
ESTIMATING THE POTENTIAL RESIDUAL EXPANSION DUE TO ALKALI SILICA REACTION (ASR) IN IN-SERVICE REINFORCED CONCRETE STRUCTURES

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Declaration

I, Janina Prakash Kanjee, declare this dissertation as my unaided work. All the work in this document, save for that which is appropriately acknowledged, is my own. It is being submitted for the degree of DOCTOR OF PHILOSOPHY at the University of Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

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Abstract

This thesis investigated the extent of deterioration caused by alkali-silica reaction (ASR) in a 78-year-old reinforced concrete (RC) railway bridge in Johannesburg. Five assessments were conducted to characterise and quantify the extent of ASR damage. These included 2D potential residual expansion measurements at a micro-scale level using digital image correlation (DIC) and manual strain readings, focusing on localised expansion at the aggregate and cement paste interface. The study also explored correlations between micro-scale ASR strain and macro-scale deformations in the concrete.

The results showed that an on-site visual assessment identified varying degrees of ASR damage, leading to the extraction of concrete cores from different locations on the bridge. A modified damage rating approach, using a grid of 2 cm squares and the naked eye, was used to assess the extent of ASR damage based on 2D cylindrical images of each core's surface. Petrography and micro-XRF spectrometry confirmed that ASR was the primary cause of deterioration, characterised by cracks radiating from reactive aggregates and ASR gel presence. With respect to alkali reserves, concrete from the slightly damaged interior region had the highest sodium oxide equivalent content, 3.2 kg/m³. This value falls above the 2.0 kg/m³ limit for Witwatersrand aggregates, thereby validating the susceptibility of the concrete to ASR deterioration. The elastic modulus decreased with increased ASR damage, aligning with existing literature. Compared to slightly damaged concrete 18.0 GPa, moderately and severely damaged concrete had a 22.2% and 31.1% reduction in elastic modulus, respectively. Due to technical challenges, efforts to employ DIC for micro-scale residual expansion strains were unsuccessful. However, manual strain readings indicated that local equivalent normal strains were generally higher, and shear strains were generally lower than their respective bulk strains. The study found that the potential residual expansion of concrete displayed similar magnitudes of bulk expansion despite varying surface damage and alkali availability. Surprisingly, the potential residual micro-expansion displayed a similar trend.

The five assessments conducted in this study provided a general basis for quantifying the extent of ASR deterioration in a 78-year-old RC railway bridge. This approach can be used as a basis for the condition assessment of existing structures subject to ASR deterioration. Engineers can use the results to model and understand how ASR-affected concrete impacts the structure's behaviour and its remaining service life. The findings, in particular of the residual potential expansion, emphasised the crucial role of moisture in facilitating future expansion and damage

in the concrete due to ASR. Furthermore, owners of RC structures can obtain information about how long their RC structures, subject to ASR deterioration, may last before any repair is envisaged.

Dedication

This thesis is dedicated to
my late father, **Prakash Ranchod Kanjee**

“Teach your children well; their father's hell did slowly go by.”

- Crosby, Stills, Nash & Young

May his memory forever be a blessing and comfort.

&

a dear friend, the late **Nathan Ananmalay**

“I would not leave you in times of trouble

We never could have come this far. Mmmm

I took the good times; I'll take the bad times

I take you just the way you are.”

- Billy Joel

He supported and encouraged me to go on every adventure, especially this one.

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1. Introduction

1.1 Background

In the 1960s and 1970s, a significant portion of the reinforced concrete (RC) road and bridge structures in the greater Johannesburg region were constructed using Witwatersrand quartzite as the coarse aggregate. The quartzite was cheaply obtained from waste rock dumps around the gold mines. Unaware at the time, the silica in these aggregates chemically reacted with alkali ions present in the pore solution, to produce an expansive gel. The gel (hygroscopic in nature) swells in the presence of moisture, generating expansive tensile pressures and causing micro-cracking of the concrete. The net effect is a bulk expansion of the structure together with severe cracking, reduction of the concrete engineering properties and compromised durability of the concrete elements (Blight and Alexander, 2011; Godart and De Rooij, 2017).

National, Provincial and Local Agencies such as South African National Roads Agency (SANRAL), Gauteng Department of Public Works and Johannesburg Roads Agency (JRA) in South Africa are tasked with managing these structures, which are now more than 50 years old. Much resources have been spent on the repair, rehabilitation, and reconstruction of sections damaged by ASR, with an acknowledgement that the deterioration is ongoing and urgent, as illustrated in Figure 1-1. It is not economically feasible to demolish or temporarily restrict access to these essential structures should they be deemed structurally unsafe for their purpose. Therefore, an emphasis has been placed on managing the deteriorating infrastructure by prolonging and extending the service life of RC infrastructure through proactive maintenance, repair and rehabilitation.

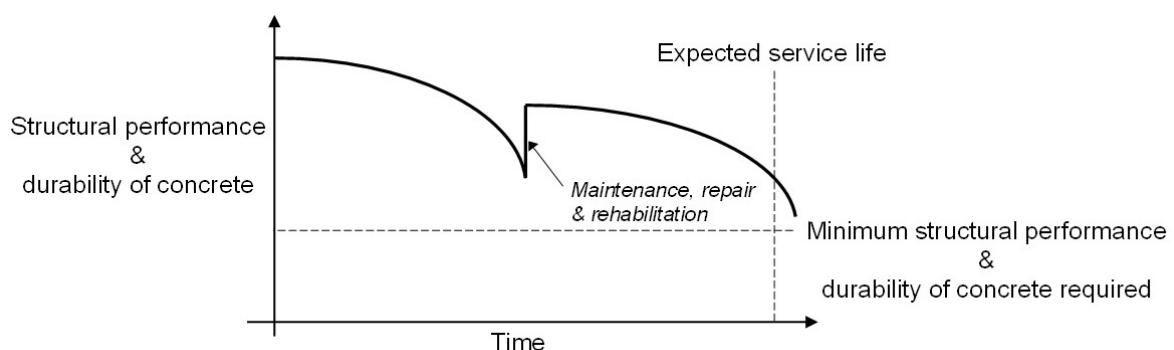


Figure 1-1: Schematic illustrating the deterioration and rehabilitation of a structure affected by ASR (Beushausen, Otieno and Alexander, 2021)

Various agencies utilise infrastructure asset management frameworks to ensure that road and bridge structures affected by ASR can operate efficiently with minimal disruptions. One crucial step in these frameworks is the collection of data on the condition with respect to structural and durability performance of the in-service structures. Engineers assigned to this role need to have a sound knowledge of the ASR mechanism, especially what signs of distress to look out for when conducting visual assessments of RC structures. The results of the evaluation indicate whether the structure's condition is deteriorating or not. To confirm the existence and extent of ASR deterioration, further laboratory testing on extracted concrete specimens may be required. Additionally, in-situ structural monitoring and load testing may be necessary to determine damage caused by ASR and its effects on the structure's performance.

The initiation of the ASR mechanism in concrete depends on the reactivity of the aggregates. In low-reactive aggregates, viable signs of deterioration may only show after 40 years on the structure's surface. However, when highly reactive aggregates were used in construction, visible damage was observed after the first five years of a structure's life (Godart and De Rooij, 2017). In ASR-affected structures, distress typically manifests as random 'map-cracking' on the structure's surface. This external damage results from internal damage caused by ASR, which primarily consists of micro and macro-cracking within the matrix and aggregates. Other signs of distress include ASR gel weeping from cracks, deformation and displacement of elements. Asset managers are particularly concerned about the effects of ASR on engineering properties, durability, and the likely potential expansion of the concrete due to ASR gel swelling pressures.

When assessing the potential swelling pressures in ASR-affected concrete, it is essential to first understand the strain development with respect to the initiation of ASR and the subsequent development of expansion in concrete. Concrete subject to ASR deterioration typically displays a characteristic "S" shaped curve with respect to the development of ASR gel swelling pressures and subsequent concrete expansion over time, as shown in Figure 1-2. Initially, the curve starts with minimal or zero expansion due to the initiation of the ASR mechanism and the formation of new ASR gel. This is followed by rapid expansion, which is attributed to the continued development of ASR gel. Subsequently, a gradually diminishing reaction rate is reflected in the expansion curve reaching a plateau. However, the "plateau" aspect of the trend is an artefact of the test method used to measure the potential strain development. When provisions for free-alkalis in the testing approach have been made using a simulated pore

solution, some concretes have shown steady expansion in the long term. Thus, for a given structure, engineers extract concrete cores from field structures for testing and try to characterise the remaining “potential residual expansion” to make statements about the likely future behaviour of the affected material.

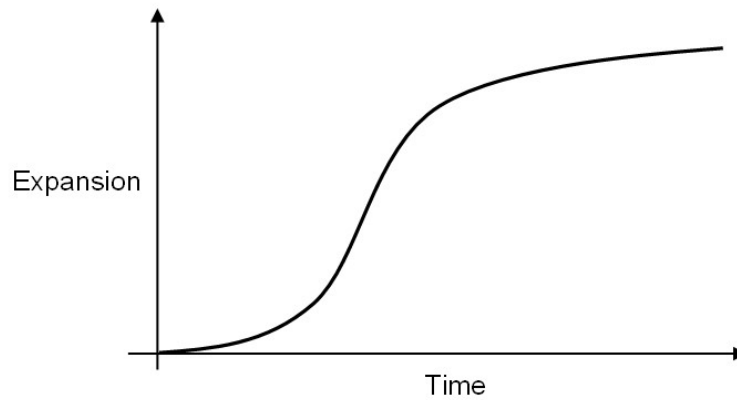


Figure 1-2: Schematic illustrating S-type expansion trend for ASR-affected concrete

The rate of reaction and subsequent expansion experienced by ASR-affected concrete depends on several intrinsic concrete factors, including aggregate composition and mineralogy, reactive aggregate content, alkalis contribution, and type of ASR gel reaction product (Rajabipour *et al.*, 2015). In addition, ASR development is dependent on various extrinsic factors, including the environmental conditions of where the structure is located, the conditions of reinforcement restraint, structural geometry, and changes in the structure’s usage (Fournier *et al.*, 2009; Aryan and Gencturk, 2021). Thus, all these factors must be considered when making reliable predictions of future expansion for a given RC structure. In summary, there is no straightforward answer when predicting the future behaviour of ASR-affected concrete.

