.66	–	AS 2758.1
		Uncompacted (t/m^3)	1.46	–	AS 2758.1
Particle density	AS 1141.5 and/or AS 1141.6.1	Apparent (t/m^3)	2.92	–	AS 2758.1
		Dry (t/m^3)	2.87	–	AS 2758.1
		SSD (t/m^3)	2.89	≥ 2.1, < 3.2	MRTS70
		Water absorption (%)	0.66	≤ 2.5%	MRTS70
Particle size distribution (PSD) (Grading)	AS 1141.11.1	Sieve size (mm)			AS 2758.1
		26.5	100	100	
		19	100	100	
		13.2	100	100	
		9.5	100	100	
		6.7	100	100	
		4.75	97	90–100	
		2.36	74	60–100	
		1.18	32	30–100	
		0.6	8	15–100	
		0.425	4	10–60	
		0.3	2	5–40	
		0.15	1	0–25	
		0.075	1	0–20	
Material finer than 75 µm	AS 1141.12	(%)	0.1	–	MRTS70

Item	Test method	Description	Average FNS result[2]	Acceptance criteria	Criteria reference
Water absorption	AS 1141.5	(%)	0.68	≤ 2.5%	MRTS70
Moisture content	AS 1289.2.1.1	(%)	0.5	Nominated	AS 2758.1
Soundness	AS 1141.24	(%)	1.04	≤ 6%	MRTS70
Degradation factor	Q208B (Queensland Department of Transport and Main Roads 2021)	Clear	97	≥ 50	MRTS70
Weak particles	AS 1141.32	(%)	0.0	≤ 0.5%	MRTS70
Clay & fine silt	AS 1141.33	(%)	0.25	–	AS 2758.1
Organics	AS 1141.34	Positive/Negative	Negative	Negative	MRTS70
Sugar content	AS 1141.35	Positive/Negative	Negative	Negative	MRTS70
Iron unsoundness[3]	AS 1141.37	Yes/No	No	No	AS 2758.1
Methylene blue absorption value (MBV)	AS 1141.66	mg/g	2.5	–	AS 2758.1
Deleterious fines index (DFI)	AS 1141.66	DFI	0.25	≤ 150	MRTS70
Chloride content	AS 1012.20.1	(%)	0.001	Report if > 0.01%	AS 2758.1
Sulphate content	AS 1012.20.1	(%)	None	Report if > 0.01%	AS 2758.1

1. Adapted from Table 2 in CQT Services (2018) and Table 7.5.4 in MRTS70 (11/2018).
2. FNS values from CQT Services (2018) report.
3. Property specific to slag materials which indicates whether slag will disintegrate when immersed in water. Iron unsoundness applies if ferrous oxide (FeO) content is > 3% and sulfur (S) content is ≥ 1%.

From Table 2.6 it is seen that the FNS aggregate complies with all listed property requirements for concrete aggregate, except for the recommended grading criteria at sieve sizes 0.600 mm, 0.425 mm, and 0.300 mm (indicated in red text in Table 2.6). Based on these observations, the FNS may be appropriate for use as a coarse sand product, provided that it is combined with a suitable natural or manufactured fine sand.

Additionally, MRTS70 (11/2018) also indicates that the sulphate and chloride content of a concrete aggregate are to be reported as per test results from AS 1012.20.1 with specified limits for chloride content set as follows (MRTS70 11/2018):

- For exposure classes B2 or less:
  - Reinforced hardened concrete chloride content limit of ≤ 0.80 kg/m^3
  - Prestressed hardened concrete chloride content limit of ≤ 0.60 kg/m^3
- For exposure class C (classifications C1 and C2):
  - Chloride content limit of ≤ 0.40 kg/m^3.

The content for chloride and sulphate in FNS is sourced from testing of ground FNS (GFNS) which is intended for use as an SCM. It is considered that a similar result could be obtained from testing of FNS fine aggregate.

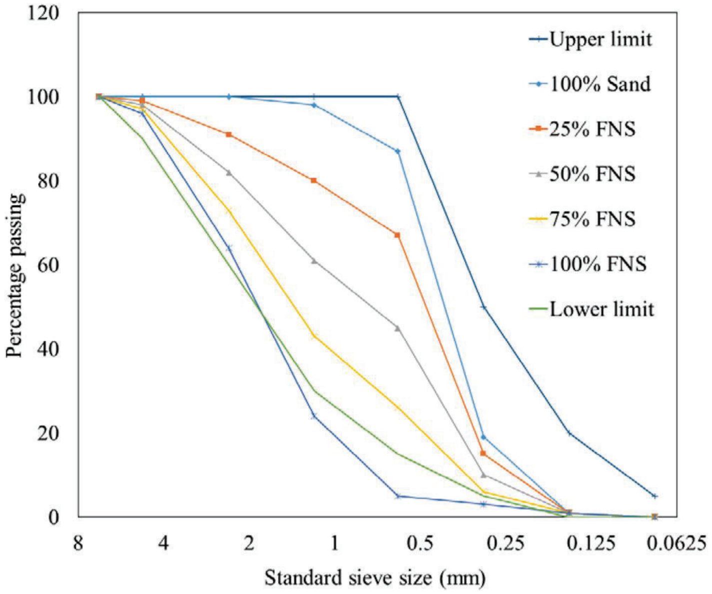
As a concrete aggregate, FNS appears to align with all specified requirements except for a notable lack of fines in the grading. In literature (Saha & Sarker, 2019), it was found that while the FNS had less fines than required, it had a higher proportion of larger particles than the natural sand aggregate. To address this, Saha and Sarker (2019) used FNS as a partial aggregate replacement only, blending it with the natural sand to obtain an optimum grading (Figure 2.4). This was similarly done in other studies by Saha and Sarker (2017 & 2020), as well as by Nguyen et al. (2019).

Based on practice to date and the observations of the researchers, it appears that FNS is most suitable as a partial fine aggregate replacement, with the optimum replacement level set with the aim of achieving satisfactory aggregate grading. CQT Services (2018) discussed several theoretical mix designs which were based on aggregate product available in Sydney NSW. These included:

- 50% FNS, 50% natural Sydney sand (grading not consistent with standard industry practice)
- 12% manufactured sand, 35% FNS, 53% natural Sydney sand (lower FNS, better grading) (10 mm max aggregate size)
- 12% manufactured sand, 44% FNS, 44% natural Sydney sand (higher FNS, lower 0.6 mm size fraction but still acceptable) (10 mm max aggregate size)
- 11% manufactured sand, 25% FNS, 64% natural Sydney sand (20 mm max aggregate size)
- 11% manufactured sand, 35% FNS, 54% natural Sydney sand (20 mm max aggregate size).

It was concluded that up to 35% inclusion of FNS in concrete could be achieved if blending with manufactured sand and natural sand. The inclusion of FNS reduces the fines in the mix, but it was considered that the inclusion of manufactured sand could offset this impact, provided that adequate fines are provided in the 0.6 mm, 0.425 mm and 0.3 mm fractions.

Figure 2.4: Particle size distribution of natural sand, FNS, blends thereof and the upper and lower specified limits as per AS 2758.1



Source: Saha and Sarker (2019).

2.6.3 Cost

Research has shown that the most effective use of FNS fine aggregate is to blend with natural sand and manufactured sand to achieve optimum aggregate grading. This adds an extra material to the concrete production process, which may lead to some increase in costs, although blending natural sand with manufactured sand is standard practice in Queensland and would require storage for an additional material. Further discussion with concrete producers is recommended with the aim of capturing sentiment regarding the addition of an extra material and expected costs.

The main FNS stockpile (25 million tonnes) is in New Caledonia, which is roughly 2000 km from Brisbane across the sea (CMS Consulting & Société Métallurgique le Nickel 2020). The primary costs associated with FNS procurement are therefore likely to be associated with shipping, transport and storage. The material is a waste by-product of the ferronickel smelting process, meaning that there is incentive for SLN to offer a price-competitive material.

2.6.4 Availability

SLN currently has a stockpile of FNS in New Caledonia which is at least 25 million tonnes in size. Furthermore, the ferronickel smelting operation produces 1.6 million tonnes/year of FNS as an industrial by-product (CMS Consulting & Société Métallurgique le Nickel 2020). This material is reportedly consistent and homogenous, meaning that it is suitable for incorporation in blended aggregate with minimal additional processing.

There are facilities in New Caledonia to facilitate ship loading (600 tonne/hour) and transport of the FNS material to overseas destinations. Ships of up to 35,000 tonne capacity are available to transport the material. Specific details relating to transport capability would require discussion with SLN. At the very least, facilities in Australia would need to be developed to support economically viable unloading, storage, and distribution of FNS for use as fine aggregate.

There appears to be reasonable availability of the FNS fine aggregate product which is suitable for partial replacement of natural fine aggregate in concrete. To facilitate use in TMR concrete production, it may be preferable to create an SEQ stockpile which is either owned by or can be readily accessed by concrete producers.

2.6.5 Performance of FNS Concrete

The following sub-sections provide a summary of the literature findings around the use of FNS aggregate in concrete mixes, with an emphasis on the reported mechanical and chemical properties. Such properties covered include the following:

- strength – compressive, tensile, and flexural
- elastic modulus
- shrinkage
- creep
- risk of ASR expansion
- chloride penetration
- permeability
- carbonation
- abrasion.

Fresh properties

The literature reviewed indicates that the workability of concrete incorporating FNS would generally be acceptable up to a replacement level of 50% of fine aggregate, with a marginal increase in slump compared to a 100% NA mix noted by Saha, Khan and Sarker (2018). A reduction in workability at 100% inclusion of FNS was also noted by this study, indicating that exceeding 50% inclusion may result in unsatisfactory properties.

CQT Services (2018) also note that the studies undertaken by Curtin University showed that the concrete mix with 50% FNS inclusion was the most workable. However, it is also noted that the grading of mixes containing 50% FNS was not consistent with standard commercial mix grading, which may lead to issues with bleeding and finishing. To address this, it was proposed to limit FNS inclusion to 35%, and blend with natural sand and manufactured sand to achieve suitable grading. These mix designs have not been trialled in Australia, and trialling with concrete producers is recommended prior to setting hard limits on inclusion.

Overall, it appears that suitable workability of FNS concrete can be achieved through careful mix design. It appears likely that inclusion of FNS in concrete beyond 50% of fine aggregate would result in unsatisfactory workability.

Strength properties

The compressive, tensile and flexural strength of the concrete and mortar mixes containing FNS as either a fine aggregate and/or an SCM is reported here alongside a discussion of the impact of FNS on the strength outcomes for the tested specimens.

The strength testing undertaken on the specimens prepared in each reviewed study have been collated in Table 2.7. A summary of these results is discussed below for both the concrete mix outcomes and the mortar mix outcomes.

Table 2.7: Strength property results reported in literature for FNS-containing concretes and mortars

Mix type	Specimen mix description	Aggregate materials[1]	Binder materials[1]	Compressive strength[2] (MPa)	Tensile strength[2] (MPa)	Flexural strength[2] (MPa)
Concrete (32/30 MPa) (CQT Services (2018)/ Nguyen et al. (2019))	Control_OPC100	Crushed basalt (coarse) Natural sand (fine)	100% OPC	36/35	3.5/3.5	6.09
	Control_F20		80% OPC 20% FA	N/A/30.5	N/A/2.9	6.86
	Control_F25		75% OPC 25% FA	45/N/A	2.9/N/A	5.74
	FNS_F25	Crushed basalt (coarse) 50% Natural sand (fine) 50% FNS (fine)	75% OPC 25% FA	46/32.8	3/3.4	5.25
Concrete (50 MPa) (Saha & Sarker (2017))	PC-FNS0	Granite (coarse) Natural sand (fine)	100% OPC	61	4.33	6.23
	PC-FNS50	Granite (coarse) 50% Natural sand (fine) 50% FNS (fine)	100% OPC	66	4.73	4.96
	PC-FNS100	Granite (coarse) FNS (fine)	100% OPC	45	3.94	
	FA-FNS0	Granite (coarse) Natural sand (fine)	70% OPC 30% FA	39	3.85	
	FA-FNS50	Granite (coarse) 50% Natural sand (fine) 50% FNS (fine)	70% OPC 30% FA	51	4.44[3]	
	FA-FNS100	Granite (coarse) FNS (fine)	70% OPC 30% FA	36	3.66[3]	

1. Proportions of materials are by weight.
2. At 28-days curing.

The studies from both universities, as reported in CQT Services (2018), Nguyen et al. (2019) and Saha and Sarker (2017) looked at concrete mixes using FNS as a fine aggregate replacement in varying percentages.

For the UNSW study (CQT Services 2018 and Nguyen et al. 2019), a concrete grade of ~30 MPa was targeted with control specimens containing natural aggregates with either 100% OPC binder, or a combination of OPC and FA in the binder, and a test specimen containing 50% FNS fine aggregate as a partial sand replacement with a 25% FA + 75% OPC binder. The compressive strengths measured for these specimens show that the partial (50%) replacement of natural sand aggregate with FNS gives some improvement to the compressive strength of the concrete. While this strength gain is minimal, it is noted that the use of 50% FNS aggregate did not subtract from the 28-day compressive strength of the concrete.

For the CU study (Saha & Sarker 2017), a concrete grade of 50 MPa was targeted with control specimens containing natural sand aggregate with either a 100% OPC binder or a blended binder containing 70% OPC and 30% FA, and test specimens containing either 50% FNS fine aggregate with 50% natural sand aggregate, or 100% FNS fine aggregate. The compressive strength testing of these specimens provided a similar outcome to that seen in the UNSW study whereby the partial replacement of natural sand with 50% FNS fine aggregate prompted a marginal increase in concrete's compressive strength. It was also concluded that a 100% replacement of fine aggregate with FNS significantly decreased the compressive strength as for the UNSW study.

Saha and Sarker (2017) also noted that for specimens containing FA the resultant strength was lower than those without FA in the binder. The w/c ratio was kept constant for all mixes (0.33), and it is noted that this strength reduction is typical when using fly ash without adjusting the water/binder ratio (Saha & Sarker 2017). This effect was not seen in the UNSW study where the Control_F25 specimen was seen to be stronger than the Control_OPC100 specimen (w/c 0.55), although this is likely due to the decreased water/binder ratio of the Control_F25 mix (w/c 0.45). These observations indicate that a reduction in w/c ratio is required when including fly ash in FNS concrete, i.e. the combined proportion of OPC and FA should be higher than the proportion of OPC which would be included in a comparable non-FA mix.

The reasoning given for the increased strength of a 50% FNS replacement of fine aggregate is that an improved grading (Figure 2.4) and therefore, packing of particles (increasing density and strength) was achieved when the natural sand and the FNS were blended (CQT Services 2018). The reduced strength seen in the 100% FNS fine aggregate specimens can be attributed to the lack of fines and more coarse particles in the FNS. This distribution leads to an increased voids content and poorer packing of the aggregate particles, resulting in a less dense and weaker concrete (Saha & Sarker 2017).