Various authors recommend the use of accelerated expansion tests to determine the potential residual expansion of concrete. This process entails measuring the expansion of concrete cores from an in-service structure (Figure 1-3) submerged in an alkaline solution at an elevated temperature. Strain measurements are acquired through the installation of steel studs on concrete cores and using a Demec gauge to take periodic strain readings (lateral, longitudinal or diametric, depending on the position of the steel studs on the concrete core) over one year. Subsequently, conventional residual expansion measurements are a resultant summation of the expansion over the bulk of the material, as shown in Figure 1-3b.

At the micro scale level normal concrete, it comprises of three phases: cement paste, aggregate and inter-transitional zone (ITZ). However, in the case of ASR-affected concrete, there is an

additional phase – ASR gel. Considering the stress-strain development of the material from first principles, when concrete is subjected to expansive swelling pressure from ASR gel, it generates tensile stresses in the concrete matrix. The stiffer aggregate particles in the matrix restrain the movement of the cement paste, resulting in shear strains developing at the ITZ region. Using the traditional strain measurement approach, there is limited insight into the maximum strains that can develop at the aggregate-ASR gel-cement paste interface, which governs the ASR mechanism and subsequent potential residual expansion, as illustrated in Figure 1-3c.

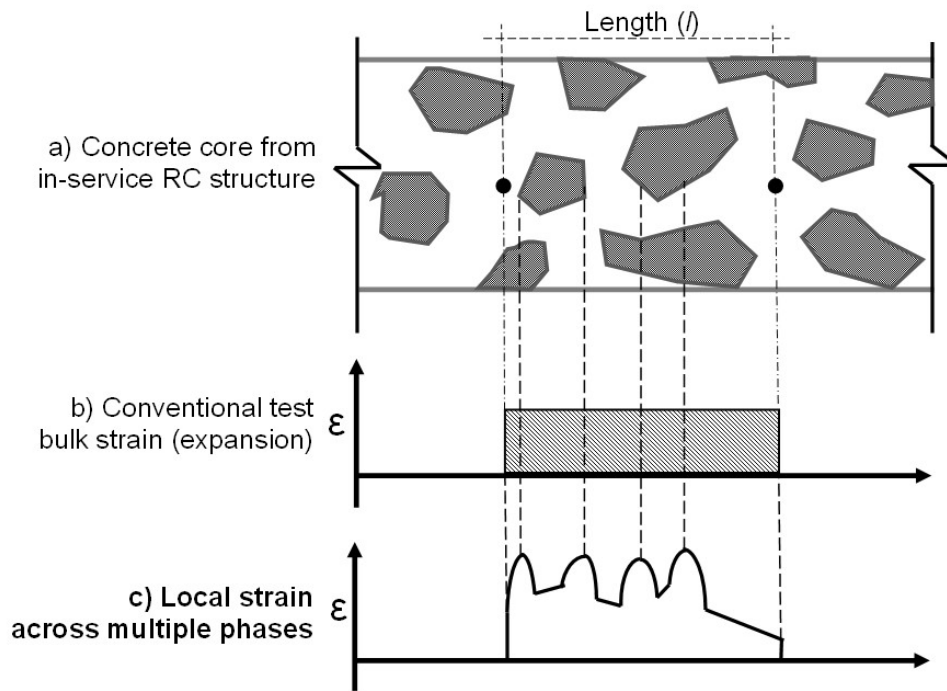


Figure 1-3: Determination of potential residual expansion (strain) of in-service concrete on a bulk versus a local scale

This research aimed to quantify these micro-strains by employing Digital Image Correlation (DIC) technique. DIC is a non-contact experimental method involving digital image acquisition and its processing to yield information about shape, motion and deformation over the region of interest (Sutton, Orteu and Schreier, 2009). Unlike traditional strain measurements that provide a single value over a specific length, DIC generates full-field displacement and strain measurements. The use of the technique has grown extensively over the past two decades owing to the relatively low cost, high accuracy, accessible and straightforward to conduct (McCormick and Lord, 2010).

By measuring and understanding the localised ASR stresses and subsequent strain development that govern the deterioration, this approach can be considered an additional diagnostic test that will provide insight into the influence of local deformation and the correlation thereafter to the global bulk concrete. This knowledge will assist in characterising the concrete with respect to

measuring the potential residual expansion and assessing the extent of ASR deterioration. These results can be used as inputs for service life prediction models that provide clients with an indication of the likely behaviour of the structure over a given period (e.g. in 50 years). This information then places the client in a knowledgeable position to plan and make decisions on how to manage the structures with respect to choosing appropriate repair techniques as well as budgeting for maintenance and necessary rehabilitation.

In summary, this study aimed to estimate the potential residual expansion of ASR in an RC structure currently in use in the greater Johannesburg region on both the macro (bulk) and micro (local) scales. In order to make accurate predictions of the behaviour of the RC structures affected by ASR deterioration, samples were taken from the actual structure and analysed. This approach was adopted as the concrete conditions from the field structures cannot simply be simulated or replicated using laboratory-casted concrete. Such factors include the condition of the concrete due to the casting process, the stress due to the loading on the structure, the availability of moisture on both the interior and exterior of the structure and environmental factors such as the climate in the region. In addition, the research investigated the presence of free alkali metals in concrete and their potential to facilitate likely residual expansion in the future. The study also examined the effect of ASR deterioration on the stiffness properties of concrete. The ultimate goal was to use the results from these tests and the potential residual expansion outcomes to provide crucial information on the prognosis of ASR in reinforced concrete structures currently in use in the greater Johannesburg region.

1.2 Problem identification

The two statements below provide a context of what needs to be addressed to better understand the mechanics governing localised ASR expansion and its correlation to the bulk expansion of the material. This knowledge can provide insight into modelling parameters, such as the potential residual expansion of the material, used to assess the extent of the deterioration and subsequent durability of in-service RC structures.

1. ASR is a localised deterioration mechanism that will only initiate when favourable conditions present themselves in the form of a reactive silica site, highly alkaline pore solution and moisture. Conventional test methods such as LPC N°44 (LPC, 1997), the Swiss approach (Leemann, Merz and Cuchet, 2021) and the Quebec method (Fecteau and Fournier, 2014) have been developed to assess the potential expansion of concrete subject to ASR deterioration. These techniques are able to quantify the bulk expansion

of the concrete. However, they cannot quantify the localised expansion across the different phases and interfaces (in particular at the aggregate and cement paste interface). Investigating the strain development at micro (local) levels can provide useful insight into the manifestation of the localised deformations on the bulk of the concrete and the correlation thereof.

2. The quantification of the potential residual expansion is a crucial parameter in assessing the degree of ASR degradation in an in-service RC structure. The ability to contextualise and frame potential residual expansion with respect to the service life of in-service RC structures is necessary to make reliable future predictions of likely deterioration.

This study aims to establish new knowledge by investigating internal strain development in concrete due to localised ASR mechanisms, and by determining how these local deformations correlate with macro-scale strain measurements, which are typically used to assess future residual expansion in concrete. By doing so, the study will enhance the ability to predict the future serviceability and remaining life of concrete structures affected by ASR.

Consider a 50-year-old RC structure subject to ASR deterioration, as shown in Figure 1-4a.

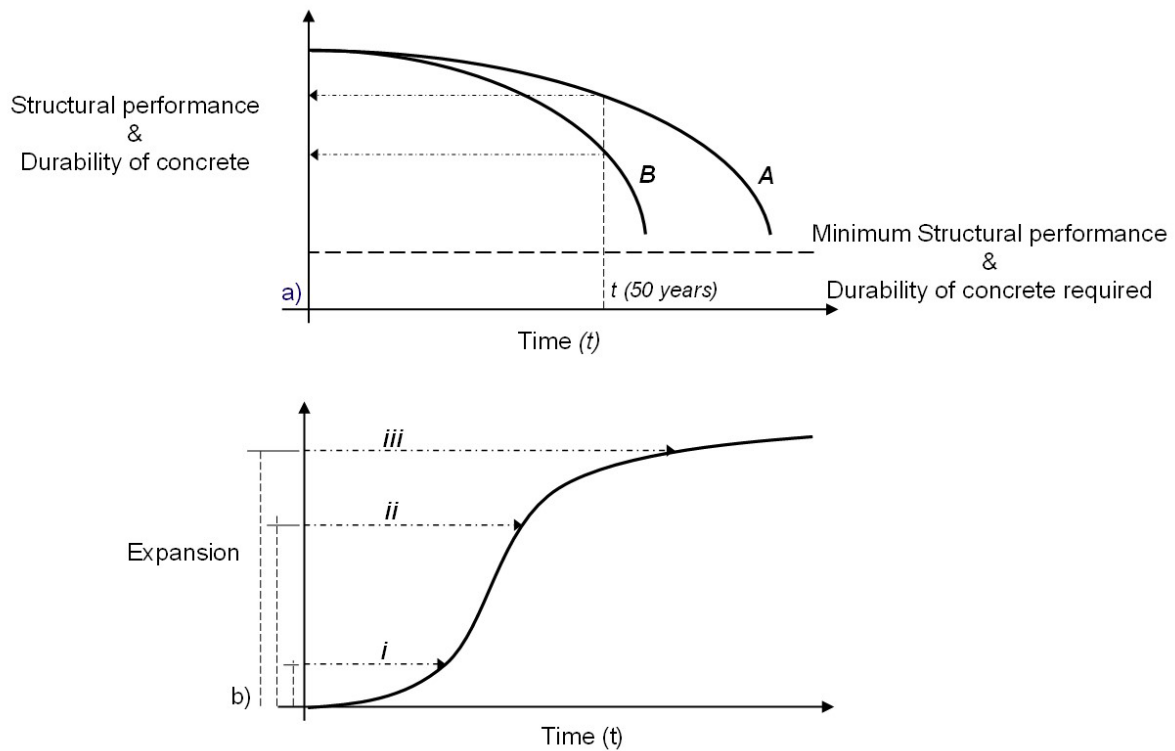


Figure 1-4: a) Deterioration of Element A and B from a 50-year-old structure affected by ASR, based on visual distress. b) Unknown concrete expansion historical data of a 50-year-old structure affected by ASR

During a visual inspection, the concrete surfaces of two structural elements, A and B, from the same structure display varying degrees of ASR distress. There can also be instances whereby the same element can have two areas, A & B, that display varying degrees of ASR distress. Considering the first scenario, the concrete surface of Element A shows little to no evidence of distress. However, Element B shows signs of severe map cracking and ASR gel weeping from the cracks. Concrete cores need to be extracted from the respective areas to assess the extent of ASR deterioration and, in particular, quantify the concrete's potential residual expansion.