The tensile strength outcomes gained from both studies showed similar patterns to those seen for the compressive strength of concrete mixes, with the control mix showing the highest strength and the mixes with FA or GFNS SCMs showing lower strengths but behaving similarly to each other (CQT Services 2018).

CQT Services (2018) and Nguyen et al. (2019) report the tensile strength for the FNS_F25 specimen was similar to that for the control_OPC100 specimen, and greater than that for the FA controls (control_F20 and control_F25). However, for the UNSW study CQT Services (2018) reports very marginal difference in tensile strength increase for the 50% FNS aggregate specimen over the FA control with the same amount of SCM in the binder, while Nguyen et al. (2019) reports a higher tensile strength value for the 50% FNS aggregate specimen, much closer to the strength of the OPC control.

Nguyen et al. (2019) also indicates that the higher tensile strength seen for the specimen with 50% FNS as a replacement for natural sand, than that without FNS, could be attributed to the lower water/binder ratio and/or to the contribution of the FNS in improving concrete strength. In regard to the latter concept, the interfacial transition zone (ITZ) of the specimen's microstructure was examined as this zone is an important factor in the mechanical and durability properties of concrete (Nguyen et al. 2019). For the specimen containing FNS aggregate, the ITZ may be improved due to the lower water absorption of FNS (0.79%) compared to that for natural sand (3.5%). This leads to less interference with cement hydration in the ITZ. Additionally, the shapes of the FNS aggregate particles are irregular and highly angular which may also play a part in enhancing the mechanical properties by strengthening the bond between the aggregate and the cement matrix in the concrete (Nguyen et al. 2019).

For the CU study, Saha & Sarker (2017) measured the tensile strength of the concrete specimens with and without inclusion of FA in the binder, exploring the effect of 50% and 100% FNS aggregate contents in place of natural sand. The trend seen here for tensile strength is similar to that seen for compressive strength with an increase in strength gained with an increase in FNS aggregate content up to 50%, followed by a decrease in strength at 100% FNS aggregate content. This decrease is again attributed to the lack of fine particles in the FNS aggregate grading, which would otherwise allow for improved packing and thus cohesion in the concrete mix. However, Saha & Sarker (2017) note that the percent decrease in tensile strength with 100%

FNS replacement of sand is less than that seen for compressive strength. This is attributed to the shape of the FNS aggregate which is angular with rough surfaces, allowing better interlocking and thus a higher resistance to tensile splitting. Furthermore, the percent increase in tensile strength for the 50% FNS aggregate specimens over the respective control mixes, show a higher increase for the series also containing FA in the binder (Saha & Sarker (2017)).

For flexural strength, the concrete specimens described in Saha and Sarker (2017) display the same pattern as seen for both compressive and tensile strength. Again, it is noted that the strength (flexural in this instance) increases with 50% FNS aggregate replacing natural sand, but then decreases with the full replacement of sand with 100% FNS. It is also highlighted in Saha and Sarker (2017) that the high angularity and therefore better interlocking of the FNS particles acted to improve the flexural strength of the specimens, much like for tensile strength, showing a lower percentage decrease in strength for 100% natural sand replacement with FNS than seen in the results for compressive strength. Furthermore, the percent increase in flexural strength for the 50% FNS aggregate specimens over the respective control mixes, again shows a higher increase for the series also containing FA in the binder (Saha & Sarker 2017), similar to that seen for tensile strength.

Overall, the incorporation of up to 50% FNS fine aggregate as a partial replacement of natural sand was seen to either maintain or improve the compressive, tensile and flexural strength of the concrete test specimens compared to 100% NA mixes. Additionally, the use of FNS aggregate in conjunction with an FA SCM in concrete mixes, can allow for improved flexural strength increases over FNS aggregate mixes without an SCM. It should be noted however, that the inclusion of FA requires a reduction in overall w/c ratio to maintain equal strength to a comparable 100% OPC mix.

Deformation properties – elastic modulus

The elastic moduli measured for each concrete specimen prepared in the UNSW and CU studies have been provided in Table 2.8. A summary of these results is discussed below for the set of concrete mix outcomes as reported by CQT Services (2018), Nguyen et al. (2019), and Saha and Sarker (2017).

Table 2.8: Elastic modulus results reported in literature for FNS containing concretes

Mix type	Specimen mix description	Aggregate materials[1]	Binder materials[1]	Elastic modulus[2] (GPa)
Concrete (32/30 MPa) CQT Services (2018)/ Nguyen et al. (2019)	Control_OPC100	Crushed basalt (coarse) Natural sand (fine)	100% OPC	28/28.6
	Control_F20		80% OPC 20% FA	N/A/26.8
	Control_F25		75% OPC 25% FA	24/N/A
	FNS_F25	Crushed basalt (coarse) 50% Natural sand (fine) 50% FNS sand (fine)	75% OPC 25% FA	27/27.1

Mix type	Specimen mix description	Aggregate materials[1]	Binder materials[1]	Elastic modulus[2] (GPa)
Concrete (50 MPa) Saha and Sarker (2017)	PC-FNS0	Granite (coarse) Natural sand (fine)	100% OPC	42
	PC-FNS50	Granite (coarse) 50% Natural sand (fine) 50% FNS (fine)	100% OPC	47
	PC-FNS100	Granite (coarse) FNS (fine)	100% OPC	38
	FA-FNS0	Granite (coarse) Natural sand (fine)	70% OPC 30% FA	36
	FA-FNS50	Granite (coarse) 50% Natural sand (fine) 50% FNS (fine)	70% OPC 30% FA	39
	FA-FNS100	Granite (coarse) FNS (fine)	70% OPC 30% FA	33

1. Proportions of materials are by weight.
2. At 28-days curing.

In the UNSW study, it is shown that the elastic modulus is marginally affected by the partial replacement of natural sand by 50% FNS aggregate, especially in comparison to the compressive strengths of the specimens. Furthermore, Nguyen et al. (2019) outlines that the elastic modulus was also not significantly affected by the use of FA in the binder as compared with the compressive and tensile strengths. Nonetheless, the results for the UNSW study do indicate that the use of 50% FNS aggregate in combination with 25% FA gives the highest modulus and the one most similar to that seen for the Control_OPC100 specimen.

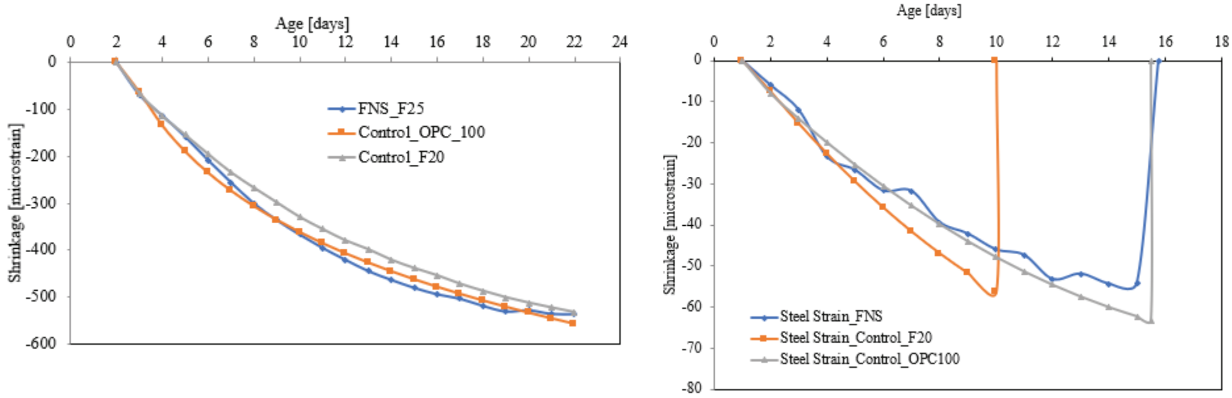
For the CU study, Saha and Sarker (2017) note that the elastic modulus is partly dependent on the concrete density, aggregate properties, and the compressive strength of the concrete. In considering the recorded moduli for this study, the mixes with 50% FNS aggregate are seen to give the highest modulus (similar to the results for concrete strengths discussed previously). This is attributed to the improved grading achieved when combining the FNS and natural sand aggregates, which provides a lower void content and improves the density of the mix to give a higher elastic modulus (Saha & Sarker 2017). In the same way, the use of 100% FNS fine aggregate, results in more voids and a lower density, giving a reduction in elastic modulus. It was further noted by Saha and Sarker (2017) that using FA in combination with FNS aggregate, works to further improve the packing of particles, and thus can increase the modulus of mixes with a higher FNS content.

Overall, similar trends to those seen for concrete strength (compressive, tensile and flexural) were also noted for the elastic moduli of the concrete mixtures. For this, the use of 50% FNS as a partial replacement for natural sand aggregate provided the highest modulus for both the OPC and FA binder series. Similar to the strength observations, a reduction in elastic modulus was observed for the 30% FA mixes compared to the 100% OPC mixes, which is likely due to the w/c ratio design as discussed previously. The inclusion of up to 50% FNS in concrete is not expected to have a detrimental impact on elastic modulus.

Deformation properties – shrinkage

In assessing the effect of FNS on shrinkage, the UNSW study undertook free shrinkage dog-bone tests, and restrained and unrestrained shrinkage ring tests, as well as a concrete prism test to determine autogenous shrinkage. The results from these are reported by Nguyen et al. (2019) and CQT Services (2018) and are discussed below as well as depicted in Figure 2.5.

Figure 2.5: UNSW study – shrinkage test results

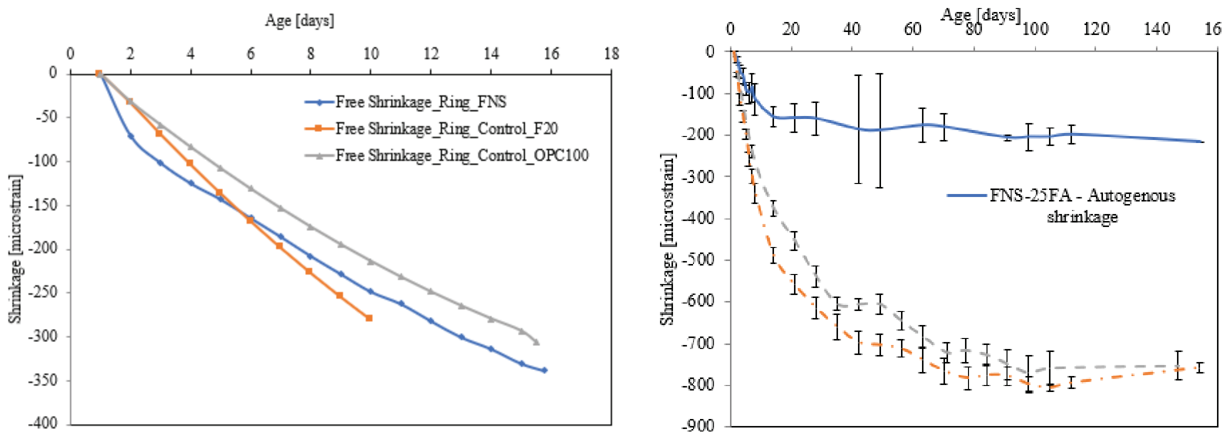


a) Free shrinkage (dog-bone) test

b) Restrained shrinkage ring test

c) Unrestrained shrinkage ring test

d) Autogenous shrinkage (alongside dry and total shrinkage) concrete prism test



Source: CQT Services (2018).

For free shrinkage using the dog-bone specimens, Figure 2.5a shows and Nguyen et al. (2019) reports that at 21 days, the FNS_F25 (50% FNS, 25% FA) (-535 με) mix was seen to behave similarly to the control mixes (-523 με for control_F20, and -546 με for control_OPC_100), and concludes that the incorporation of FNS fine aggregate does not influence the free shrinkage strain of the concrete mix.

For restrained shrinkage, Nguyen et al. (2019) reports that while the use of FA raises the risk of early-age cracking, the partial replacement of natural sand by 50% FNS aggregate significantly improved performance of the concrete containing FA in the binder. As per Figure 2.5b, the mix containing 50% FNS aggregate also showed a slightly lower shrinkage than both control mixes, and a comparable life to that seen for the OPC control mix. Meanwhile, the unrestrained shrinkage seen in Figure 2.5c for the mix containing 50% FNS aggregate is seen to be higher than both control specimens. However, the FNS mix also demonstrated the longest life before cracking occurred. Figure 2.5d shows that autogenous shrinkage is relatively constant over time for the FNS_F25 mix, following initial shrinkage in the first 10–15 days, which is favourable.

Overall, the inclusion of FNS in concrete appears to result in shrinkage similar to that seen for control mixes. Therefore, it is expected that FNS would not have a significant detrimental impact on concrete shrinkage at fine aggregate replacement levels up to 50%.

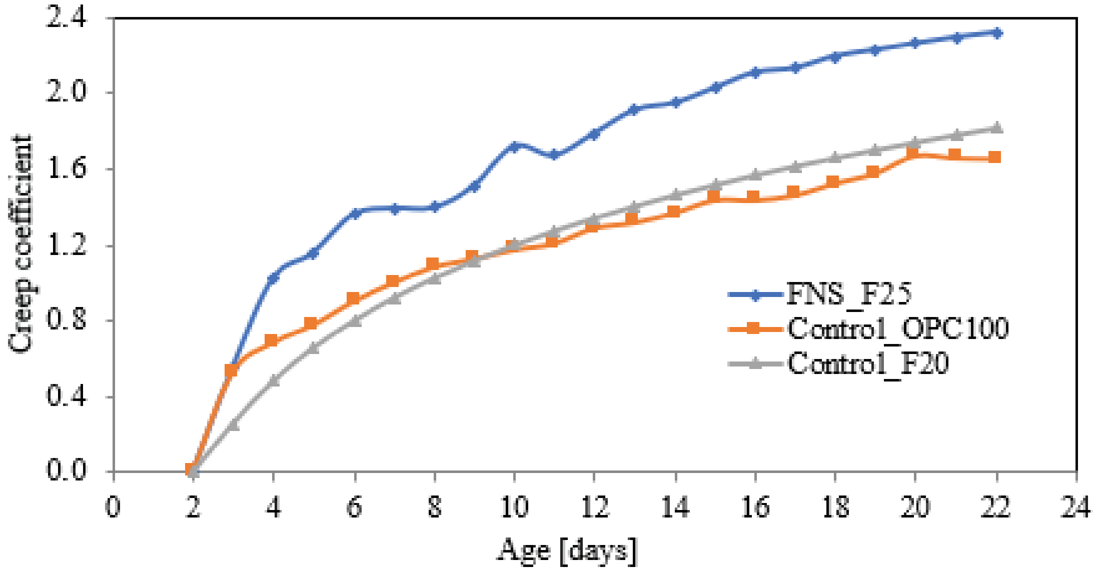
Deformation properties – creep

The tensile creep of control concrete mixes and a mix containing a partial replacement of natural sand with 50% FNS aggregate was measured in the UNSW study as reported by both CQT Services (2018) and Nguyen et al. (2019). For this, the evolution of creep was measured through the application of sustained axial tension to dog-bone shaped specimens for three of the UNSW mixes (FNS_C25, control_OPC100, and control_F20).