Often, no historical or limited data is available on previous characterisation tests, which were done to assess the condition of concrete subject to ASR deterioration. Thus, the engineer has no insight into how much concrete expansion has occurred in the past 50 years. As illustrated in Figure 1-4b, it is unclear where the concrete lies on the characteristic strain curve. For example, the concrete could have experienced a small amount of expansion illustrated by point (i), in contrast to a significant amount of expansion illustrated by point (iii).

Thus, when considering the potential residual expansion results of the extracted concrete, the engineer can only “frame” the potential residual expansion for the concretes taken from various locations of the RC structure against one another, as seen in Figure 1-5. Because there is no data on past expansion tests, the results are plotted from zero strain, with an acknowledgement that concrete has undergone some degree of expansion (based on the signs of distress shown on the concrete surface).

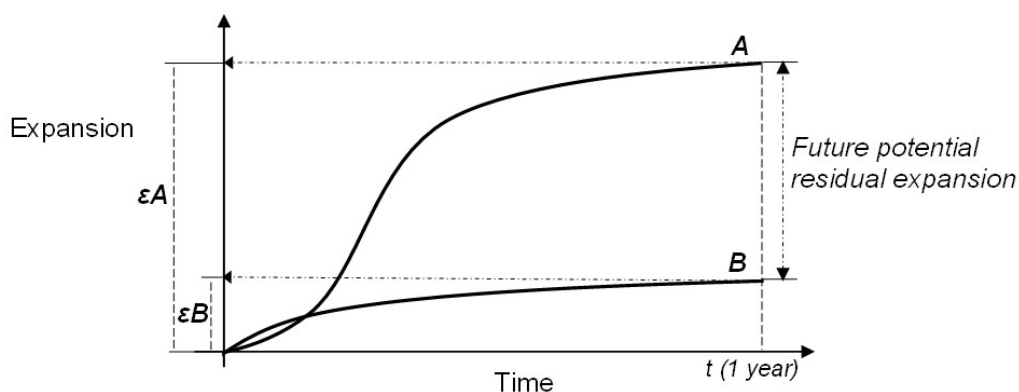


Figure 1-5: Framing potential residual expansion results from Element A and B from a 50-year-old structure affected by ASR

The residual expansion curve of the concrete from Element A (Figure 1-5), where ASR has not visually manifested, holds the greatest expansion potential, ϵ_A . The concrete from Element B (Figure 1-5), taken from the same structure, exhibits less expansion potential, ϵ_B , with regard to accumulated expansion. This results from ASR having advanced, and subsequently, the constituents required for the reaction to take place have been consumed. Framing Element A

concrete potential expansion alongside Element B concrete potential expansion can assist in trying to determine the extent of potential residual expansion of the Element A concrete with respect to likely future expansion of the in-service RC structure in the remaining service life of the structure.

An improved understanding of the role of localised mechanics governing the potential residual expansion of concrete will help improve the accuracy of predicting the durability performance and serviceability of in-service concrete structures subject to ASR.

1.3 Aims

The primary aim of this study is to:

1. Quantify and interpret the expansion exhibited (with respect to the swelling pressure measured indirectly through the strain development caused by ASR-induced swelling) by concrete subject to ASR deterioration on two scales (micro and macro).
1. Microscale (local) level: Quantify the ASR-induced displacement and subsequent strains generated locally across multiple phases, particularly at the interfacial transition zone (ITZ) between cement paste and aggregate in concrete, over a randomised 20 mm square area.
2. Macroscale (bulk) level – quantify the ASR-induced displacement and subsequent strains over the bulk material.
3. To determine if there is a correlation between the localised micro-strain and the expansion measured on the bulk of the concrete. This knowledge will assist in characterising the concrete with respect to measuring the potential residual expansion and assessing the extent of ASR deterioration.

1.4 Objectives

The objectives of this study include:

1. Diagnosis of an in-service RC structure subject to ASR deterioration using petrography and micro X-ray fluorescence analysis.
2. Determine the intrinsic concrete properties that favour ASR deterioration, such as free alkali content (Na^+ , K^+ and Ca^{2+})

3. Determine the concrete engineering properties owing to ASR deterioration, in particular, the modulus of elasticity.
4. Synthesise information to improve the understanding of the local mechanics of induced strain as a result of ASR gel formation and subsequent swelling.
5. Compare the potential residual expansion in concrete from slightly damaged (ASR-initiated) and damaged (as a result of ASR having advanced) areas of an existing RC structure.
6. Assess the impact of potential residual expansion of the damaged concrete in relation to the undamaged concrete on the service life of existing RC structures with regard to likely future expansion.

1.5 Hypothesis

Strain measurements in concrete subjected to ASR degradation can be tracked on a micro-scale within a randomised 20 mm square area. *(Two techniques were employed for this purpose: a newly developed Digital Image Correlation (DIC) rig for micro-scale measurements, and a conventional strain gauge setup, which served as a reference point of comparison.)* The residual potential strains measured at the micro-scale can be directly correlated to the residual potential macro strains observed in the bulk material of concrete undergoing ASR deterioration.

1.6 Scope and limitation

The scope of this study is limited to the following aspects:

1. Only one RC in-service field structure located in Johannesburg was considered in this study. Thus, the results are specific to the selected field structure and cannot be generalised to all RC structures subject to ASR deterioration.
2. 2D strains were investigated in this study to explore the localised mechanics of induced strain caused by ASR. It is crucial to recognise that when contemplating the implications of the laboratory results, with respect to concrete behaviour in the field, the 2D nature of strains should be considered with caution, bearing in mind that ASR expansion is inherently a 3D phenomenon.

1.7 Outline of thesis

This thesis is composed of five chapters.

Chapter 2 presents a review of the literature and is divided into two parts. Part I introduces the fundamental principles of alkali silica reaction in concrete; emphasis is placed on the mechanism governing the reaction and contextualising the scale of the issue within South Africa. Additionally, various diagnostic and assessment methods for ASR deterioration are covered in detail. Part II presents the fundamental principles of digital image correlation and reviews preceding research on the use of DIC in the testing of cementitious materials.

Chapter 3 presents a detailed description of the experimental work carried out in this study. This includes the field and laboratory experiments carried out in three sequential phases. Details on specimen geometry, the number of specimens and various diagnostic and assessment techniques are provided.

Chapter 4 presents the results, analyses and discussion of the experimental work (as described in Chapter 3). In this chapter, results for all three phases are discussed. Empirical correlations are established between the extent of ASR damage and variables such as structural defects, petrographic features, alkali metal content, stiffness and potential residual expansion. The results are synthesised to obtain a holistic view of the inter-relationships between the extent of ASR deterioration and other experimental variables.

Chapter 5 presents the conclusions of this work. Based on identified gaps and limitations, recommendations for future research are also noted.

Chapter 2 Literature review

2.1 Introduction

This chapter has been divided into two parts, as follows:

1. Part I: Alkali–silica reaction in concrete;
2. Part II: Digital image correlation (DIC)

The first part provides background on the deleterious alkali-aggregate reactivity in concrete, particularly on alkali silica reactivity. It provides an overview of the fundamental chemistry and constituents that govern the mechanisms of the alkali-silica reaction. The structural consequences of ASR deterioration on RC structures are also touched on. Various techniques used to diagnose and assess the impact of ASR on structures are critically reviewed. In particular, the potential residual expansion of concrete subject to ASR is a useful parameter to determine the extent of ASR deterioration and the structure's service life. Lastly, the current test methods used to determine the potential residual expansion and their limitations are reviewed.

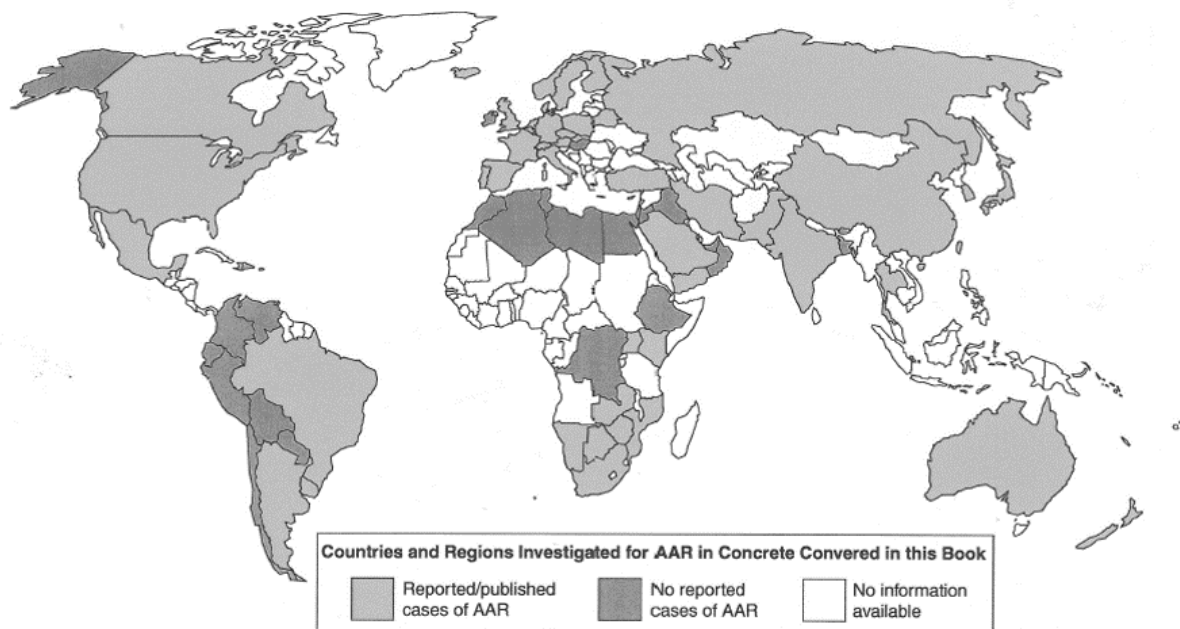
The second part provides background on digital image correlation (DIC) as a full-field displacement and strain measurement technique. The basic concepts of DIC are covered. Lastly, previous studies showing the application of DIC to obtain displacement and strain measurements in cementitious materials are presented.

The chapter concludes by emphasising the need to understand the mechanics governing localised ASR expansion in concrete. This is important as most conventional test methods used to assess the potential expansion of concrete subject to ASR deterioration have been developed to quantify the bulk expansion of the concrete. However, based on the critical review of the literature, it will be clear why the basis for quantifying the localised expansion across the different phases and interfaces (in particular at the aggregate and cement paste interface) should be explored. This would ultimately provide a more accurate method of assessing and predicting the durability and inherent service life of RC structures subject to ASR.

2.2 Part I: Background of alkali reactivity in concrete

Alkali aggregate reaction (AAR) was first observed and identified in concrete pavements in California by Thomas Stanton during the late 1930s. He published the first comprehensive investigation describing the reaction and its effects in December 1940 (Stanton, 1940). Since

then, a significant amount of research has been undertaken to (a) understand the fundamental mechanism of ASR (Chatterji, 1979; Glasser, 1991; Poole A.B, 1992), (b) understand and predict the long-term durability, serviceability, and safety of the structures affected by alkali-silica reaction (Bruno and Jonathan, 2016; Ferche and Vecchio, 2022; Hei *et al.*, 2022) (c) develop rapid and reliable test methods to identify reactive minerals and aggregates (Diamond, 1976; Ahmad et al., 2018) 2018)and (d) propose and evaluate effective methods to mitigate ASR in new or existing structures (Thomas, Fournier and Folliard, 2008; Fournier *et al.*, 2010; AASHTO PP 65-11, 2013). These studies have been driven partly by the significant costs involved in addressing the damage caused by AAR and partly by the worldwide extent of the problem, as seen in Figure 2-1: **A world map indicating countries and regions where cases**



of AAR in concrete structures have been identified, investigated, reported or where avoidance specifications are in place (Poole, 2017)Figure 2-1.

Figure 2-1: A world map indicating countries and regions where cases of AAR in concrete structures have been identified, investigated, reported or where avoidance specifications are in place (Poole, 2017)

As observed in Figure 2-1, concrete susceptible to AAR has been reported worldwide in numerous case studies, with few countries or regions being exempt. This indicates the significance of this particular form of concrete degradation and the importance of preventing new structures from developing AAR. Additionally, for existing structures subject to AAR degradation, it is crucial to find suitable approaches to stop the deterioration from continuing. Studies have shown that two types of deleterious alkali-aggregate reactions can occur in concrete, depending on the type of aggregate involved in the reaction:

1. Alkali carbonate reaction (ACR) (Swenson, 1957)
2. Alkali silica reaction (ASR) (Stanton, 1940)

Alkali-carbonate reaction (ACR) involves carbonate aggregates such as limestone and dolomite (Hadley, 1961; Swenson and Gillott, 1964), while ASR involves disordered siliceous aggregates. Unlike ASR, ACR does not produce a gel. Instead, the expansion occurs due to the alkali hydroxides reacting with small dolomite crystals, leading to a dedolomitisation reaction. This reaction eventually results in extensive cracking and expansion of the coarse aggregate particles (Swenson and Gillott, 1964; Dolar-Mantuani, 1983).

This study focused primarily on ASR, the alkali-aggregate reaction associated with Witwatersrand quartzite, which is the aggregate that was used in the construction of in-service RC structures in the greater Johannesburg region.

2.3 Reactants involved in the initiation of ASR in concrete

Alkali-silica reaction is a deleterious swelling reaction that occurs only when the following three conditions are present simultaneously in the concrete matrix (Swamy and Al-Asali, 1988; Fournier *et al.*, 2010): the presence of reactive silica in the aggregate, a sufficiently high alkali pore solution and sufficient moisture available in the concrete. The following subsections elaborate on these factors in greater detail and their role in facilitating ASR in concrete.

2.3.1 Reactive silica in the aggregate

In the production of concrete, the majority of rock types used as aggregates contain several mineral components. The crystalline silica (SiO_2) found in aggregates, typically in the form of quartz, is chemically inert. However, aggregates prone to ASR contain a “reactive” form of amorphous silica. The reactivity of siliceous aggregates depends on not only the mineralogy but also the mechanics of the formation of the aggregate (Katayama, 1997) and the degree of deformation of the quartz (Monteiro *et al.*, 2001), whether natural or induced as a by-product of industrial processes.

Aggregates susceptible to reactivity exhibit a poorly crystalline structure and contain many lattice defects, residual strains, or internal micro-cracks (ACI Committee, 1998). Examples of poorly crystallised silica include opaline silica, chalcedony, cristobalite, tridymite, volcanic glasses, cryptocrystalline quartz and strained quartz (Glasser, 1991). Table 2-1 shows commonly used concrete aggregates that are known to be reactive.

Table 2-1: Rock types commonly used as aggregate in concrete and their potentially reactive components with alkalis in concrete (Alexander and Mindess, 2005)

Geological type	Rock type	Reactive component
Igneous	Rhyolite, andesite, dacite, obsidian, tuff	Silicic to intermediate silica-rich volcanic glass, devitrified glass, tridymite
	Basalt	Chalcedony, cristobalite, palagonite, basic volcanic glass
	Granite	Strained quartz
Metamorphic	Gneiss, schist	Strained quartz, microcrystalline quartz
	Quartzite	Strained and microcrystalline quartz
	Hornfels, phyllite, argillite	Strained quartz, microcrystalline to cryptocrystalline quartz
Sedimentary	Sandstone	Strained and microcrystalline quartz, chert, opal
	Greywacke (may be partially metamorphosed)	Strained and microcrystalline to cryptocrystalline quartz, opal
	Shale, tillite, chert	Strained quartz, microcrystalline to cryptocrystalline quartz
	Argillaceous dolomitic limestone (marl)	Dolomite, clay minerals exposed to dolomitisation
	Recycled glass	Amorphous silica from glass

*Microcrystalline quartz < 100 µm, cryptocrystalline quartz < 10 µm

From Table 2-1, it can be seen that a common reactive feature in most rock types susceptible to ASR is the presence of strained quarts and the size of the quarts grains ranging from microcrystalline to cryptocrystalline (100 µm to 10 µm). Both fine and coarse aggregate constituents can exhibit reactivity, with reactive components' concentration, density and particle size influencing overall expansion. Aggregate density should also be considered, as porous materials provide void space for expansive gel accommodation without disrupting the concrete matrix (Mladenovic *et al.*, 2004).

Various test methods have been developed to evaluate aggregate reactivity. The choice of which procedure to use depends on the nature of the assessment and, subsequently, the desired information. A summary of aggregate tests for susceptibility to ASR and relevant test standards is given in Table 2-2, where aggregate assessment is considered in three main categories: initial non-quantitative screening tests (used to make a provisional assessment), indicator tests to differentiate between potentially reactive and innocuous aggregates; and performance tests, giving information on limiting alkali contents to avoid damaging expansions.

Based on the information in Table 2-2, issues may arise when selecting the appropriate procedure to determine the aggregate reactivity for a given project. This is a result of differing national standards and test methods in which criteria and approaches may vary, to quantify the aggregate reactivity.

Table 2-2: Aggregate assessment of AAR (excluding carbonate rock types) (Blight and Alexander, 2011)

Test designation and purpose	Test standards
1 Initial screening tests (non- or semi-quantitative)	
Petrographic examination procedures for aggregates	ASTM C294 & C295 BS 812:Part 104 BS 7943 RILEM TC 191-ARP (AAR-1)
Gel-pat test detection of reactive silica	Appendix to BS 7943
2 Rapid indicator tests (to determine whether aggregates are potentially reactive or innocuous)	
Potential alkali reactivity of aggregates	ASTM C289
Potential alkali-reactivity of cement–aggregate combinations (mortar-bar method)	ASTM C227
Accelerated mortar bar (mortar prism) test	AST C1260 BS DD 249 CSA: A23.2-25 A RILEM TC 191-ARP (AAR-2) SANS 6245
3 Performance Test (to provide information on limiting alkalis or structural performance)	
Concrete prism method	ASTM C1293 BS 812:Part 123 CSA A23.2-14A RILEM TC191-ARP (AAR-3)
Modified concrete prism method	CSAA23.2-14-A ASTM C1293
Long-term structural monitoring	RILEM TC191-ARP (AAR-4)

Nixon and Sims (2016), as part of the RILEM technical committees, propose that aggregate reactivity should be classified as non-reactive, moderately reactive, highly reactive, or very highly reactive. It has been reported that as little as 2% “reactive” silica in the aggregate can cause significant concrete distress (Poole, 2010). Table 2-3 lists the reactivity of aggregates with respect to % expansion for various amorphous silica contents.