The measured creep coefficients are depicted in Figure 2.6 from CQT Services (2018), with values provided by Nguyen et al. (2019) for each mix, at 21.2 days as follows:

- FNS_F25 creep coefficient = 2.29
- Control_OPC100 creep coefficient = 1.61
- Control_F20 creep coefficient = 1.78.

Figure 2.6: UNSW study – creep coefficient results



Source: CQT Services (2018).

From this, it is seen that the mix containing 50% FNS fine aggregate presents the highest tensile creep coefficient. Nguyen et al. (2019) notes that this is a positive characteristic as high tensile creep aids to relax undesirable stresses in the concrete that may arise from the effects of shrinkage, thermal gradients, and support-restraint.

The compressive creep of FNS concrete is not discussed by the papers reviewed, but due to the non-structural scope of this project, this is not considered to be of serious concern. The tensile creep data reviewed does not appear to show any factors which would be detrimental to the use of FNS in non-structural TMR N-Class concrete.

2.6.6 Durability of FNS Concrete

Permeability

For the mortar specimens produced in the CU study, Saha & Sarker (2019 & 2020) report the volume of permeable voids (VPV) at 28-days of curing (Table 2.9) noting that the VPV is highest in samples without an SCM (i.e. VPV decreased with the use of FA or ground FNS (GFNS) in the binder). This reduction in VPV with SCM use is attributed to the pozzolanic activity of the FA or GFNS which improves the microstructure of the mortar (Saha & Sarker 2020). Moreover, the specimens with GFNS used as an SCM showed a lower VPV than those using FA, which for this study can be attributed to the higher fineness of the GFNS compared to the fineness of the FA (Saha & Sarker 2019). Meanwhile, the use of FNS aggregate resulted in an increase in VPV for the mixes with both binder combinations (FA or GFNS as SCMs) due to the presence of voids in the FNS aggregate (caused by the rapid cooling of FNS from molten slag), and the larger, highly angular particle size and thus poorer packing of the FNS aggregates compared with the natural sand (Saha

& Sarker 2020). The void content in the mortars, and thus the permeability, increased with the increase in FNS aggregate content.

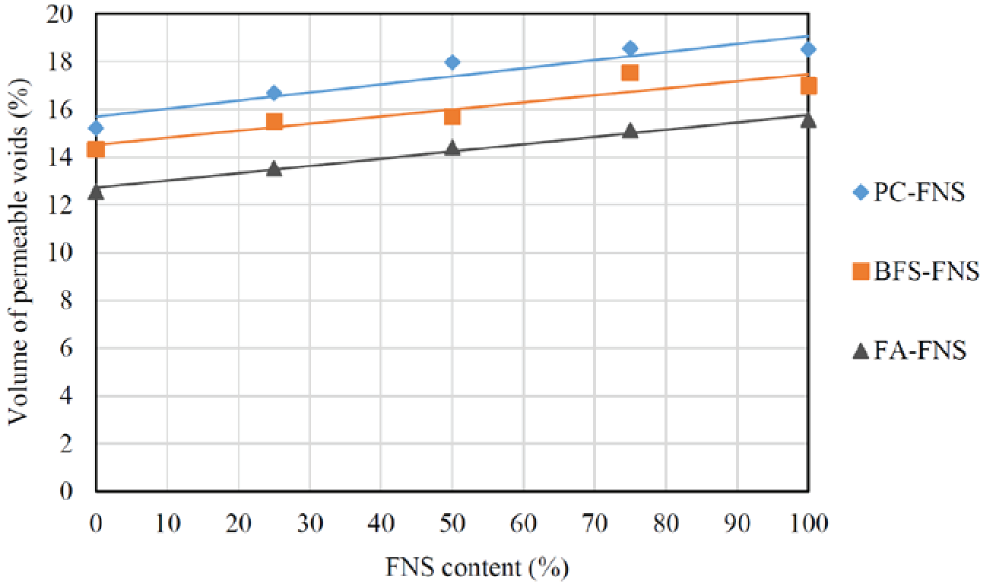
Table 2.9: **Volume of permeable voids (VPV) for specimens with varying FNS and GFNS contents**

Specimen description	Fine aggregate materials[1]	Binder materials[1]	VPV[2,3] (%)
PC-FNS0	Natural sand	100% OPC	14.7
FA-FNS0	Natural sand	70% OPC, 30% FA	10.1
FA-FNS25	75% Natural sand, 25% FNS	70% OPC, 30% FA	11.8
FA-FNS50	50% Natural sand, 50% FNS	70% OPC, 30% FA	12.1
GFNS-FNS0	Natural sand	70% OPC, 30% GFNS	9.8
GFNS-FNS25	75% Natural sand, 25% FNS	70% OPC, 30% GFNS	10.1
GFNS-FNS50	50% Natural sand, 50% FNS	70% OPC, 30% GFNS	11.3

1. Proportions of materials are by weight.
2. At 28-days curing.
3. Reported in both Saha and Sarker (2019) and Saha and Sarker (2020).

CQT Services (2018) also reports on the VPV in mortar specimens with and without FNS fine aggregate, for the CU study. In this, three mortar series were compared, each series containing a different binder type (either OPC, blast furnace slag (BFS), or FA) and spanning several FNS content levels (0, 25, 50, 75, and 100% FNS replacement of natural sand). The results of this are depicted in Figure 2.7, and show that the VPV increases linearly with increasing FNS aggregate content for all binder types, although a reduction in VPV is seen when a typical SCM is utilised. The cause of the VPV increase with FNS content is attributed to the larger, more angular FNS particles which act to increase capillary pores throughout the mortar (CQT, 2018).

Figure 2.7: **CU study – VPV with increasing FNS aggregate replacement of natural sand**

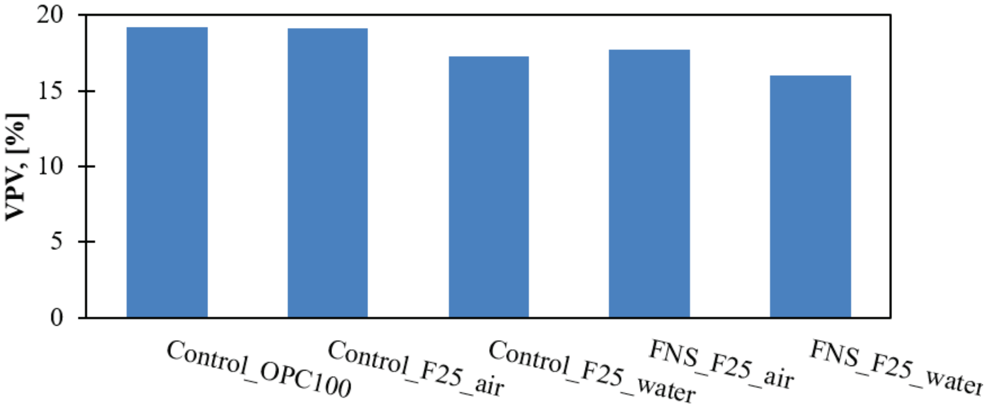


Note: BFS is typical ground granulated blast furnace slag (GGBFS).

Source: CQT Services (2018).

Further to this, CQT Services (2018) also reported on the VPV results for the concrete mixes in the UNSW study. In this study, mixes were cured via two methods: in air, or in water. The comparison between VPV for the control and FNS containing mixes for the UNSW study is depicted in Figure 2.8 where it is seen that the VPV is lowest in the water-cured mix containing both FA as an SCM and a 50% FNS aggregate partial sand replacement content. This may be attributed to the improved grading and packing achieved by blending the finer natural sand with the coarser FNS, alongside including the very fine FA particles.

Figure 2.8: UNSW study – VPV of concrete mixes



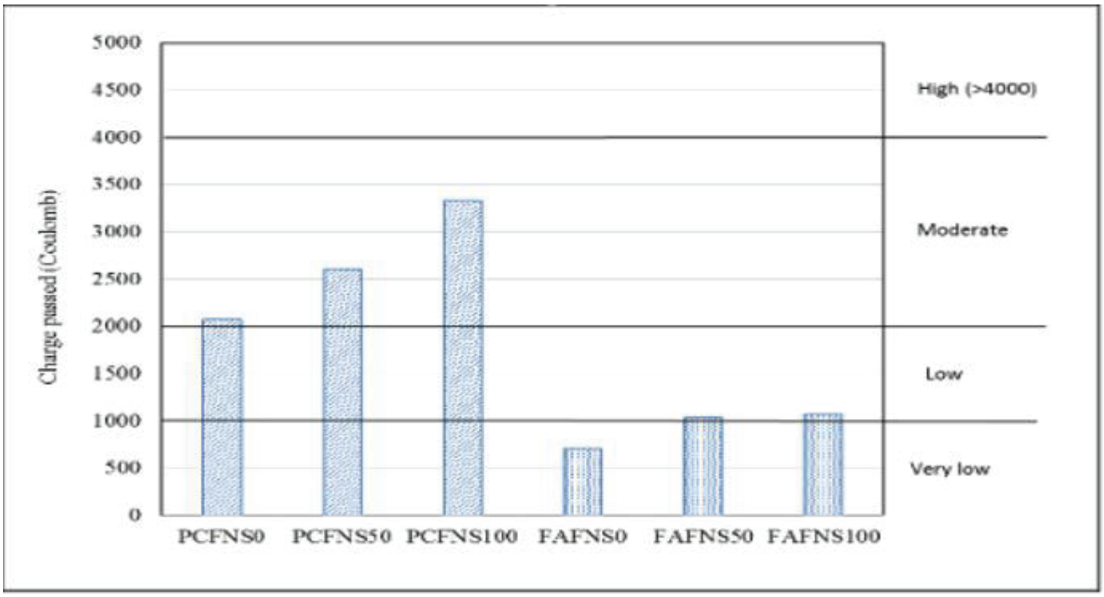
Source: CQT Services (2018).

Overall, the inclusion of FNS as a partial fine aggregate replacement in both mortar and concrete mixes, acts to increase the VPV due to the large angular nature of the FNS particles and the intrinsic pores created in the rapid cooling process for the slag. However, the use of 50% FNS aggregates alongside 25% FA in the binder and use of curing with water for concrete, may provide an overall reduction in VPV (as seen in Figure 2.8). For TMR N-Class concrete used in low-risk applications as per Section 1.3, the use of up to 50% FNS as fine aggregate is likely to be acceptable from a durability perspective.

Chloride penetration

For the CU study, CQT Services (2018) notes that for concrete mixes containing FNS aggregate as a partial replacement of natural sand, the chloride penetration is higher than for mixes without FNS. This is attributed to an increase in concrete porosity from both the large angular shape of the grains and the intrinsic voids in the FNS created when rapidly cooled from its molten state (CQT Services 2018). However, it is also noted that with the incorporation of FA into the binder, the chloride penetration for FNS containing mixes is much lower than for mixes without FA (CQT Services 2018). The chloride permeability levels reported by CQT Services (2018) are depicted in Figure 2.9.

Figure 2.9: CU study - chloride permeability



Source: CQT Services (2018).

In general, without use of an SCM in the binder, inclusion of FNS aggregate in the concrete mix will result in relatively higher chloride permeability, though it may be acceptable for low-risk non-structural applications.

The inclusion of 30% FA reduces chloride penetrability in 50% FNS concrete to low/very low levels which would likely be suitable for aggressive exposure conditions.

Carbonation

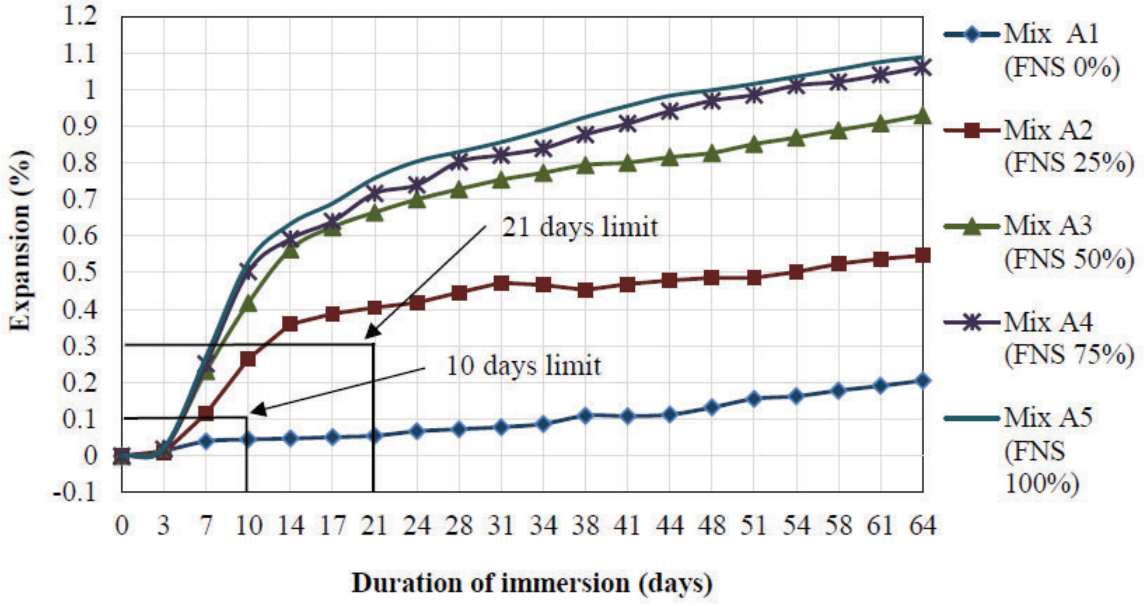
The carbonation of concrete mixes studied by UNSW are reported on by CQT Services (2018). For the mix containing 50% FNS aggregate as a partial sand replacement (FNS_F25), the carbonation levels are seen to be less penetrative than for the mix without FNS (control_F25). This suggests that the inclusion of FNS aggregate in the concrete may increase the resistance to carbonation (CQT, 2018). This may be attributed to the improved grading and particle packing gained when blending FNS with natural sand, as well as improved interlocking of particles given the angular shape and rough surface of the FNS aggregate providing an improved barrier to carbon penetration.

Alkali-Silica Reaction (ASR)

In considering the risk of ASR expansion when using FNS materials, CQT Services (2018) notes that when FNS is produced it contains no free silica to undergo ASR. However, during the quenching process a glass coating forms around the FNS aggregates. CQT Services (2018) states that the use of SCMs such as FA (around 25–30% by mass in the binder) in combination with FNS materials is recommended to ensure mitigation of potential ASR.