Table 2-3 illustrates the expansion results for various aggregate types using the concrete prism test (CPT), an indicator of field performance, and the accelerated mortar bar test (AMBT) to detect reactive aggregates. In the CTP results, flint with a 50 % amorphous silica content resulted in the most significant expansion, 1.482, compared to Chert, with 71.2 % amorphous silica content that resulted in only 0.42 expansion. In the AMBT tests, Opal aggregate with 84% amorphous silica content resulted in the greatest expansion of 0.89 compared to Sintered silica fume with 85 % silica content, which resulted in only 0.65 expansion.

Table 2-3: Expansion of aggregates with different amorphous silica content.

Type of aggregate	Source	Amorphous silica (%)	Expansion (%)		References.
			CPT (1 year)	AMBT (14 days)	
Chinese limestone	China	4.6	0.064	-	Grattan-Bellew et al., 2010
Spratt	Ottawa	29.7	0.275	0.375	Shehata and Thomas, 2000
Ferronickel slag	SLN, New Calidonia	50.0	-	0.653	Saha and Sarker, 2016
Flint	Thames Valley	50.0	1.482	0.61	Thomas et al., 2011
Waste glass	Turkey	71.0	-	0.82	Topçu, Boğa and Bilir, 2008
Chert	Texas	71.2	0.42	0.7	Moser et al., 2010
Glass	Canada	73.0	-	0.631	Shafaatian et al., 2013
Opal aggregate	Celite	84.0	-	0.89	García-Lodeiro, Palomo and Fernández-Jiménez, 2007
Sintered silica fume	-	85.0	-	0.65	Juenger and Ostertag, 2004
Dolomitic limestone	Pittsburgh, Kingston	91.5	0.616	-	Grattan-Bellew et al., 2010
* Allowable limit CPT (1 year) - $E \leq 0.04$ [ASTM C1293], AMBT (14 days) - $E \leq 0.1$ [ASTM C1567]					

Studies highlighted in Table 2-3 revealed that the ASR-related expansion is not necessarily proportional to the content of reactive minerals in aggregates and varies non-monotonically with aggregate size and proportion. At low proportions, the expansion increases with the volume of reactive phases, but beyond a certain amount, it decreases. With high proportions, no expansion occurs at all. The proportion of reactive aggregates corresponding to the peak expansion is called the “pessimum content”.

In a study by Hobbs (1988), a reactive aggregate containing 5% opal was mixed with a non-reactive aggregate in increasing proportions, as illustrated in Figure 2-2. As the amount of opal increased from 0.5% to 2% by mass, the expansion gradually increased. However, expansion begins to decrease when the opal phase exceeds 2.5 wt.%. Beyond 4.5 wt.% of opal in the aggregate, no expansion was observed.

This expansion curve in Figure 2-2 can be divided into four regions as a function of the opal content. In region A, the reactive silica content is low, which means that gel formation is too small to induce any expansion or cracking. In Region B, the amount of expansive gel is such that it can easily induce expansion and mortar or concrete cracking. In Region C, the opal content is higher than in the previous region, but the expansion decreases with the increase in opal content. In Region D, no expansion occurs despite the high opal content. In this region, the reactive sites are numerous and well dispersed, which causes the gel reaction to be very fast and consume a large proportion of soluble alkalis. Although the global proportion of ASR gel

is high, the proportion of gel in each reactive site is too low to cause expansion pressure higher than the tensile resistance of the mortar or concrete prism, so no damage occurs.

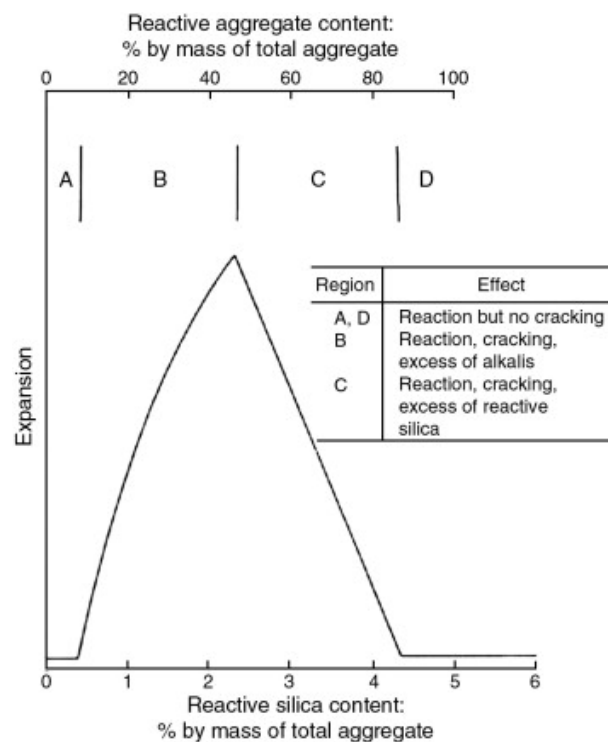


Figure 2-2: The pessimum behaviour exhibited by an aggregate with opaline silica (Hobbs, 1988)

In conclusion, when assessing an aggregate's susceptibility to ASR, merely identifying a reactive rock type based on its internal structure is insufficient to evaluate its reactivity. It is also essential to consider the pessimum content of silica present, the particles' size, and the dissolution rate of silica. These factors are crucial in determining the potential for maximum concrete expansion owing to ASR as a function of aggregate reactivity (Ichikawa, 2009; Gao *et al.*, 2013; Kawabata and Yamada, 2017).

2.3.2 A sufficiently high alkalinity of the pore solution

Alkalis, particularly those of Na_2O and K_2O , are a minor component of Portland cement clinker and originate from the raw materials used to manufacture Portland cement (Jawed and Skalny, 1977). Due to the difference in the composition of raw materials used in production, the alkali content in cement manufactured by different factories varies greatly in different regions (Mengxiao S, Qiang W and Zhikai Z, 2015).

The alkali in cement exists in two forms: sulphate and solid solution in silicates or aluminates in clinker (Jawed and Skalny, 1978). When Portland cement comes into contact with water, the sulphates dissolve into a liquid phase due to the high dissolution rate. As a result, the pore

solution of concrete made with a high-alkali cement contains almost entirely Na^+ , K^+ and OH^- ions with very low concentrations of Ca^{2+} , SO_4 and Cl^- , and the pH of the pore solution is in the range of 13 to 14, depending on the alkali level. Compared if no or very little alkali is present, then the pH of the pore solution is between 12.5 and 12.9.

Diamond (1989) confirmed the correlation between the cement-alkali content and the concrete pore fluid's OH^- concentration (pH). An environment with higher alkalinity enhances the dissolution of reactive silica (Fournier *et al.*, 2010), facilitating ASR. In addition, the fineness of Portland cement also plays a vital role in the alkali content; finer cement releases alkalis more rapidly, consequently accelerating expansion (Bektas, Wang and Ceylan, 2008).

It is customary to refer to the alkalis in cement in terms of sodium oxide equivalent, as shown in Equation 2-1.

$$\%Na_2O_{-eq} = \%Na_2O + (0.658\%K_2O) \quad \text{Equation 2-1}$$

ASTM C 150 (2012) states the value of 0.6 % $\text{Na}_2\text{O}_{-eq}$, as the threshold below which ASR is unlikely to result in an expansive reaction. However, the product of the alkali content of the cement and the cement content of the concrete determines the alkalinity of the pore solution and, therefore, to what extent a reaction with a reactive aggregate will occur. Thus, it is more appropriate to limit the alkali content per m^3 of the concrete than to restrict the alkali content of the cement.

Putterill and Oberholster (1985) demonstrated that the alkali content limit per volume (m^3) of the concrete varies based on the type of aggregate used. International approaches to mitigate ASR effects differ, as shown in Table 2-1, with some countries having a single alkali limit adjusted for specific conditions and risks. It can be seen that the limits specified for total alkali in concrete vary, ranging from 4.0 kg/m^3 to 1.8 kg/m^3 . The range in limits can be attributed to varied approaches countries have chosen to adopt based on the type and subsequent reactivity of aggregates occurring in various countries, the cement alkali limits, the use of supplementary cementitious materials, and whether a prescriptive or performance approach is preferred.

Table 2-4: Alkali limits for controlling expansion from reactive aggregate used internationally. (Poole, 2010)

Country & specification	Specified total alkali in concrete (kg/m ³)	Comments and limit in concrete performance guidelines
USA ASTM A23.1 & 2 – 2016	1.8-3.0	Prescriptive and performance approaches were used. Performance using ASTM C1293 & C1567
Canada CSA A23.1 & 2 – 2014	2.0-3.0	Limits based on environment and risk for each structure
UK BRE Digest 330.2 – 2004	3.5	Alkali limits based on cement alkali limits and aggregate reactivity
RILEM AAR 7.1 – 2016	2.5-3.5	Limits based on aggregate reactivity, with classification being low, medium or high
Japan JIS A5308 – 2009	3.0	Prescriptive limit based on andesite data. Performance limits from 1.2-3.0 kg/m ³
Australia HB79 – 2015	2.8	The risk with reactive aggregate is almost always controlled using SCMs
New Zealand CCANZ TR3 – 2012	2.5	Prescriptive limit is widely used. Performance limits from 1.8-3.0 kg/m ³
South Africa SANS 6245 - 2006	2.0-4.0	Limits based on aggregate reactivity. SCMs use encouraged in specifications to control ASR.

¹ SCM: Supplementary cementitious material, such as fly ash or slag, also termed ‘additions’

The work done by Thomas, Fournier and Folliard (2013) in Figure 2-3 illustrates the prescribed total alkali limits (kg/m³) for a concrete mixture design, factoring in cement alkali content, the maximum cement content used, and the reactivity of the aggregate. This is a useful graph as it guides the determination of a concrete mixture design to limit the susceptibility of ASR developing in an RC structure by taking into consideration the alkali content, cement content as well and aggregate reactivity, which was previously not considered together. Omar et al, (2025) developed a testing framework that determined the optimal SCM dosage to mitigate ASR, taking into account the varying degrees of aggregate reactivity. The framework also, incorporates the use of the modified R3 test (which evaluates the reactivity levels of SCMs based on their pozzolanic reactivity) and bulk resistivity. These tests evaluate the pore solution chemistry and the internal structure characteristics of the concrete, which are likely to facilitate ASR.

An additional internal source of alkalis may stem from the leaching of minerals in the aggregate (Kawamura, Koike and Nakano, 1989). Common minerals in aggregates that have the potential to release alkalis include micas (K), feldspar (Na and K), illite clays (K), and volcanic ash (Na and Na). Constantiner and Diamond (2003) indicated that aggregates may contribute up to 10% of the alkalis in the pore solution. Another possible source of alkalis can be derived from

chemical admixtures such as sodium lignosulphonate and mixing water, particularly recycled water, used in concrete production. External sources of alkalis may come from de-icing salts or seawater.

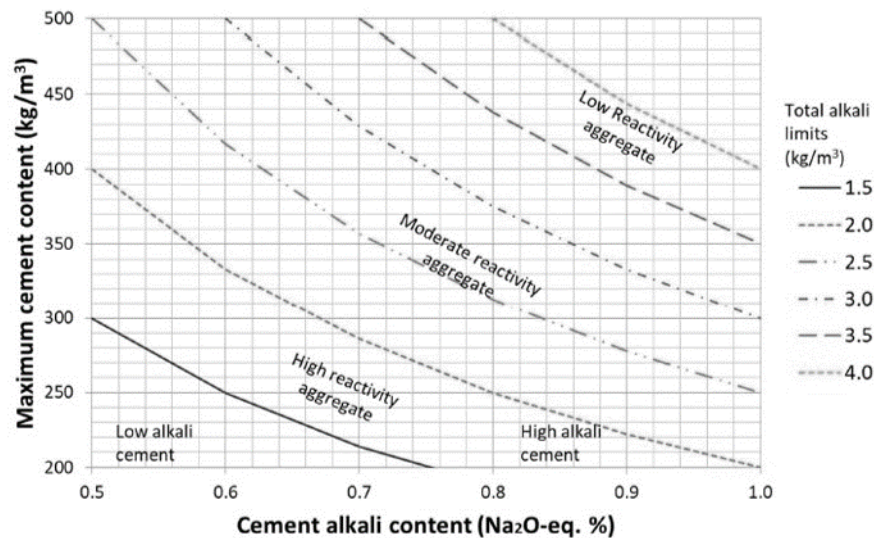


Figure 2-3: Maximum allowable cement contents for different cement alkali levels (Thomas, Fournier and Folliard, 2013)

Furthermore, only some of the total alkalis dissolve in the pore solution during cement hydration and are available for aggregate reaction; they are termed active alkalis. One hypothesis suggests that only alkali metal ions in solution contribute to ASR initiation, while those bound in solid phases do not. The proportion of active alkali varies, with South African CEM I cements having, on average, about 85% of the total Na₂O-eq available. Blended cements have a lower proportion of active alkalis, ranging from 25-55%, due to materials like fly ash and slag, where a substantial portion of alkalis are not soluble (Malvar, 2002; Oberholster, 2009).

In conclusion, a highly alkaline pore solution in concrete is required to facilitate the development of ASR. It is important to consider not only the type of cement used but also the binder content, aggregate reactivity, and additional sources of alkalis. Additionally, the total alkali limit is used as a precautionary measure to prevent ASR development in newly constructed structures.

2.3.3 Sufficient moisture availability in concrete

Moisture is always present in the pores of concrete, except for the surface layer, which may be dry. Subsequently, concrete's relative humidity (RH) will typically remain above 80%. In concrete, moisture is a crucial factor for the initiation and progression of ASR. The presence

of moisture in ASR-affected concrete is primarily due to the mixing water (w/c) and the ingress of water from external sources such as rain, ambient environmental conditions, drainage, etc., or a combination of these factors. Moisture plays a dual role in the initiation of ASR. Firstly, it acts as a carrier for the alkalis and hydroxyl ions, which initiate the ASR reaction. The transportation of ions can be further enhanced in concrete with high water/cement ratio, which creates a highly porous and permeable medium. Secondly, moisture is absorbed by the hygroscopic ASR gel reaction product, causing swelling and developing pressures that may crack the concrete (Jones and Poole, 1986).

Experimental studies indicate that ASR gel formation ceases below an internal relative humidity (RH) threshold of 80%, halting concrete expansion (Stark, 1991). This result is attributed to the absorption capacity of the gel greatly diminishes when the RH of the air in the pores falls below 85% (Bazant and Steffens, 2000a). However, above 80%, the RH expansion increases significantly. It must be noted that the ASR gel swelling that can arise in this condition may not be sufficient to exert significant pressure on the surrounding paste. If the concrete is re-exposed to water or moved to a more saturated condition, the reaction rate and expansion are significantly improved. Evidence of the effect of internal relative humidity is that reaction product is typically only observed from about 60 mm inward in cores drilled from exposed concrete structures as seen Figure 2-4. Two halves of a core showing that the reaction product does not develop in the outer. 60 mm of concrete, apparently because the relative humidity is too low. As a result, the concept of the RH threshold and limiting moisture availability has been used to prevent and maintain ASR-affected structures.

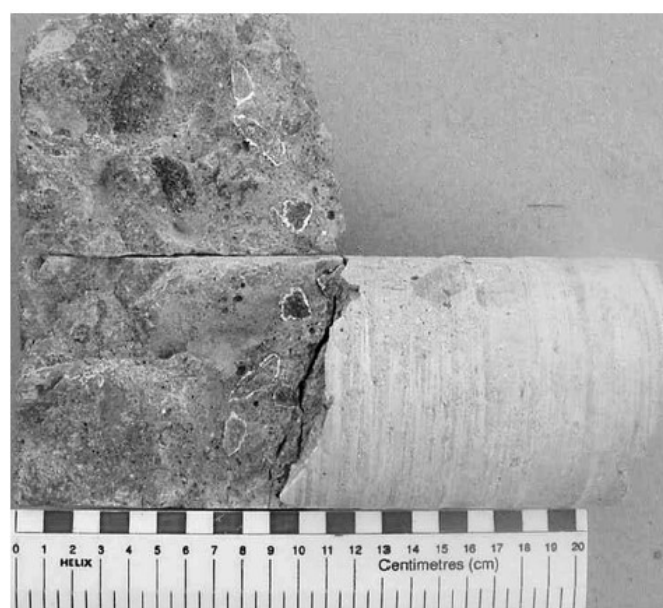


Figure 2-4: Two halves of a core showing that the reaction product does not develop in the outer ca. 60 mm of concrete (Oberholster, B, 2009)

However, when conducting accelerated tests Kurihara and Katawaki (1989) and Tomosawa, Tamura and Abe (1989) reported a threshold of 75%, when the temperature was 40 °C, while Poyet et al., (2006) reported a threshold of less than 59% at 60 °C. The studies from which the reported RH% thresholds differ are attributed to various temperatures, sample sizes, reactivity level of aggregates, and aggregate type (fine versus coarse) (Olajide, Nokken and Sanchez, 2023).

Olajide et al. (2024) have noted that moisture and temperature are known to exacerbate ASR in concrete; however, the extent of damage and the reduction in mechanical properties across a wide range of moisture and temperature conditions, as well as the evolution of these responses over time, are not fully understood. Wet and dry cycles create moisture gradients in the concrete, with high RH at the core (> 80–90 %). While exterior surfaces dry, triggering subsurface expansion and surface cracking. These heterogeneous moisture patterns result in thicker members retaining moisture longer and are more vulnerable inside, even when exposed to ambient drying. In one study, Olusola et al. (2023) demonstrated that the coupling of ASR with drying shrinkage due to the dry regimes can accelerate the reaction at late ages, and the cyclic conditions can attain a higher ultimate expansion.

In summary, the presence of moisture in concrete pores, usually exceeding 80% relative humidity, plays a crucial role in initiating and advancing ASR. Furthermore, it is important to restrict external sources of moisture in existing reinforced concrete structures to prevent the concrete from further deterioration.

2.4 Chemistry initiating ASR development in concrete

The initiation of ASR in concrete is a physio-chemical (acid-base) reaction. The acid reactant is silica in the solid state, and the basic reactants are potassium and/or sodium hydroxide in the pore solution. While, the reaction medium is water. The reaction involves three sequential reactions: (i) dissolution of metastable silica, (ii) formation of silica gel and lastly, (iii) swelling of the gel (Chatterji, 1979; Ichikawa and Miura, 2007; Oberholster. B, 2009). Figure 2-5 provides an overview of the initiation and development of ASR in concrete.