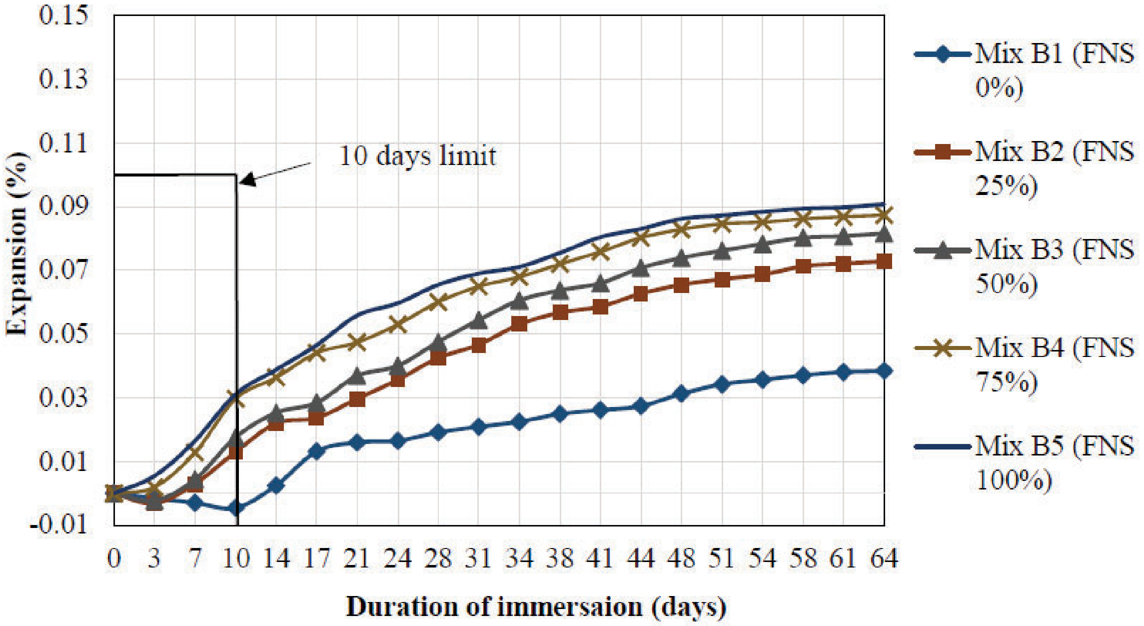
CQT Services (2018) further reports on the accelerated mortar bar tests (AS 1141.60.1:2014) undertaken by CU in exploring the ASR of FNS material in the mortar mixes. The results of this work are depicted in Figure 2.10 and Figure 2.11, and indicate that as levels of FNS replacement increase, the ASR expansion also increases. However, the inclusion of 30% FA saw a significant reduction in ASR expansion, to the point that while all 100% OPC binder specimens were deemed reactive, those with FA were all able to be deemed non-reactive regardless of FNS content. This is the same result also noted by Saha and Sarker (2017 & 2020), whereby the use of 30% FA was able to successfully mitigate the ASR susceptibility of FNS-containing mixes.

Figure 2.10: CU study – ASR expansion of mortars without an SCM (100% OPC binder)



Source: CQT Services (2018).

Figure 2.11: CU study – ASR expansion of mortars without an SCM (30% FA + 70% OPC binder)



Source: CQT Services (2018).

Similarly, for the concrete specimens in the UNSW study, Nguyen et al. (2019) reports that the slightly lower inclusion of 25% FA in mixes containing FNS aggregate showed no signs of ASR gel formation at the FNS-to-cement paste interface. This outcome was desirable as Nguyen et al. had faced difficulties in achieving the required 28-day compressive strength when using 30% FA but were able to achieve a reasonable strength and successfully mitigate ASR expansion with the use of the lower, 25% FA content in the binder.

It can be concluded that the inclusion of at least 25% FA as an SCM in concrete produced using 50% FNS would reduce the potential for ASR expansion to acceptable levels. This is in line with standard TMR practice for S-class concrete, Exposure Class B2, and may be adopted for N-Class concrete if required.

2.6.7 Field Trials

While there do not appear to have been publicly documented field trials of FNS concrete in Australia, the material has been used as a fine aggregate replacement with success in various field applications in New Caledonia. Documented projects include (CMS Consulting & Société Métallurgique le Nickel 2020):

- use in breakwaters and shore protection as concrete blocks (Maré Island – Loyalty Islands)
- material for production of structural blocks (Besser blocks)
- construction of electrical switch room (foundations, walls, slab).

Figure 2.12: Accropode breakwater blocks in situ

Source: CMS Consulting & Société Métallurgique le Nickel (2020).

The breakwater application has been in place for at least 20 years prior to 2020, and no evidence of premature deterioration has been detected on the Accropode blocks which have been tested (no silica gel or ettringite). The electrical switch room was constructed more recently using 35% inclusion of FNS as fine aggregate and 340 kg/m^3 of cement (target strength 42 MPa, slump > 160 mm, concrete grade C30/37, XS2 exposure class as per EN 1992-1-1). Good workability was reported for this concrete mix.

Apart from the points identified above, the literature reviewed was light on details regarding the performance of FNS concrete field applications in New Caledonia. The performance of FNS concrete appears to have been satisfactory over a mid-length period (20 years), but further review of field trial details is recommended if these details are available. Trialling in TMR N-Class (non-structural) applications may also be beneficial to capture any potential issues relating to mix design or concrete production.

2.7 Power Station Bottom Ash (BA)

2.7.1 Overview of BA and Current State of Practice

Power station bottom ash (BA) is a coal combustion product (CCP) which is produced (together with fly ash and other materials such as boiler slag) as a by-product of coal-powered furnaces at coal-fired power stations. BA accounts for roughly 15% of total CCPs produced annually in QLD, equating to roughly 798,715 tonnes/year of BA. Over 1 million tonnes/year of BA are produced in NSW, and there are millions of tonnes of the material stockpiled (generally in ash dams) in QLD and NSW. Ash dams have the potential to leach heavy metals into the soil and water table, and reduction in the volume of material entering these dams would have a positive environmental impact.

BA is generally a sand-sized material, and when considering concrete applications, it is primarily considered as a potential fine aggregate replacement (Kim et al. 2020). Use as an SCM is also possible, although the need for milling and challenges relating to material handling detract from the viability of this approach. Other uses of BA include as a partial aggregate replacement in asphalt, and as a partial sand replacement in geotechnical applications such as backfilling and pavement base material (Moghaddam 2021).

BA has been trialled as a partial fine aggregate replacement in a concrete pavement in Queensland at the Stanwell power station (Lee 2021), and has also been used in the production of concrete blocks and panels for commercial construction (Moghaddam 2021). BA has also been trialled in asphalt and as a flexible pavement base material during the Stanwell power station trial. Due to the lightweight nature of the BA material, Moghaddam (2021) notes that lightweight concrete for precast applications is a potential target market.

The properties of BA vary depending on the properties of the source coal, and the operational characteristics of the producing power station (Moghaddam 2021). Lower quality bituminous coals produce denser particles which are reportedly more suitable for aggregate usage than the sand-like BA produced from the burning of higher quality coal (e.g. anthracite). Power demand and variability in power station loading can also result in BA with variable properties, which is not ideal for concrete aggregate applications (Moghaddam 2021). The mechanical properties of BA are discussed further in Section 2.7.2.

There are likely to be significant environmental and economic benefits for power station operators associated with finding practical large-scale uses for BA and other CCPs. Consequently, there is interest from power station operators who are exploring the feasibility of various use options for CCPs, particularly in QLD and NSW where the highest volumes of the materials are produced. While there are various infrastructure applications that BA may be suitable for (including pavement applications), the review in the report is focused on the use of BA as a fine aggregate replacement in TMR N-Class (non-structural) concrete.

2.7.2 Mechanical Properties of BA Aggregate

As noted previously, the mechanical properties of BA are influenced by the chemical makeup and quality of the coal being combusted, as well the operation of the power station (influencing the combustion process) linked to whether there is a consistent or varying energy demand (CCP Handbook 2017 cited by Moghaddam 2021; Gooi, Mousa & Kong 2020). Coal from different sources may be blended prior to use in some circumstances, which may result in inconsistent BA properties (Bennett 1997).

The majority of coal mined and used in power generation in NSW and QLD is bituminous coal, meaning that the BA produced from combustion is more likely to be suitable for use as aggregate compared to that produced from lower quality sub-bituminous or lignite (brown coal) which is relatively more common overseas and in Victoria (Hunt 2016). The variation in BA properties due to different coal sources and different countries of origin can be significant (e.g. up to 30% range observed for water absorption for different BA sources (Gooi, Mousa & Kong 2020)), which means that the outcomes of overseas studies on BA concrete properties may not directly translate to what could be expected for Australian BA.

Aggregate test data for BA produced by Bayswater Power Station (NSW) (Boral 2008) was provided for review by an industry contact who indicated that the properties for this product would likely be typical of BA produced by power stations in NSW/QLD. The properties determined by this testing have been summarised in Table 2.10, alongside the criteria specified by AS 2758.1:2014 and MRTS70 (11/2018). It is noted that the criteria specified by MRTS70 (11/2018) are generally requirements for special class concrete only and may not be directly applicable to the normal class non-structural concrete in scope of this project.

BA generally falls within the recommended grading limits specified by AS 2758.1. The density of BA is lower than the limit for normal class aggregate specified by AS 2758.1:2014, meaning that the aggregate could be classed as lightweight, although Moghaddam (2021) advises that certain BA products will have densities in the normal class range (> 2.1 t/m^3).

BA exhibits higher water absorption than typical fine aggregate, which can be expected to have some impact on the characteristics of the concrete mix at the fresh stage. Negative impacts on workability and pumpability are noted by Moghaddam (2021). Impacts on concrete permeability/transport parameters can also be expected, which are discussed further in Section 2.7.6. It is worth noting that while the water absorption recorded for the Bayswater ash is higher than the recommend aggregate limits, it is significantly lower than the water absorptions recorded by some overseas studies for BA, which can be over 30% in some cases (likely due to the presence of non-combusted carbon which is typical for lignite (brown coal) BA) (Gooi, Mousa & Kong 2020, Saridakis & Dentsoras 2008).

Typical BA contains small amounts of heavy metals, which pose an environmental issue when the material is stored in ash dams. While these metals are not likely to impact on the workability, strength or durability properties of produced concrete, there is a possibility that leaching will occur from produced concrete. There is also potential for certain heavy metals to act as a set retardant, although this behaviour does not appear to have been specifically highlighted for BA (Wiesława, Barbara & Sylwia 2015).

Gooi, Mousa & Kong (2020) notes that testing for heavy metal ion leaching is required to minimise potential of groundwater contamination. While there does appear to be some level of leaching risk, Gooi, Mousa & Kong (2020) states that studies conducted overseas found that tests on mortar incorporating 30% fine BA resulted in 'minor' leaching (test method from NSF/ANSI 61-2009) which was lower than what could be expected for raw BA. There may also be health and safety risks associated with exposure to raw BA and placement of fresh concrete which incorporates BA, and further investigation is recommended to quantify these associated risks.

It has been reported that BA is prone to being caught in hoppers due to the packing properties of the BA particles (Moghaddam 2021). This has reportedly not been overcome despite efforts to modify grading. This may lead to issues at concrete batching plants if standard equipment is used, and further investigation into this phenomenon is recommended to determine the viability of BA inclusion in concrete production processes.

Table 2.10: Standard limits for fine aggregate properties vs. reported BA properties

Item	Test method	Description	BA result	Acceptance criteria	Criteria reference
Bulk density	AS 1141.4	Compacted (t/m^3)	0.86	–	AS 2758.1
		Uncompacted (t/m^3)	0.80	–	AS 2758.1
Particle density	AS 1141.5 and/or AS 1141.6.1	Apparent (t/m^3)	1.82	–	AS 2758.1
		Dry (t/m^3)	1.7	–	AS 2758.1
		SSD (t/m^3)	1.8	≥ 2.1, < 3.2	MRTS70
		Water absorption (%)	3.7	≤ 2.5%	MRTS70
Particle size distribution (PSD) (grading)	AS 1141.11.1	Sieve size (mm)			AS 2758.1
		6.7	100	100	
		4.75	96	90–100	
		2.36	83	60–100	
		1.18	62	30–100	
		0.6	45	15–100	
		0.425	39	10–60	
		0.3	33	5–40	
		0.15	20	0–25	
		0.075	9	0–20	
Material finer than 75 µm	AS 1141.12	(%)	9	–	MRTS70
Material finer than 2 µm	AS 1141.3.1	(%)	0.2		
Moisture content	AS 1289.2.1.1	(%)		Nominated	AS 2758.1
Soundness	AS 1141.24	(%)	2.3	≤ 6%	MRTS70
Degradation factor	Q208B (1)	Clear		≥ 50	MRTS70
Weak particles	AS 1141.32	(%)	Nil	≤ 0.5%	MRTS70
Clay & fine silt	AS 1141.33	(%)	3	–	AS 2758.1
Organics	AS 1141.34	Positive/Negative	Pass (negative)	Negative	MRTS70
Sugar content	AS 1141.35	Positive/Negative	Negative	Negative	MRTS70
Methylene blue absorption value (MBV)	AS 1141.66	mg/g	No data	–	AS 2758.1
Deleterious fines index (DFI)	AS 1141.66	DFI	No data	≤ 150	MRTS70

Item	Test method	Description	BA result	Acceptance criteria	Criteria reference
Chloride content	AS 1012.20.1	(%)	0.008	Report if > 0.01%	AS 2758.1
Sulphate content	AS 1012.20.1	(%)	0.07 (as SO3)	Report if > 0.01%	AS 2758.1
ASR reactivity			No data		

1. As per Queensland Department of Transport and Main Roads (2021)
2. Source: Boral (2008).

2.7.3 Cost

BA is a waste product, meaning that the primary costs associated with its use are likely to be associated with transport, storage, processing and modifications to existing concrete production processes (if required). Increased cement content may be required to compensate for the inclusion of BA in some cases, though acceptable performance may be achieved if BA is used together with at least 20% FA inclusion as a GP cement replacement (noting that a reduction in performance could be expected when compared to a 100% NA mix with similar FA inclusion). It is recommended that the costs associated with BA processing to produce a 'concrete-ready' product are investigated with input sought from concrete producers and BA producers/suppliers. Costs associated with logistics and storage should also be investigated.

2.7.4 Availability

There is significant availability of waste BA in Australia, and particularly in QLD/NSW due to the high prevalence of coal-fired power generation in these states. Around 800,000 tonnes of bottom ash are produced in QLD each year, with over 1 million tonnes/year produced in NSW. BA stockpiles in Australia are estimated to be in the order of 400 million tonnes magnitude (Moghaddam 2021). Power station operators are generally supportive of efforts to develop usage options for the BA product, meaning that availability of the material is not likely to be a constraint provided that transport of the material to sites where concrete is being mixed is feasible.

The most common BA storage method in Australia is wet ponding, which is desirable since water quenching of ash from the furnace is a common practice (Moghaddam 2021). Before the saturated BA can be used in concrete applications, drying is required, which adds to the processing requirements of the material. This has the potential to reduce the attractiveness of use for the stockpiled material in Australia.

2.7.5 Performance of BA Concrete

Fresh properties

BA has generally been observed to reduce the workability of fresh concrete, with slump decreasing as fine BA inclusion is increased (constant w/c ratio) (Al-Fasih et al. 2019; Gooi, Mousa & Kong 2020; Rafieizonooz et al. 2016). This can be linked to the rough texture and irregular shape of the BA particles which lead to higher friction between particles, and the higher water absorption of BA compared to typical fine NA which can result in absorption of mix water by the fine BA (Al-Fasih et al. 2019; Gooi, Mousa & Kong 2020). Studies overseas have generally utilised superplasticiser (high-range water reducer) admixtures to control the workability of BA aggregate test mixes.