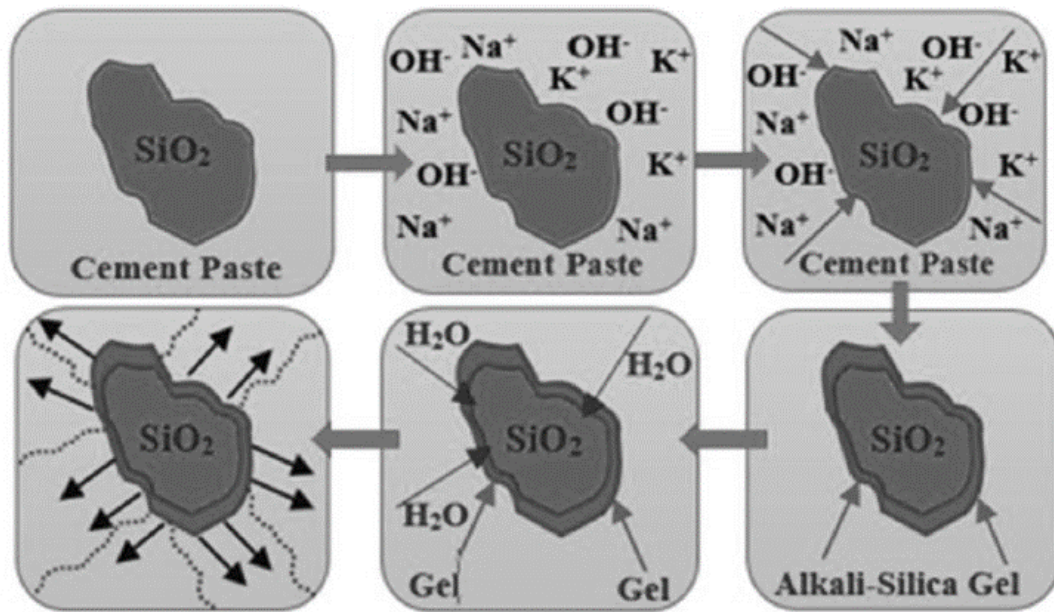


Figure 2-5: A schematic of ASR mechanism in concrete (Diamond S, 2000; M.D.A. Thomas et al., 2011; Saha A.K et al., 2018)

In the highly alkaline environment, the hydroxyl ions (Na_2O and K_2O) attack the non-bridged oxygen atoms in the amorphous silica structure, rupturing the Si-O-Si bonds. This results in a network of dissolved silica, as seen in Figure 2-6a, which allows hydroxyl ions to penetrate further into the interior phase of the poorly crystallised silica. Silica dissolution is often the slowest, controlling the rate of ASR in concrete (Brantley, Kubicki and White, 2008). Different aggregate types have different dissolution reaction rates.

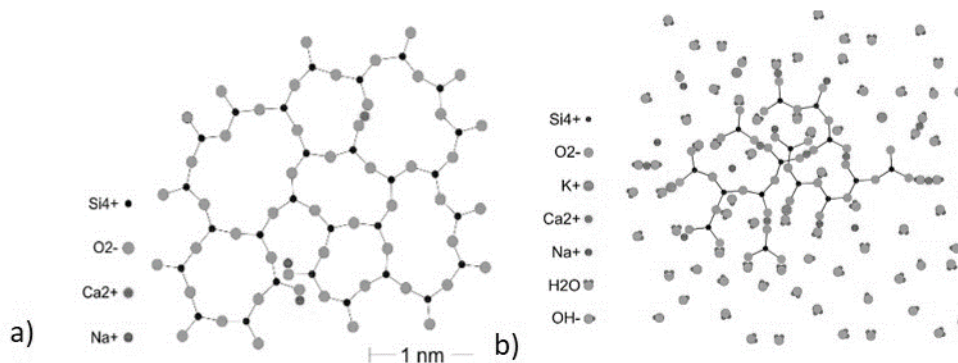


Figure 2-6: a) Amorphous solid SiO_2 , showing non-uniform rings and the contribution of alkalis and Ca in forming non-bridging oxygens atoms; b) alkali-silica (ASR) gel composed of clustering of colloidal silica particles surrounded by the gel's pore solution; the silica structure is highly degraded with lots of non-bridging oxygens atoms (Varshneya, 1993; Buckley and Greenblatt, 1994)

Figure 2-6b illustrates the aqueous silica species that are negatively charged, which link with metal ions (e.g., Na, K) to form poly-metal silicates. Once a nucleus reaches a critical size, it becomes nano-colloidal through further condensation. ASR produces non-swelling C-N(K)-S-

H and swelling N(K)-S-H gels (Helmuth and Stark, 1992). Distress occurs only when both gel types are formed.

2.4.1 Internal and external factors relating to the reactivity and expansion of ASR in concrete

i. Concentration and reactivity of the reactants

ASR will continue to advance if the required constituents and conditions in sections 0, 0 and 2.3.3 are present in the concrete. However, as the reaction proceeds, an increasing proportion of OH⁻ ions are consumed in forming ASR gel, forcing the reaction to slow down gradually. Thus, when the availability of one of the constituents is exhausted, the formation of new ASR gel is halted. Despite the reduction in the formation of new ASR gel, existing ASR gel in concrete still poses a potential risk when exposed to external sources of moisture, as stated by Vivian (1950).

ii. Chemical gradient within concrete

Alkali and hydroxyl ions migrate toward reactive silica sites to initiate ASR. As ions become fixed in the gel, the concentration gradient decreases at each reaction site. The rate of ion migration is contingent on the concentration of alkali hydroxide in the pore solution, the permeability of the reactive silica and surrounding cement matrix, water availability, and system temperature.

Additionally, reactive silica particles with an open or porous structure facilitate easy access for the pore solution to the mineral interior, enabling chemical attack and silica dissolution. The reaction occurs rapidly if the entire aggregate is permeable to the pore solution through microcracks or along grain boundaries. Conversely, if aggregate grain particles are “tightly” packed, only the reactive silica at the aggregate margins is susceptible to dissolution.

iii. Temperature of reaction

Numerous studies confirm that elevated temperatures accelerate the ASR reaction (Yang *et al.*, 2024). Standard test procedures like ASTM C1260 and ASTM C1293 involve storing mortar bars or concrete prisms at higher temperatures. This accelerated temperature evaluation allows for monitoring expansion over months, providing insights into the reaction that typically takes years to manifest in concrete structures. However, caution is necessary as increased

temperature speeds up the reaction and ASR gel formation and may induce modifications. The heightened temperature can reduce the gel's viscosity, leading to faster drying and dehydration. The increased temperature may also alter the gel's composition, potentially changing its swelling characteristics (Kawabata *et al.*, 2019).

2.5 Mechanism of ASR expansion leading to stress development and cracking

In a study by Leemann *et al.* (2024), an image taken of concrete, with the addition of caesium, made it possible to identify the first ASR products in the pre-cracking state of the concrete using a scanning electron microscope (SEM), as seen in Figure 2-7.

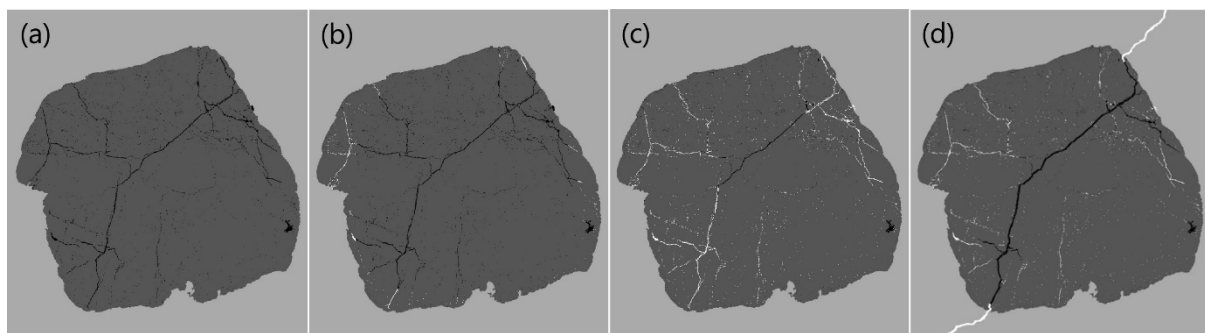


Figure 2-7: Illustrative sequence of ASR with an aggregate particle (dark grey) embedded in the cement paste (light grey) of a concrete: (a) aggregate particle with pre-existing micro-cracks, (b) precipitation of ASR products (white) starting close to the interface with the cement paste, (c) continuous precipitation towards the interior of the aggregate particle and (d) cracking of aggregate and adjacent cement paste with the diffusion of alkali-silica solution into the cement paste and its precipitation along the crack (Leemann *et al.*, 2024)

ASR products are formed initially in the voids present within the aggregates located at a distance of a few micrometres to tens of micro-meters from the interface with the cement paste (as shown in Figure 2-7a & b). As the reaction progresses, the ASR products continue to precipitate towards the interior of the aggregate (as shown in Figure 2-7c). This progression of ASR product formation within the aggregates is associated with the generation of pressure, which eventually leads to cracking (as shown in Figure 2-7d). Concrete deterioration becomes more noticeable once the cement paste starts cracking, and deterioration proceeds by increasing the width and density of cracks. The width of such cracks caused by ASR ranges from a few micrometres up to dozens of micrometres (Fournier and Bérubé, 2000; Katayama, 2012a; Leemann *et al.*, 2016; Shakoorioskooie *et al.*, 2021, 2022). Wider cracks typically propagate into the cement paste and provide a pathway for the highly concentrated alkali-silica solution present in the aggregates to diffuse into the cement paste (Wang *et al.*, 2023). Finally, tensile stresses can cause severe damage and expansion of concrete, which may lead to structural failures, including the rupture of rebar, distortion, and loss of structural integrity.