The Stanwell (QLD) pavement concrete trial utilised a water reducer (BASF 8190), and slump of 130 mm and 115 mm was observed for mixes incorporating 53% (sub-base) and 30% (base) inclusion of BA as a percentage of total fine aggregate respectively (Lee 2021). There is no mention of target slump in the data provided, but these slump values appear to be on the high side for the intended application (target slump range for concrete base mixes to be placed using fixed-form paving is 50-70 mm as per MRTS40 (11/2018).

This may indicate that the influence of BA on workability was overestimated when designing the mixes. A slump of 55 mm was achieved for a base mix incorporating 47% BA (incorporating Sika Plastiment 10), though lower strength was observed for this mix, which is discussed further in the following section.

It can be concluded that while BA inclusion in a mix has the potential to reduce workability, the impact can be controlled through careful mix design including the use of water reducers or superplasticisers where appropriate.

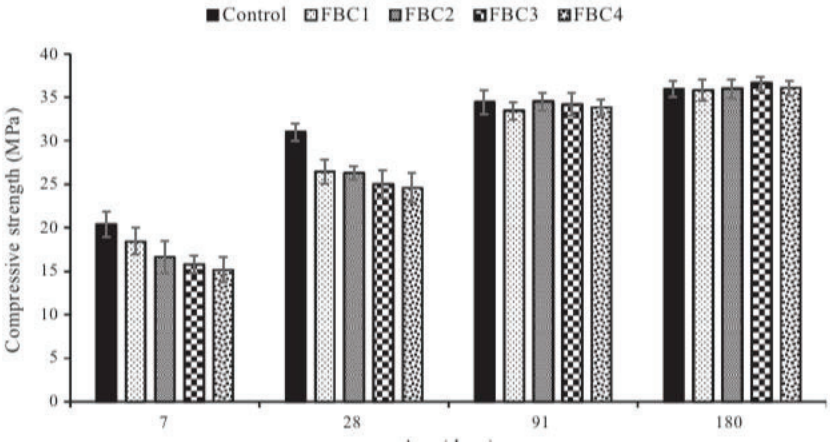
Strength properties

The literature reviewed generally indicates that the inclusion of BA as a partial replacement of natural fine aggregate results in a reduction of concrete strength properties (Gooi, Mousa & Kong 2020; Nadig et al. 2015) This reduction is particularly pronounced for compressive strength at early ages (pre-28 days), with significant strength gain observed at 28 days and later ages (56 days and later) which often results in compressive strengths which are only marginally lower than those observed for 100% NA mixes. This effect has been observed for both mixes which incorporate BA as a fine aggregate only, and mixes which incorporate BA as a fine aggregate and FA as an SCM. The strength increases in mixes incorporating BA and FA can be partially attributed to the pozzolanic behaviour of the FA (Rafieizonooz et al. 2016). It has been speculated that BA as fine aggregate also demonstrates pozzolanic behaviour, which is particularly pronounced between the ages of 28 and 91 days, leading to the observed compressive strength gain (Singh & Siddique 2015 cited by Rafieizonooz et al. 2016).

The Stanwell power station pavement concrete trial (Lee 2021) made slightly different observations to those found in the overseas literature for compressive strength gain. A mix incorporating 30% fine BA by mass as a replacement of natural sand exceeded the specified minimum compressive strength of 35 MPa at 28 days (achieved 46 MPa). This was significantly lower than what was measured for a control mix containing natural sand and manufactured sand, which reached 54 MPa at 28 days. A similar margin between the compressive strengths of the two mixes was observed at 56 days (53.5 vs 62 MPa), which equated to a strength gain on the 28-day value of 16% for the BA mix, and 14% for the control. Both mixes incorporated 30% fly ash as an SCM, so it could be speculated that most of the late age strength gain observed could be attributed to the pozzolanic behaviour of the FA. A 47% fine BA by mass mix was also tested which achieved lower compressive strength than the 0% and 30% BA mixes (36.8 MPa at 28 days) slightly higher than the 35 MPa specified minimum strength. This mix achieved 43 MPa at 56 days, an increase of 17% on the 28-day value.

Considering overseas research, the Malaysian study by Rafieizonooz et al. (2016) investigated mixes which incorporated 0% (100% NA), 25%, 50%, 75% and 100% BA replacement of natural fine aggregate by mass (constant w/c = 0.55, BA was sieved and graded prior to use). The mixes tested also incorporated 20% FA by mass replacement of OPC. 7-day and 28-day compressive strengths for the BA mixes were consistently lower than that measured for the 100% NA mix, with strength reduction increasing with BA content. At later ages (91 days and 180 days) it was observed that all BA mixes reached near-equal strength with the control mix (Figure 2.13), which could be attributed to the pozzolanic behaviour of the included FA and BA. Similar early age and strength development behaviour was observed by Kadam and Patil (2013), although in this study it was concluded that 30% inclusion of fine BA was optimal, with higher inclusions up to 100% generating compressive strengths over 30% lower than the control mix at ages up to 112 days.

Figure 2.13: Compressive strength of BA mixes at progressively increasing ages



1. Mixes FBC1 through FBC4 represent fine BA inclusion from 25%-100% at intervals of 25%.
2. Source: Rafieizonooz et al. (2016).

Other studies have indicated that the inclusion of finely ground BA as an SCM can also increase compressive strength relative to 100% NA, 100% OPC mixes, with Al-Fasih et al. (2019) showing that mixes incorporating up to 20% fine BA (aggregate) and 20% ground BA (SCM) recorded higher compressive strengths at all ages (7-days onwards).

Tensile (splitting) and flexural strength for mixes incorporating fine BA were tested by several overseas studies. It was generally observed that tensile strength followed a similar pattern to the compressive strength (lower at early ages and similar to the control mix at 28 days onward), while flexural strength was similar to the 100% NA control mixes (Rafieizonooz et al. 2016). Increases in tensile strength compared to control mixes were observed by Kadam and Patil (2013) for up to 30% fine BA inclusion, with decreases observed for higher inclusions. A similar pattern was observed by these authors for flexural strength.

Generally, it appears that the compressive, tensile and flexural strength of concrete is negatively impacted at early ages by the inclusion of fine BA, but comparable strength properties to 100% NA can be achieved at 28-days and later ages. Some studies have indicated that 30% replacement of fine NA with BA is the optimal replacement level, which is also supported by the early results of the Stanwell power station trial. Other studies have found that up to 100% inclusion of fine BA with 20% inclusion of FA as an SCM can produce satisfactory strength properties. It is recommended that an initial limit of 30% inclusion of fine BA is observed, with opportunity for further research at higher replacement levels using local BA products.

Deformation properties

Where measured, elastic modulus was generally observed to be similar to that of 100% NA control mixes for fine NA replacement levels up to 40%, after which a decline was registered (Cadersa & Auckburally 2014; Kadam & Patil 2013). For example, Cadersa and Auckburally (2014) observed a reduction from 27 GPa to 18 GPa for 40% and 80% replacement respectively at 28 days, with a similar pattern observed by Kadam & Patil (2013). Such a reduction may not be of critical concern for the non-structural applications in scope of this review and may not discount higher levels of fine BA inclusion.

Considering shrinkage, inclusion of fine BA has been shown to reduce 56-day shrinkage strain by some overseas studies (Rafieizonooz et al. 2016), although this effect was not observed for the Stanwell trial where 56-day shrinkage strain was practically equal for the 0% and 30% fine BA mixes (700 vs 710 microstrain). Rafieizonooz et al. (2016) found that shrinkage strain for mixes containing up to 100% fine BA (20% FA as SCM) was lower at all ages compared to the 100% NA control mix. Similar results were observed by Cadersa and Auckburally (2014) at replacement levels up to 80%. This reduction in shrinkage would be a positive for non-structural applications where finish is important such as footpaths.

2.7.6 Durability of BA Concrete

Permeation

Investigation of permeability for concretes incorporating fine BA has been undertaken using a variety of test methods. Kadam and Patil (2013) tested water permeability of concrete specimens at 28 days in accordance with DIN-1048-5:1991-06 and found that permeability was comparable to the 100% NA control mix at replacement levels up to 30% fine BA, after which a linear increase was observed for replacement levels up to 100% (94% increase for 100% fine BA mix). Remya Raju and Aboobacker (2014) tested fine BA concrete to ASTM C642-97 and noted water absorption of 7.21% for 30% fine BA concrete compared to 5.9% for the control mix at 28 days. Sorptivity testing to ASTM C1585-13 was undertaken by Saridakis and Dentsoras (2008) and it was observed that the sorptivity for BA mixes up to 100% inclusion was higher than the control mix but still within acceptable limits.

Chemical attack

Various authors have undertaken tests on chloride, sulphate and acid attack resistance for mixes incorporating fine BA. Saridakis and Dentsoras (2008) undertook rapid chloride penetration testing to ASTM C1202 (no year provided for test method) and determined that chloride permeability increased significantly with increasing fine BA content, from 1100 coulombs passed for the control mix, to 2410 coulombs passed for 100% fine BA (moderate permeability). Low permeability (< 2000 coulombs passed) was observed for mixes containing up to 60% fine BA. These authors also tested sulphate resistance to ASTM C1012-10 and found no difference compared to control concrete at 20% fine BA inclusion (4.8% reduction in strength), with reduction in strength of 6.6% at 40% fine BA inclusion, and 9% at 100% BA inclusion.

Other authors have indicated that the inclusion of fine BA can improve chloride and sulphate resistance, which has been attributed to reduction in pore solution alkalinity generated by the BA (Gooi, Mousa & Kong 2020). While there is some contention in the literature, it appears that up to 40% inclusion of fine BA would not have a significant impact on chemical attack resistance of concrete which is used in applications where chemical attack is of concern. For certain non-structural applications where chemical attack is of lower concern (e.g. benign environmental conditions, non-reinforced concrete), higher replacement levels may be appropriate. Further investigation using NSW/QLD BA may be appropriate to determine whether the improvements in chemical attack resistance observed by some overseas studies can be reproduced.

Physical attack

There were limited details available in the reviewed literature regarding the abrasion resistance of concrete incorporating BA as a fine aggregate replacement. One study noted that the abrasion resistance of 100% fine BA concrete was reduced compared to a control mix, but no further details were provided (Gooi, Mousa & Kong 2020). Abrasion resistance of BA concrete is noted as a research gap by Gooi, Mousa and Kong (2020). If abrasion resistance is thought to be of concern for any of the TMR non-structural applications in scope of this review (e.g. kerb and gutter, pavements), abrasion testing of fine BA mixes in accordance with the appropriate Australian test method is recommended.

Alkali-Silica Reaction (ASR)

Discussion of ASR potential due to the inclusion of fine BA as a fine aggregate replacement in concrete was not found in the literature reviewed. The only mention of ASR in relation to BA was to the potential ASR-mitigating ability of fine-ultrafine BA (less than 0.075 mm diameter particles) when used as an SCM (Abbas et al. 2020, Gooi, Mousa & Kong 2020). When sieved or ground to this size, BA has been shown to exhibit pozzolanic behaviour similar to FA, but it is possible that at larger particle sizes (up to 4.75 mm) the presence of amorphous silica in BA would lead to ASR-reactivity.

The inclusion of FA as an SCM at up to 20% replacement of OPC has been shown by several studies including the Stanwell power station trial to be beneficial for the strength properties of fine BA concrete. It could then be postulated that the inclusion of FA at this level would ameliorate any ASR potential of the BA which is incorporated as a fine aggregate replacement. If the ASR-potential of BA is thought to be of concern, mortar bar testing in accordance with AS 1141.60.1:2014 or concrete prism testing in accordance with AS 1141.60.2:2014 could be undertaken to investigate ASR potential of various BA inclusion levels and mixes which incorporate FA as an SCM.

3 Industry Engagement

3.1 Background and Approach

The Year 1 scope of industry engagement was to engage with RCA, RCG and RA suppliers, waste recyclers, concrete producers and construction companies with the aim of receiving information on the types of recycled aggregate available, sources of recycled aggregate, advantages and disadvantages of recycled aggregate use, and aggregate properties/performance data. A survey was circulated to industry contacts to facilitate the collection of this information, provided in Appendix A.

Through efforts spent on searching online and assistance provided by TMR, a total of 23 survey recipients were identified and consequently contacted. After two to three rounds of communication, nine of the recipients responded with either completed survey forms and relevant data or indicated unwillingness to provide any information. Moreover, the limited data and information received are not consistently qualified. The unqualified data is mainly due to the lack of technical assessment, and data and the information provided is based on respondent experience.

Almost no unpublished performance data could be sourced through the Y1 industry engagement process. This is due to the reluctance of recycled aggregate suppliers and producers to share confidential data, and some contacts specifically requested that no data or information relating to production processes be published. Due to this fact, the feasibility review for each material (Section 4) has primarily focused on the information and data sourced through the review of the publicly available literature.

In Year 2 of the S51 project, TMR facilitated further industry engagement with a focus on Queensland contacts. This aimed to capture further information to supplement the findings of the Y1 engagement and source additional performance data where available.

3.2 Recycled Concrete Aggregate

A survey response from a Queensland concrete producer indicated that it was believed that the use of RCA would result in poorly performing and more costly concrete. This was reportedly due to the water absorption of RCA above 6% which leads to a higher water demand and need for more cement to maintain target strengths. Higher concrete drying shrinkage was also identified as a concern, with examples of up to a 40% increase detected in past applications of RCA. The producer indicated that the use of RCA resulted in a set time that was 1.5 hours shorter compared to 100% NA concrete.

A NSW university also provided a survey response which discussed ongoing work on the testing of concrete produced using RCA. Research projects completed by the university had generally focused on the use of 10 mm coarse RCA, with some more limited works completed for fine RCA. It was indicated that the high water absorption of RCA needed to be controlled to maintain concrete workability.

Overall, the limited responses from industry and academia mirror some of the concerns highlighted by the literature review, including the reduced strength properties and increased water demand of RCA concrete and reductions in workability. Based on the review of literature, there appears to be potential for some of these issues to be mitigated through use of SCMs including fly ash and blast furnace slag, which is discussed further in Section 4.1. Despite this, there may still be a drop in performance compared to typical N-Class TMR concrete, which generally incorporates SCMs as part of standard practice.

3.3 Recycled Crushed Glass

3.3.1 Brisbane Site Visit

A site visit was conducted with a Brisbane-based RCG supplier in March 2021. This allowed for inspection of the RCG processing equipment and the range of available RCG products as well as discussion with the plant operators.

The current fine aggregate products produced by the supplier are split into five gradings, ranging from coarse grading (1.7 mm to 3.35 mm) to super fine grading (0.075 mm to 0.750 mm). The supplier also produces a ‘concrete-grade’ product, which has a wider grading band. Some further consultation and testing may be required to determine whether this product is suitable for concrete applications without any further processing.

The supplier advised that loss on ignition (LOI) provides a reasonable indication of product quality. The LOI percentage is determined from the proportion of the material that is lost when a sample of the product is heated and is generally influenced by the product moisture content and presence of organic materials. Effectively, the LOI test allows for the presence of volatile contaminants such as paper and plastic to be detected. A good quality product is identified as having an LOI less than 2%, and the products tested by the supplier all fall below this limit.

The presence of sugar in the RCG products is a possibility, but the supplier stated that the products often test negative to the presence of sugar. It was speculated that the rubbing and grinding motions which occur during conveyor belt and trommel processing of the RCG material may result in reduction of sugar content.

A cost of $30/tonne was quoted for fine aggregate which is currently used in pavement applications. Prices generally increase with increasing fineness and quality (lack of contamination), up to around $500/tonne for superfine products (not likely to be appropriate or economic for concrete applications).

The supplier stated that waste glass supply of 50-60,000 tonne/year was available for use as RCG. The supplier was in the process of constructing a new plant to increase output six-fold at the time of the site visit, which indicates that there is likely to be reasonable supply of RCG for concrete applications within the Greater Brisbane area in the future, although concrete applications would be competing with the other applications for the product, particularly the higher value applications.

Overall, the produced RCG appeared clean and well graded within each product category. It can be concluded that there is potential for the inclusion of the RCG material in concrete. Further work may be required to reliably maintain a low level of sugar contamination to acceptable levels. Further work which investigates ‘acceptable’ levels of sugar inclusion in concrete may also be warranted if compliance with AS 2758.1:2014 proves to be an issue.

3.3.2 NACoE P76

Engagement with Queensland RCG suppliers also occurred as part of a NACoE pavements project (P76) (Latter 2021a). This resulted in the supply of RCG conformance data for properties such as sugar content, presence of other contaminants and RCG grading. The properties of the produced RCG were found to depend on the processing techniques used, with materials which underwent less stringent processing and quality control more likely to contain contaminants. Details of the actual processes used by certain RCG producers and suppliers are unclear and may not be publishable due to confidentiality requirements.

3.4 Reclaimed Aggregate

Mapei supplies the RA-producing admixture Re-Con Zero Evo in Australia and was contacted to discuss the use of the admixture including performance of concrete produced using the RA and current scope of use in

Australia. It was advised that several Queensland concrete producers either operate RA separator equipment or have performed trials, but this information is generally kept confidential, and further details on the properties and feasibility of the aggregate product were not able to be sourced from industry.

3.5 Ferronickel Slag

A meeting was held in November 2020 with a representative from CMS Consulting who have been working together with SLN (the company that produces the FNS in New Caledonia as a waste by-product from the ferronickel smelting process) to develop and market the FNS product for various uses including concrete fine aggregate replacement and as an SCM. During this meeting, a presentation was made to TMR and ARRB project staff detailing the current body of research into FNS as a fine aggregate for concrete along with current uses and product availability. The research detailed in this presentation has been discussed in Section 2.6 of this report.

3.6 Power Station Bottom Ash

A meeting was held in November 2020 with a representative from Bilmar Consulting who have been working together with power stations to develop use options for CCPs including BA. During this meeting, current field trials in Australia were discussed, with the most relevant to the S51 review being the pavement concrete trial at the Stanwell power station in Queensland. In this trial, BA has been used as a partial fine aggregate replacement (30% and 47% inclusion) in rigid concrete pavements designed in accordance with MRTS40 (11/2018). A rigid concrete control pavement incorporating natural and manufactured fine aggregate was also installed adjacent to the BA pavements. In this trial BA was also used as a base and sub-base material for flexible pavements, but this is outside the scope of this report. A summary report and concrete test data were received for the Stanwell trial, which have been discussed where relevant in Section 2.7.

4 Feasibility Review

4.1 Recycled Concrete Aggregate

4.1.1 Discussion

There is a significant body of research available which has focused on investigating the properties of RCA concrete, with both coarse and fine RCA considered for replacement of NA at mass to mass replacement levels from 20% up to 100%. Some research has focused on adopting RCA for use in structural concrete, while others have considered RCA most appropriate for non-structural use. Research on structural applications has formed an input into the review conducted for this project, but the primary focus has been on suitability for non-structural applications in accordance with the expected scope of use outlined in Section 1.3.

The majority of research findings indicate that, at replacement ratios below 30% by total aggregate mass, there are no significant adverse influences of the RCA on the strength or durability properties of concrete, and the resultant properties are within the acceptable range. At high replacement levels, (50% by mass up to 100%), the influence of RCA on the properties of the produced concrete varies from minor to significant (from below 10% to a 20% reduction in the properties tested compared to control concrete), depending on the quality of the RCA. It can be concluded that, for non-structural concrete in line with the S51 project scope, a 30% replacement of NA by coarse RCA that complies with all relevant aggregate specification requirements would not result in a significant reduction in mechanical properties and durability, and that design lives of up to 50 years would be achievable. There is less certainty relating to the use of fine RCA, though some research has indicated that use of up to 30% fine RCA by mass may result in acceptable concrete properties. Further investigation is recommended if fine RCA is considered desirable for inclusion in N-Class (non-structural) TMR concrete.

Regarding strength properties, a 10–20% reduction in compressive strength can be expected for concrete produced using 100% of either coarse or fine RCA. Small reductions in flexural and tensile strength are also possible at high replacement levels, though RCA appears to have less of an impact on these properties, although the reductions are consistent with the relationship of these properties to compressive strength. The reduction in strength properties observed for RCA concrete is thought to be due to development of a weaker interfacial transition zone linked to the presence of old mortar in the RCA (Kisku et al. 2017).

The quality of RCA largely depends on the content of attached mortar, and the original concrete grade. The presence of contaminant materials such as bricks, metals and plastics will also negatively impact concrete performance, and it is recommended that quality control procedures are in place that can maintain the contaminants below 1% of the RCA by volume. The presence of old mortar in RCA leads to higher aggregate water absorption, generally up to 6-8% for coarse RCA. Fine RCA may present higher water absorption (as high as 13%) due to the higher content of fine mortar in this fraction, and for this reason, fine RCA should be used with caution. It is recommended that a limit of 6% water absorption is adopted for coarse RCA in line with the recommendations of HB-122-2002. Appropriate limits for fine RCA are not clear, and further investigation may be required in this space if fine RCA is to be pursued for inclusion in concrete.

The higher water absorption of RCA generally contributes to the development of poorer durability (especially for transport properties such as water absorption, chloride permeability, carbonation) for the produced concrete at high replacement levels of coarse and fine RCA (50% by mass and over). The research has shown however, that at 30% replacement of NA using coarse RCA the impact on transport properties will not be significant.

The workability of RCA concrete is also impacted by the higher water absorption of the fine or coarse RCA, since the produced concrete will typically have a higher water demand which may lead to slump loss. This is less of a concern for lower replacement levels (up to 30% by mass), and the use of superplasticiser admixtures is likely to allow for the control of reduced workability for the concrete applications in scope of this report.

Concrete containing RCA as a replacement for NA generally contains a lower volume of natural aggregate due to the presence of mortar, which can lead to a reduced modulus of elasticity and higher deflection of the components, for a given compressive strength. If RCA has been sourced from a higher strength original concrete (i.e. 50 MPa and higher), the strength of the attached mortar will enhance resistance to deformation, but the heterogenous nature of a typical RCA means that this effect cannot be relied upon.

Some studies have shown that the ASR reactivity of RCAs can be higher than the reactivity of the original NA. This may be linked to the crushing process that RCA undergoes, which can expose fresh reactive faces of the original aggregate. The research has shown that in general, the current MRTS70 (11/2018) replacement levels of fly ash (25%) or silica fume (5% in combination with 25% fly ash) will be sufficient to mitigate the development of ASR in RCA concrete using replacement levels up to 30% of coarse RCA. The use of SCMs in RCA concrete will also improve the strength and durability properties of the produced concrete.

RCA is a heterogenous material and may contain original aggregates from various alkali-reactive and non-reactive sources. This means that the reactivity of RCA may vary from batch to batch, depending on the source of the original aggregate. Assuming that RCA is sourced from Queensland, further confidence in the mitigating ability of the standard SCM replacement levels could be gained by undertaking mortar bar testing and/or concrete prism testing on mixes produced using RCA from old concrete containing known reactive aggregate.

The inclusion of RCA in new concrete has the potential to deliver environmental benefits through diversion of material from landfill and reduction in the need to produce new aggregate materials. This is the primary benefit of RCA use from a concrete perspective, since review of the literature has found that the use of coarse and fine RCA as a like for like (100%) replacement for natural aggregates is likely to result in the production of concrete mixes which exhibit reduced strength properties, lower durability, and decreased workability. Despite this, it has generally been found that at replacement levels up to 30% of coarse RCA, concrete produced using standard TMR N-Class mix designs (including SCMs at exposure class B2 levels or higher) will develop acceptable strength, durability and workability properties. To facilitate this, controls on RCA should include a limit of 1% inclusion of contaminants (metal, plastics, brick) by volume, and an upper limit of 6% water absorption for coarse RCA.

4.1.2 Further Work

Further consultation with concrete producers and Queensland RCA suppliers is recommended to gain a better understanding of current barriers to implementation and methods to overcome these. This should focus on concerns including cost of RCA, current processing methods for RCA, and issues associated with concrete production such as workability and slump loss. The availability of RCA which is suitable for new concrete applications (i.e. meets contaminant and water absorption limits) should be investigated with suppliers.

A staged field trial of RCA concrete using TMR mix designs is recommended prior to implementation. This should allow for incrementing replacement percentages (10%, 20%, 30% coarse RCA by mass) to be tested in order to work out any issues relating to workability or mix design. If acceptable results are obtained for 30% coarse RCA concrete, consideration may be given to extending the trial to incorporate 50% coarse RCA.

As discussed in the previous Section, it is expected that the current MRTS70 (11/2018) SCM inclusion levels for fly ash (25%), blast furnace slag (60%),and silica fume (5% in combination with 25% fly ash) are expected to be sufficient to mitigate ASR in RCA concrete with up to 30% coarse or fine RCA inclusion. Further confidence in this approach could be gained by undertaking mortar bar testing (AS 1140.60.1:2014) and/or concrete prism testing (AS 1140.60.2:2014) on mixes produced using crushed RCA containing known reactive aggregates.

4.1.3 Opportunities

The findings of the literature review for the use of RCA in concrete has found that there is potential for RCA to be used as a coarse aggregate replacement in non-structural concrete as per the S51 scope at replacement levels up to 30% by total coarse aggregate mass. There may be opportunity for collaboration between TMR, RCA suppliers and concrete producers to trial the product and work through any practical concerns that may arise. A staged trial of RCA concrete is recommended prior to implementation, using replacement levels ranging from 10% up to 30% of coarse RCA by mass. Concrete production should be consistent with the requirements for TMR N-Class concrete.

4.2 Recycled Crushed Glass (RCG)

4.2.1 Discussion

Most recycled glass aggregate research conducted to date has focused on the use of recycled glass as a replacement for fine aggregate in concrete (Obe et al. 2018), with recycled glass generally considered undesirable as a coarse aggregate due to its flat particle shapes and tendency to crumble under applied force (Polley, Cramer & de la Cruz 1998). Despite this, some researchers have focused on the inclusion of recycled glass as partial replacement of coarse aggregate, which has found that inclusion of coarse recycled glass produces acceptable concrete properties up to a level of 20% replacement of coarse aggregate (Serpa, de Brito & Pontes 2015, Shayan & Xu 2004). Considering the limited body of research available and the risks associated with the use of coarse glass, it is not recommended to be pursued by TMR at this time.

Use of crushed glass sand as a partial fine aggregate replacement in concrete is more widely accepted compared to use as a coarse aggregate replacement. There is currently allowance for up to 15% replacement of fine aggregate using crushed glass sand by TfNSW, and up to 30% by mass replacement allowed by VicRoads. The majority of research available indicates that concrete strength and performance for fine crushed glass concrete is either increased or unchanged compared to reference NA concrete for replacement levels up to 30% by mass (Obe et al. 2018).

There is a lack of long-term performance data for RCG concrete, with the majority of literature focusing on laboratory studies and field trials of (so far) limited duration (~1 year at time of writing). While laboratory studies can quantify the expected performance of produced concrete reasonably well, field data is also valuable for the quantification of concrete performance, particularly for chemical processes such as ASR which proliferate in concrete over a long period of time (years to decades). Recent trials in Australia (Cairns, Melbourne, Pacific Highway) will in time produce insights into the long-term performance of RCG concrete for non-structural applications, but in the meantime, there is inevitably some risk associated with the uptake of RCG concrete regarding long-term performance and durability.

Contamination of RCG aggregate with substances such as sugar or heavy metals, or deleterious materials such as metal, paper and ceramics is of concern with regard to concrete production. Trials documented in the literature have generally either found no detectable levels of sugar or have pre-washed RCG prior to concrete production. No record was found in the literature of negative impacts on concrete properties directly attributed to the presence of sugar (or other deleterious materials). The TMR standard MRTS36 (11/20) for recycled glass use in pavements specifies limits on concentrations of chemicals and heavy metals, and it is expected that these limits would be appropriate for concrete applications from a health and safety/environmental perspective.

ASR reactivity is the primary concern for the durability of concrete produced using recycled glass aggregate, with some concerns also existing linked to increased carbonation depth of recycled glass sand concrete.

Other concrete durability aspects including chloride penetration, permeability and abrasion are generally either not impacted or improved by the substitution of recycled glass as fine aggregate.

The ASR potential of recycled crushed glass used as fine aggregate in concrete has been widely investigated, with varying results obtained by different researchers as to the reactivity of the glass and the degree of expansion observed in concrete mixes produced using recycled glass aggregate. It has been shown that soda-lime glass is the least reactive commercially available type of glass, which is encouraging given that soda-lime container glass is the most prevalent recycled glass stream in Australia and worldwide. Nevertheless, soda lime glass has been shown to have varying levels of alkali-reactive potential by multiple authors.

The use of SCMs to mitigate alkali-reactivity in concrete is a widespread practice which is adopted for all concrete mixes used in TMR construction. Several authors have investigated the effectiveness of SCMs for the mitigation of ASR in recycled glass concrete. Replacement levels of ≥ 25% fly ash, ≥ 10% metakaolin, ≥ 60% GGBFS and ≥ 10% silica fume have been identified as effective at preventing deleterious ASR expansion in mixes with up to 100% replacement of fine aggregate with crushed glass. Based on these recommended levels, the current SCM practice enacted by TMR is likely to be sufficient to prevent the development of deleterious ASR in concrete produced using fine RCG.

Considering the findings of the literature reviewed, an upper limit of 30% replacement by mass of fine aggregate with recycled glass sand is recommended for adoption for use in TMR N-Class (non-structural) concrete which has a target design life of up to 50 years. This is in line with the current practice of VicRoads. Current SCM replacement levels for fly ash, GGBFS and amorphous silica should be maintained for RCG concrete. Fine RCG should comply with the aggregate requirements outlined by MRTS70 (11/2018) and discussed in Section 2.4.2.

Considering contaminants, regular testing of RCG supplies for sugar content is recommended, with negative results (as per AS 2758.1-2014) required for acceptance of glass in concrete. If the presence of sugar is found to be a limiting factor for implementation of RCG, further research and testing may be warranted to determine typical levels of sugar contamination and impacts on concrete production with the aim of establishing an appropriate alternative limit.

Contaminants such as metal, ceramics, paper etc. should be maintained below 2% by weight of RCG as recommended by VicRoads. Testing of loss on ignition (LOI) is recommended to control organics and other combustible content, with a limit of 2% LOI recommended. It is recommended that levels of potentially harmful chemicals and heavy metals (such as lead) are maintained below the limits specified by MRTS36 (11/20).

The replacement of natural coarse aggregate with coarse RCG is not recommended at this point in time considering the limited volume of research available and the potential drawbacks related to strength and durability which have been highlighted in the literature.

4.2.2 Further Work

Trials are recommended prior to full-scale adoption of recycled glass in concrete in order to identify any potential issues associated with concrete production, placement and long-term mechanical and durability performance. These should use RCG products which are available in Queensland. Close monitoring of outputs from current trials in Cairns and Melbourne, and the performance of the Pacific Highway upgrade trial are also recommended. A staged approach to trialling is recommended, whereby various replacement percentages of fine RCG are tested. Recommended replacement percentages for initial trials include 10%, 20%, and 30%.

The cost of RCG compared to NA may be prohibitive for large scale adoption at this point in time, though industry consultation could provide further clarity regarding the expected cost of adopting RCG for concrete production in Queensland. Collection, haulage and processing costs need to be considered when developing price comparisons with NA.

The availability of good quality RCG (complying with limits on contamination) may also prove to be an issue for large scale adoption in concrete, since there is some competition for use of the RCG in pavements and fill applications, as well as to produce new glass products. Further industry consultation is recommended to determine what volumes of RCG may be devoted to concrete production, and whether current processing methods used by Queensland suppliers are appropriate to meet requirements for limits on contaminants.

As noted above, it is likely that the recommended SCM replacement levels would sufficiently prevent ASR in concrete produced using up to 30% recycled glass sand. However, further confidence in the effectiveness of SCMs for the mitigation of ASR in concrete produced using Queensland recycled glass could be gained by undertaking mortar bar testing in accordance with AS 1140.60.1:2014. If the AMBT shows that the RCG concrete is reactive, less conservative and more accurate results could potentially be gained through concrete prism testing in accordance with AS 1140.60.2:2014. This could also support the adoption of higher fine glass replacement levels if desired.

Potential mix designs include:

- 30% glass sand as percentage of fine aggregate, 25% fly ash, balance of GP cement and natural coarse/fine aggregate.

If higher replacement percentages are thought to be desirable:

- 50% glass sand as percentage of fine aggregate, 25% fly ash, balance of GP cement and natural coarse/fine aggregate.

Fly ash in the above mixes may be substituted for 60-70% GGBFS as per MRTS70 (11/2018).

4.2.3 Opportunities

The findings of the literature review for the use of RCG in concrete indicate that there is potential for RCG to be used as fine aggregate replacement in non-structural concrete as per the S51 scope at replacement levels up to 30%. There is likely to be opportunity for collaboration between TMR, recycled glass suppliers and concrete producers to trial the product and work through any practical concerns that may arise. A future trial of 30% fine RCG concrete in a non-structural application is recommended. Intermediate trials of 10% and 20% replacement of fine aggregate using fine RCG are also recommended to allow for comparison between approaches.

4.3 Reclaimed Aggregate (RA)

4.3.1 Discussion

There appears to have been limited work to date which has focused on the properties of RA or concrete produced using RA. There are two main methods for producing RA, which are water washing and use of a chemical admixture product which absorbs the free water in the mix and produces a material similar to the original aggregate with a portion of hydrated cement paste. The review presented in this report has focused on RA produced using the latter method, using materials such as Re-Con Zero Evo or Cycrete.

Research to date has focused on the use of coarse RA as a partial replacement for natural coarse aggregate, with acceptable strength and deformation properties (as per AS 3600:2018) found for concrete produced using up to 90% coarse RA produced from 40 MPa concrete using the Cycrete system. Where 25 MPa original concrete was used to produce the RA, concrete could be produced using up to 70% coarse RA. There has been less of a focus on the durability properties of concrete produced using RA, with the available research indicating that the permeability of RA concrete is high at early age but reduces to acceptable levels by 90 days. The impact of this on properties such as chloride penetration or carbonation does not appear to have been directly tested. Since RA does contain a portion of cement mortar, some penalty to durability could be expected, but it is likely that this could be managed with the use of SCMs in line with TMR practice. Testing to verify this hypothesis is recommended prior to implementation.

Regarding cost, there does not appear to have been a comprehensive economic analysis conducted for Australian concrete producers, although the use of aggregate reclaimers has been shown to be potentially economically viable for concrete producers in Hong Kong and Turkey. To make the use of RA an economically attractive proposition, the cost saving of diverting leftover concrete from landfill must be sufficient to outweigh the cost of the aggregate reclaimer equipment and admixtures, and concrete plants must have sufficient space to store the produced RA material prior to use in new concrete. Further industry engagement is required to establish whether RA could be viable for Queensland and Australian concrete producers.

The availability of RA depends on the volumes of over-ordered or leftover concrete produced by the concrete plant. This is not likely to be a steady supply volume since it is dependent on the variability of concrete orders and the practices of the concrete producer. To be viable, RA must be considered suitable for inclusion in everyday concrete production or be stockpiled for use when required.

4.3.2 Further Work

Engagement with the producers of RA admixtures and concrete producers is recommended to further investigate and clarify the economic viability and practicality of the RA approach. Further testing and trialling of RA and produced RA concrete is recommended to verify the strength and durability properties of concrete produced using RAs created through use of the available chemical admixtures, i.e. Re-Con Zero Evo, Cycrete or other suitable options.

If pursuing further concrete testing and trialling, it is recommended that coarse RA is tested at varying replacement percentages, from the 30% by mass replacement recommended by Ferrari and Brocchi (2012) up to the 70% and 90% replacement by mass recommended by Gunasekara et al. (2020).

4.3.3 Opportunities

There appears to be opportunity to use coarse RA as up to a 90% replacement of natural coarse aggregate by mass if a 40 MPa concrete is used to produce the RA. A lower replacement percentage may be required for lower grades of original concrete, e.g. 70% by mass for RA produced from 25 MPa original concrete. Further trialling is recommended to determine the most appropriate replacement percentages for use in field applications.

The use of RA in new concrete may provide an opportunity for concrete producers to reduce costs associated with dumping leftover concrete and improve environmental outcomes, though to be economically viable the cost savings must outweigh the cost of disposing of the leftover concrete through other means.

4.4 Ferronickel Slag (FNS)

4.4.1 Discussion

The literature review conducted for this project has focused solely on the body of research conducted on the FNS by-product produced by SLN in New Caledonia. This approach is justified by the fact that the SLN product would likely be the sole source of FNS for TMR concrete if the material was to be adopted for use. FNS in concrete may be used either as a fine aggregate replacement or in a ground form as an SCM (GFNS), with this report focusing on fine aggregate usage only.

FNS exhibits mainly favourable physical and chemical properties for inclusion in concrete as a fine aggregate with some exceptions including:

- low proportion of fines in the grading (0.6 mm – 0.3 mm fractions)
- relatively higher permeability in FNS concrete compared to 100% NA mixes
- ASR risk linked to presence of silica.

The low proportion of fines can be overcome by blending the FNS with natural sand and manufactured sand at appropriate ratios. Increased permeability can be controlled through the inclusion of 25% FA as an SCM where permeability is a concern (particularly in aggressive environments), but it may not be possible to achieve equivalent permeability to 100% NA concrete which includes 25% FA (as per common TMR practice). Permeability may be of lower concern in low-risk applications such as footpaths/bike paths. Finally, ASR may also be controlled by the inclusion of at least 25% FA as an SCM binary blend. Other SCM options such as GGBFS may also be effective for ASR mitigation, but this approach has not been tested and therefore cannot be recommended without further testing.

The primary costs of FNS are likely to be associated with transport of the material from New Caledonia to Queensland, and with the need for concrete producers to stockpile and include the material in a ternary blend of aggregates (FNS, manufactured sand and natural sand). There is significant availability of the FNS material in New Caledonia (25 MT stockpiled, 1.6 MT/year production), and it appears that SLN is keen to work with potential adopters of the material.

Overall, it appears that up to 35% by mass inclusion of FNS as fine aggregate would be viable for TMR non-structural concrete as per the scope detailed in Section 1.3. A mix which includes up to 50% FNS inclusion may also be viable, though such a mix may not meet optimal grading for fines and may have issues with mix bleed or finishing (CQT Services 2018). It is likely that concrete producers would need to undertake some experimentation with the product to achieve an optimal blend which accounts for the properties of FNS and local aggregate materials. To mitigate ASR, FNS mixes should include at least 25% FA as an SCM.

4.4.2 Further Work

Prior to pursuing any further research work, discussion with concrete producers is recommended to investigate the viability and appetite for using an additional material which is likely to require blending with natural sand and manufactured sand. Sourcing and review of detailed performance data from field trials which have been conducted to date in New Caledonia are also recommended to better understand field performance. More detail on material costs from the procurement and usage perspectives is also required to assess the viability of the material. If concrete producers are receptive to the concept, it is recommended to work with producers to develop an optimal mix design which meets production and placement requirements.

4.4.3 Opportunities

The findings of the literature review indicate that there is potential for FNS to be used as fine aggregate replacement in non-structural concrete as per the S51 scope in combination with manufactured sand and natural sand at a proportion of up to 35%. There is likely to be opportunity for collaboration between TMR, SLN and concrete producers to trial the product and work through any practical concerns that may arise. A field trial of the mix designs presented by CQT Services (2018) (or variations thereof) is recommended prior to any large-scale implementation. Proposed mix designs include (all mixes to include at least 25% FA):

- 50% FNS, 50% natural sand
- 12% manufactured sand, 35% FNS, 53% natural sand (10 mm max aggregate size)
- 12% manufactured sand, 44% FNS, 44% natural sand (10 mm max aggregate size).

The proportions of manufactured sand and natural sand recommended here are based on products available in Sydney and may need to be adjusted based on the gradings of manufactured/natural sands which are available in Queensland. Investigation of mixes with and without the inclusion of manufactured sands is recommended.

4.5 Power Station Bottom Ash (BA)

4.5.1 Discussion

Coal-fired power station BA is relatively less well developed as a recycled aggregate material for inclusion in concrete compared to materials such as RCG and RCA. There is a reasonable body of international research which has focused on investigating the use of BA as a fine aggregate in concrete or as an SCM, but there has been limited Australian research conducted to date. The exception to this is the Stanwell concrete pavement trial, which has been reviewed and shows some promising results for partial inclusion of fine BA in concrete as a fine aggregate replacement.

The mechanical properties of BA aggregate, particularly water absorption and density are largely dependent on the properties of the source coal, meaning that it is difficult to directly compare the outcomes of overseas studies to what could be expected from the use of NSW/QLD BA. Despite this, satisfactory results have been observed from overseas studies for strength and durability properties, often in cases where the fine BA displayed water absorption above 10%. This is encouraging for the use of local BA as fine aggregate replacement since the properties recorded for the Bayswater BA (Section 2.7.2) are comparable to the AS 2758.1:2014 limits, although the water absorption (3.7%) is higher than the recommended limit of 2.5%

There is some variance between studies, but the outcomes of the Stanwell trial and certain overseas studies indicate that up to 30% inclusion of fine BA in concrete as a replacement of natural sand will not have a significant impact on strength and durability properties. Fine BA has been shown to have an impact on early age concrete strength properties in some studies, with comparable strengths to the control mix achieved by 28 days onwards. This could be an issue for applications where early age strength of concrete is important. Higher levels of BA inclusion may be feasible depending on the intended application, particularly for non-structural applications where strength and durability properties are less critical.

Inclusion of at least 20% FA as an SCM in concrete that incorporates fine BA aggregate has been shown to assist in achieving satisfactory strength and durability properties. FA appears to be a good complement to fine BA, being capable of balancing out some of the potential negative impacts of BA as an aggregate replacement. The inclusion of FA in BA concrete should also ameliorate ASR potential, though this does not appear to have been directly verified.

Availability of BA is not expected to be a constraint, as the low-risk applications that are attractive for some other recycled aggregate materials such as geotechnical applications are less attractive for BA since the coarse fractions are prone to crumbling, and the heavy metal content of BA leads to a risk of leaching (although there is some potential for use as fill material for low/mid-level loading applications). Costs associated with BA are likely to be primarily associated with transport, storage and processing, and further investigation into the costs and general viability of including BA in standard concrete production processes is recommended.

Overall, the inclusion of BA as a partial fine aggregate replacement in the non-structural concrete applications in scope of this report appears to be a technically viable proposition. An initial limit of 30% replacement in TMR normal class concrete is recommended, with higher limits potentially viable for certain applications, though further work would be required to investigate this hypothesis.

4.5.2 Further Work

Trials of BA as a fine aggregate replacement in TMR non-structural concrete conducted in collaboration with the BA producers and concrete producers could assist with identifying any issues associated with concrete production, placement or hardened properties. Notwithstanding this, ongoing review of performance outcomes from the Stanwell concrete pavement trial is recommended. More detail on material costs from the procurement and usage perspectives is also required to assess the viability of the material. Testing of BA products from QLD power stations is also recommended to determine how the material compares to the Bayswater BA and the AS 2758.1:2014 limits and determine how much material variability exists between different sources.

4.5.3 Opportunities

The findings of the BA literature review indicate that there is opportunity to include up to 30% fine BA as a by mass replacement of natural fine aggregate in TMR non-structural concrete (normal class). Trials in conjunction with concrete producers and BA suppliers are recommended to allow for any issues to be identified and tackled accordingly.

5 Conclusions and Recommendations

This report has discussed the findings of a national and international literature review and an industry survey into the utilisation of recycled aggregates in concrete. The aim of this project was to investigate the properties of recycled aggregates and the performance of concrete produced using recycled aggregates with a view to determine whether certain recycled aggregates could be used in N-Class (non-structural) concrete by TMR. The scope of the Year 1 project was to investigate recycled concrete aggregate (RCA), recycled crushed glass (RCG) and reclaimed aggregate (RA). RCA and RCG were investigated for use as either a coarse or fine aggregate replacement, while RA was investigated as primarily a coarse aggregate replacement. The scope of the Year 2 project was to investigate ferronickel slag (FNS) and power station bottom ash (BA) for use as replacement of fine natural sand in N-Class (non-structural) concrete. Conclusions and recommendations for each material are presented in the following subsections.

5.1 Key Findings

5.1.1 Recycled Concrete Aggregate

Key findings of the literature review and industry engagement for RCA are summarised as follows:

- RCA aggregate properties are influenced by the properties of the source concrete, resulting in variability between the properties of different batches of RCA.
  - The amount of attached old mortar on the RCA influences the density and water absorption of the RCA.
- Variability in the properties of RCA results in variable performance of the produced concrete.
- Coarse RCA can be used to replace up to 30% coarse natural aggregate in concrete grades up to 40 MPa while maintaining similar strength and durability properties to concrete produced using 100% NA.
- Higher levels of coarse RCA lead to degradation of most concrete properties, including strength properties (compressive, tensile, flexural), durability transport properties (chloride/sulphate resistance, carbonation resistance and concrete water absorption), shrinkage, creep and abrasion.
- Fine RCA is more detrimental to strength and durability properties and is not recommended for use in new concrete based on the literature review findings.
- ASR reactivity is a potential concern for RCA, since source concrete aggregate may be reactive. Some studies have shown that higher levels of SCM replacement (compared to MRTS70 (11/2018) recommended levels) may be required to mitigate ASR in RCA concrete, but these results appear to be linked to highly reactive original aggregates which may not be directly relevant to QLD aggregate behaviour.
  - At 30% inclusion of coarse RCA in concrete, the binary and ternary blends specified by MRTS70 (11/2018) are likely to be sufficient to mitigate ASR (25% inclusion of fly ash, 60% inclusion of GGBFS, or 25% fly ash combined with 5% silica fume)
- Minor adjustments to mix design and processes may be required to achieve equivalent performance. This could include adjustments to added water or proportioning of added RCA to optimise aggregate grading.
- Cost and availability of an RCA product which is suitable for use in new concrete appear to be key constraints, with the majority of RCA used for lower-risk applications such as drainage base.

5.1.2 Recycled Crushed Glass

Key findings of the literature review and industry engagement for RCG are summarised as follows:

- Properties of RCG aggregate include low water absorption (<1%) and densities within the AS 2758.1:2014 limits for normal weight aggregate.
  - Fine RCG can be crushed and graded to meet particle size and grading requirements.
  - Coarse RCG is generally flaky and prone to crushing and is not considered suitable for inclusion in concrete.
- Up to 30% replacement of fine aggregate with fine RCG in new concrete is unlikely to have a detrimental impact on strength and durability properties.
- Higher levels of replacement using fine RCG up to 100% are likely to be detrimental for strength properties and workability.
- The presence of contaminants in RCG may impact on the properties of the aggregate product
  - Sugar is known to act as a set retarder, and it currently unclear whether washing is required to consistently produce RCG which contains no detectable amount of sugar. Further investigation to identify appropriate limits on sugar contamination may be warranted.
  - Other contaminants such as metal, plastic, paper may also negatively impact aggregate and properties of produced concrete.
- ASR is of concern for RCG due to the presence of silica, but the review of literature indicates that the inclusion of SCMs at levels of 25% fly ash, 60% GGBFS or 4–8% silica fume combined with 25% fly ash are likely to mitigate ASR risk at replacement levels up to 30% (or higher).
- Cost viability of RCG depends on the proximity of RCG supply to concrete mix sites, and the degree of processing which is required prior to use in concrete (including any need for washing).
- There is reasonable availability of RCG for infrastructure applications (50–60k tonne/year in SEQ), though it is unclear what proportion of the supply could be dedicated to economically use in concrete applications.

5.1.3 Reclaimed Aggregate

Key findings of the literature review and industry engagement for RA are summarised as follows:

- Coarse RA may be produced from left over concrete using either water washing or chemical admixtures which absorb the free water in the concrete mix.
- Coarse RA will contain some proportion of mortar, which may result in higher water absorption and impacts on the strength and durability properties of the new concrete, although this was not quantified by the limited literature that was available.
- Coarse RA may be suitable as up to a 90% replacement of natural coarse aggregate, but this conclusion is based on one study only, conducted on the Cycrete product.
- Availability of leftover concrete for conversion to RA will depend on the variability of concrete ordering approaches and the production processes adopted by concrete suppliers.
- Cost of RA production will need to be weighed against the cost of disposing of leftover concrete.

5.1.4 Ferronickel Slag

Key findings of the literature review and industry engagement for FNS are summarised as follows:

- SLN in New Caledonia is the largest producer of FNS in the Australasian region and would be the primary source of the material for use in TMR concrete.
- Notable properties of FNS aggregate include low water absorption, low proportions of fines in the 0.6 mm – 0.3 mm fractions and some ASR susceptibility due to the presence of silica.

- Up to 50% replacement of fine aggregate using FNS should not significantly impact on most strength and durability properties, but a limit of 35% FNS inclusion is likely to be ideal to prevent mix bleed and assist with finishing. 35% inclusion is in line with past practice in New Caledonia concrete production.
- Inclusion of a minimum 25% FA as an SCM in FNS mixes is required to mitigate ASR potential. This is in line with TMR MRTS70 (11/2018) recommended practice.
- Where FA is included as an SCM in FNS concrete, a reduction in w/c ratio compared to 100% GP cement mixes is required to maintain equal strength with the 100% GP cement mix.
- Ternary fine aggregate blends of FNS with manufactured sand and natural sand are likely to provide optimal properties for commercial concrete production but discussion and trialling with concrete producers is recommended to identify optimal mix designs.

5.1.5 Power Station Bottom Ash

Key findings of the literature review for BA are summarised as follows:

- There are large stockpiles (400 million tonnes) of BA in Australia, with 800,000 tonnes additional produced by power stations each year in QLD alone.
- The presence of heavy metals in BA is of concern from an environmental (leaching) standpoint but is likely to be less of a concern for BA which is encased in concrete compared to raw storage of BA in ash dams.
  - Presence of heavy metals may be of concern from an environmental and work health and safety perspective for handling of the raw BA, and for placing of fresh BA concrete.
- The mechanical properties of BA aggregate are variable depending on the source coal and the way in which the coal is burnt. It has been indicated that NSW/QLD BA will have relatively similar properties across different sources, but this has not been verified.
- BA generally demonstrates water absorption higher than the AS 2758.1:2014 recommended limit, and density which places BA in the lightweight aggregate category. Other properties including typical grading are comparable to the AS 2758.1:2014 and MRTS70 (11/2018) recommended limits.
- Strength and durability properties of concrete incorporating fine BA are generally not significantly impacted at replacement levels up to 30%.
- At levels up to 100% replacement, there may be significant reductions in strength properties observed.
- The inclusion of BA in concrete may result in penalty to early age strengths at all inclusion levels, although strengths from 28-days onward are not significantly impacted at lower inclusion levels.
- There is likely to be an impact on workability resulting from inclusion of fine BA at any level, but it is likely that this can be managed through mix design and inclusion of water reducers or superplasticisers as appropriate.
- The inclusion of at least 20% fly ash as an SCM (partial GP cement replacement) in BA concrete has been shown to have positive impacts on strength and durability properties.
- Under some circumstances, heavy metals (particularly nitrate compounds of heavy metals) may act as set retarding agents, but this has not been specifically observed for BA.

5.2 Recommendations for Further Work

5.2.1 Recycled Concrete Aggregate

RCA appears to be technically viable for inclusion in concrete as up to a 30% replacement of coarse aggregate (> 4.75 mm particle size). Compared to the other materials considered in this study, there are more significant availability and behavioural barriers to the inclusion of RCA in new concrete, and most of the current RCA supply in SEQ is reportedly used in lower risk applications such as pavement base and drainage bedding. Further engagement with suppliers and concrete producers to determine availability are recommended prior to deciding whether further trials/testing are worthwhile.

If considering the development of a specification for RCA, the following limits are recommended in addition to the standard limits for coarse aggregate specified by AS 2758.1:2014 and any other relevant limits specified by MRTS70 (11/2018):

- Maintain combined inclusion of contaminants such as brick, metals, plastics and other demolition wastes at a level ≤ 1% by mass of the RCA product
- Coarse RCA water absorption ≤ 6%
- Inclusion of SCMs (as a percent of total cementitious materials) to manage ASR and improve strength/durability properties. The ternary blend described provides the lowest level of ASR risk but may not be necessary depending on the reactive potential of the RCA source aggregate. It appears likely that the ASR potential of Queensland RCA would be such that use of the binary blends would be acceptable:
  - ≥ 25% fly ash
  - ≥ 60% GGBFS
  - 25% fly ash combined with 5% silica fume (ternary blend).

A possible course of action to gain further confidence in the proposed SCM replacement levels is to undertake mortar bar testing and/or concrete prism testing on an RCA concrete produced using coarse RCA that is known to contain the most alkali-reactive Queensland aggregate. This would establish a 'worst-case' baseline which could be referred to for future mixes produced using Queensland RCA.

It is recommended that TMR considers allowing the use of up to 30% coarse RCA by total coarse aggregate mass in N-Class (non-structural) concrete which is intended to have a design life of 50 years or lower. A staged field trial approach is recommended prior to full scale implementation, whereby replacement levels of 10%, 20%, 30% are tested to verify field performance and investigate any issues associated with workability and mix design. This should include testing of RCA from various sources to identify any source-specific issues. If no adverse impacts on concrete strength or durability are observed for 30% replacement, consideration may be given to trialling 50% by mass replacement of coarse RCA.

5.2.2 Recycled Crushed Glass

Fine RCG (< 4.75 mm particles) appears to be the most viable option to be actively pursued for partial inclusion in TMR N-Class (non-structural) concrete. A limit of 30% inclusion of fine RCG in N-Class concrete (design life up to 50 years) is recommended based on the findings of the literature review. The RCG material should primarily consist of crushed soda-lime glass. If considering the development of a specification for fine RCG, the following limits on properties are recommended:

- Loss on Ignition (LOI) ≤ 2%
- Contaminant materials (ceramics, metals, specialist glasses etc.) limited to ≤ 2% by weight of RCG.
- Limits on concentration of chemicals, heavy metals and other attributes as per Table 6.2 of MRTS36 (11/2020)
- Inclusion of SCMs required to manage ASR:
  - ≥ 25% fly ash; or
  - ≥ 60% GGBFS; or
  - 25% fly ash combined with 4 to 8% silica fume (ternary blend).

Further confidence in the mitigating abilities of these SCM replacement levels could be gained by undertaking AMBT testing in accordance with AS 1140.60.1:2014, or concrete prism testing in accordance with AS 1140.60.2:2014.

The limits described above are in addition to the standard limits for fine aggregates specified by AS 2758.1:2014 and any other relevant limits specified by MRTS70 (11/2018).

It is assumed that RCG would need to comply with the AS 2758.1:2014 requirement relating to sugar content (shall test negative in accordance with AS 1141.35:2019). If this is found to be a limiting factor on implementation, further investigation into the typical sugar content of RCG product and the impact of sugar content on concrete properties may be warranted with the aim of determining whether an alternative limit is more appropriate.

It is recommended that TMR trials the use of RCG at varying replacement percentages from 10% up to 30% prior to any large-scale implementation. These field trials should use standard TMR mix designs, incorporating fly ash, GGBFS or silica fume at the levels specified above. These trials should be structured to facilitate the identification of any specific requirements for RCG blending or concrete mix adjustments. It is also recommended that TMR continues to monitor outputs from current field trials, including the Cairns Regional Council trial, Hanson Victoria trial and Pacific Highway upgrade trial.

5.2.3 Reclaimed Aggregate

There is currently limited information available to support the inclusion of RA in TMR N-Class concrete. It appears likely that the feasibility of RA development will be linked to costs associated with managing waste concrete. TMR is likely to have little influence over this, though it is recommended that TMR maintains contact with industry to gauge the availability and feasibility of this material going forward. If RA is thought to be potentially viable, further testing and field trials on replacement levels up to 90% of coarse aggregate by mass are recommended to gain more statistically significant data on strength and durability properties.

5.2.4 Ferronickel Slag

FNS appears to be a technically viable option for partial inclusion as a fine aggregate material in TMR concrete. An initial limit of 35% inclusion in TMR N-Class concrete (design life up to 50 years) is recommended based on the findings of the literature review. Sourcing and review of performance data from field applications in New Caledonia are recommended prior to any further works. Discussion with concrete producers is recommended to determine whether FNS is an attractive option is recommended prior to pursuing any further field trials or other research.

If considering the development of a specification for FNS, the following limits on properties are recommended:

- Iron unsoundness: negative
- Inclusion of SCMs to mitigate ASR:
  - ≥ 25% fly ash.

The limits described above are in addition to the standard limits for fine aggregates specified by AS 2758.1:2014 and any other relevant limits specified by MRTS70 (11/2018).

If inclusion of ≥ 60% GGBFS to mitigate ASR is a preferable option, AMBT (AS 1141.60.1:2014) or concrete prism testing (AS 1141.60.2:2014) may be undertaken to confirm the viability of this approach.

Prior to any full-scale implementation, trials with concrete producers should be conducted to facilitate the development of an optimum mix design and identify any issues associated with concrete production, placing or ongoing performance.

5.2.5 Power Station Bottom Ash

BA appears to be technically viable for partial inclusion in TMR N-Class concrete at levels of up to 30% replacement of natural fine aggregate by mass. Ongoing review of the Stanwell power station trial and investigation of BA aggregate properties from various sources are recommended in the short term. Prior to any full-scale implementation, trials with concrete producers should be conducted to facilitate the development of an optimum mix design and identify any issues associated with concrete production (particularly issues reported with material build-up in hoppers), placing or ongoing performance.

It is recommended to further investigate the potential environmental or health and safety hazards associated with heavy metal leaching from BA, and whether any specific mitigations or limits are required. This work should consider risks associated with handling of raw BA, placing of fresh BA concrete and long-term service of BA concrete.

If considering the development of a specification for BA inclusion in TMR concrete, the following limits on properties are recommended:

- Water absorption ≤ 6%.
- Inclusion of SCMs: at least 25% fly ash as a percentage of total cementitious materials.

The limits described above are in addition to the standard limits for fine aggregates specified by AS 2758.1:2014 and any other relevant limits specified by MRTS70 (11/2018).

The limit of 6% for water absorption is in line with previous recommendations for RCA (HB155:2002) and appears to be a reasonable figure, considering the Bayswater data, and the prevalence of significantly higher water absorption figures for BA tested in overseas studies. Depending on the outcomes of any further investigations, limits on the concentration of heavy metals within the BA product may also be required.

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Appendix A Survey Form Circulated to Industry

Question	Answer	Note
What types of recycled aggregates do you use or supply for concrete?		
What are the sources of the applied recycled aggregates?		
If possible, please provide as many details as possible about the company/supplier where the recycled aggregates are from.		
What are the pros of this application from your side? If possible, please be specific.		
What are the cons of this application from your side? If possible, please be specific.		
Do you have any properties of the recycled aggregates that you can share with us? It is very helpful if you can provide as much data as possible.		
Are there any other non-specified properties being tested for your own reference?		
Which projects have adopted this recycled aggregate? Could you please provide detailed information for each project?		
What specifications have been referred to for the use of recycled aggregates?		
What specifications have been referred to for the concrete using recycled aggregate?		
Do you have any in situ data or performance evaluations of the concrete using recycled aggregate?		