It has been reported that the measured expansive pressure for a synthetic gel exceeded 10 MPa (Struble and Diamond, 1981). The expansive pressures measured for mortars confined in a particular metal container ranged from 2.5 to 14 MPa (Pike, 1958). Expansive pressures of 2.5 and 4 MPa were also obtained by constraining the expansion of concretes with reinforcing steel bars (Lenzner and Ludwig, 1981). In the study by (Ferraris, Brown and Prabhakar, 1995), 9 and 2 MPa values were obtained for two mortar mixes containing aggregates with high and low reactivity levels, respectively. Wide variations of measured expansive pressures may be derived not only from differences in the measuring method and the degree of restraint to the free expansion of specimens but also from differences in the amount and composition of ASR gels produced in mortars and concrete.

Larger aggregate tends to retain higher concentrations of OH and alkali ions inside the aggregate. Hence, the formation of expansive alkali-rich ASR gel initiates inside the coarse aggregate rather than on the surface where calcium from the cement paste dominates. This explains the severe cracking typical in coarse aggregate and the earlier termination of the reaction in fine aggregate. Once ASR gel migrates from the reacted aggregate and reaches the cement paste, it takes up calcium and loses alkali thus its composition approaches the one of CSH gel and thus loses its potential for expansion, gradually leading to termination of the ASR (Katayama, 2008).

Although there has been considerable advancement in understanding ASR products and their impact on concrete, the mechanisms responsible for generating stress are still not fully understood. Currently, pressure generation and expansion are attributed to two distinct processes, as explained below.

- The most often postulated process is water uptake by the ASR products after their formation, resulting in swelling. It has been proposed that the mechanism for water uptake is osmosis. Moreover, it was concluded from various tests restricting or allowing water uptake by concrete specimens expanding due to ASR that neither water imbibition by the ASR products nor osmosis can be the primary cause for expansion. (Struble and Diamond, 1981; Chatterji, Jensen and Thaulow, 1986; Moranville-Resocurd, 1988; Bazant and Steffens, 2000b; Kirkpatrick *et al.*, 2005).
- The second proposed process is the pressure generation by ASR products directly during their formation within the aggregates. Expansion could be attributed to

crystallisation pressure caused by the formation of ASR products within water containing small pores and narrow cracks.

Although the mechanism responsible for the generation of stress is not fully understood, it can be seen that the development of ASR initiates in the aggregate particles. Hence, this was considered when determining the region of interest to investigate the maximum strain development in concrete subject to ASR deterioration.

2.6 ASR gel reaction products

Previous research has indicated that the composition of ASR gels varies over time (Knudsen and Thaulow, 1975a; Hou, Struble and Kirkpatrick, 2004; Kawamura and Iwahori, 2004). The composition of the ASR gel and the crystalline products can vary depending on the cement's alkali content, the dominant alkali species, and the mineral composition of the reactive aggregate.

After the initial stages of the reaction, the Ca/Si-ratio and the (Na + K)/Si-ratio of ASR products are typically in the range of 0.25 (Thaulow, Jakobsen and Clark, 1996; Thomas, 2001; Fernandes, 2009; Katayama, 2012b; Leemann, Shi and Lindgård, 2020). On the other hand, the Ca/Si-ratio of the initially formed products at the front of this inward progressing precipitation can be as low as 0.05 (Leemann and Münch, 2019; Leemann, 2022). Calcium is a major component in cement hydrates; its solubility in the alkaline pore solution is relatively low due to the high pH values (Lothenbach and Winnefeld, 2006). However, calcium is a governing factor for the precipitation of ASR products. Calcium leads to supersaturation and precipitation when added to an alkali-silica solution.

Table 2-5 shows the variation of the oxide composition of ASR gel taken from various structures subject to ASR deterioration. The variation is significantly large, implying no characteristic ASR gel composition. The gel's chemical composition determines its rheological properties, such as yield stress and viscosity, which subsequently govern the extent of internal stress development in concrete. ASR gel with low yield stress and low viscosity expands easily, filling cavities (Kawamura and Iwahori, 2004). Conversely, gel with high yield stress and viscosity, similar to C-S-H gel, exerts pressure on voids, leading to tensile stress development in concrete (Wang *et al.*, 2023). Lastly, calcium increases gel viscosity and yield strength, amplifying stress magnitudes during gel expansion (Struble and Diamond, 1981).

Table 2-5: Composition of alkali-silica gels from various concrete structures

Oxides						Reference
SiO ₂	Na ₂ O	K ₂ O	CaO	MgO	H ₂ O	
53.9	12.9	5.2	2.9	0.6-10.0	-	Stanton (1940)
61.7	14.9	0.5	0.6	0	-	Idorn (1967)
52.2	4.3	14.2	26	0.1	-	Poole A.B (1975)
48.6	0.9	6.8	27.4	-	-	Hanson et al (2003)
47.8	1.1	2-8	36.2	-	-	Hanson et al (2003)
56-86	0.4-20	9-18	1-28	0-10	10-30	Berube and Fournier (1986)
66-77	8-13	0-2.6	1-18	-	-	Davis and Oberholster (1986)
36-72	0-3.6	0-14	25-57	-	-	Mladenovic et al (2009)
27-75	0-18	-	0-35	-	-	Hanson et al (2003)

2.7 Structural effects of ASR on RC structures

To the author's knowledge, no cases have been reported where ASR has been the primary cause of structural failure. However, it is vital to recognise the structural consequences of ASR deterioration, which have, in some cases, led to structures being deemed structurally unfit for their purpose. The occurrence of ASR can alter concrete's durability, serviceability, and mechanical properties due to its changing internal structure and resultant cracking (Bach, Thorsen and Nielsen, 1993; Kapitan, 2006).

Cracking occurs when the surface tensile strain in concrete reaches 200-400 microstrains from the sum of all effects. This includes structural, thermal (early age and long-term) strains, shrinkage, and creep effects. The magnitude of these effects, in all three directions, needs to be quantified to differentiate the "free expansion" that develops in a structure.

ASR is primarily known to modify the durability properties of concrete due to the swelling of ASR gel exceeding the tensile strength of the concrete, leading to the development of cracks. In some cases, the visual outward appearance of map cracking can be viewed as aesthetically unacceptable. However, of greater concern is that cracks can facilitate the ingress of aggressive species such as water, chlorides, and carbon dioxide, leading to steel reinforcement corrosion.

ASR can also lead to the deterioration of concrete properties. Reductions of up to 80% for direct tensile strength, 60% for the elasticity modulus and 60% for compressive and splitting strength have been reported ((Institution of Structural Engineers, 1992; CSA, 2000; Siemes, Han and Visser, 2002; Fournier *et al.*, 2010). Structural and flexural members such as beams and slabs are of the most concern. The development of cracks can lead to members deflecting and under serviceability conditions characterised as having failed. Furthermore, the

development of cracks can also change the design distribution of loading in the member. The effects on compression members are minimal, as compressive loading tends to confine expansion due to ASR.

Many case studies have shown that structures affected by ASR damage can still meet and exceed their design requirements and design life. This has been achieved through correct diagnosis, successful repairs, and preventive maintenance throughout and beyond the structure's design life (Blight and Alexander, 2011).

2.8 ASR in South Africa

In South Africa, ASR was first identified in the 1970s on structures in the Cape Peninsula (now Western Cape) and extensively investigated by Oberholster and Brandt (1974). ASR deterioration is most prevalent in the Western Cape, owing to the common use of Greywacke rocks from the Malmesbury Group as coarse aggregate. Affected structures include various buildings, bridges, culverts, dams and a hydroelectric power station. Figure 2-8 illustrates a weir on a hillside stream in the Western Cape, subject to severe ASR deterioration (Courtesy of Y.Ballim). The presence of weeping ASR gel and map-cracking indicate signs of distress owing to ASR deterioration.



Figure 2-8: a) ASR gel and severe cracks present on the sidewall of the outlet works b) Severe cracks present on the weir wall (Courtesy of Y.Ballim)

In Gauteng during the 1960s and 1970s, an abundant source of aggregates (Witwatersrand quartzite and shales) were cheaply obtained after being mined at depth and dumped at the surface around the gold mines. After crushing, the rocks were extensively used to construct a significant portion of the reinforced concrete road and bridge structures (Brink, 1979). Davis. D. EM, McLver J. R and Wollhuter C. W (1978), at the time, stated that Witwatersrand quartzite provided sound engineering properties for use in concrete, as seen in Table 2-6.

Table 2-6: Concrete properties of quartzite of the Central Rand Group (Davis. D. EM, McLver J. R and Wolhuter C. W, 1978)

Specific gravity	Loose bulk density of coarse aggregate (kg/m ³)	Loose bulk density of fine aggregate (kg/m ³)	Mortar shrinkage	Coefficient of linear expansion of concrete (per °C)
2.71	1350	1470	0.055	12.2 x 10 ⁻⁶

Unaware of the reactivity of these aggregates, ASR developed over time, affecting structures including reservoirs, bridges, and sections of the De Villiers Graaff motorway in Johannesburg. Figure 2-9 illustrates the severity of crack-width openings owing to ASR deterioration on the M1 Empire Road Bridge (part of the De Villiers Graaff motorway).



Figure 2-9: Strain measurement of crack width openings taken periodically to monitor the progress of ASR deterioration on the East side, South Abutment, of the M1 Empire Road Bridge. (Courtesy of Y. Ballim)

The “double-decker” stretch of the De Villiers Graaff motorway (designed in 1963) is the most prominent structure known to be affected by ASR. In particular, the portal frame structures showed signs of ASR deterioration. Selected portal frames were subjected to full-scale load testing in 1982 and again in 1988, where deflections, rotations and strain measurements were recorded and compared to predicted quantities. Overall, there was a general agreement between in-situ measurements taken on the structure and measured concrete properties (from extracted concrete cores tested in the laboratory), which were consistently lower than predicted.

Although the frames' structural response was adequate, the knee joint portion of specific portal frames was rehabilitated based on the visual outward appearance (new cracks had appeared, and existing cracks had continued to widen). The rehabilitation involved the demolition of the damaged length of the upper beam (forming part of the knee joint) and the reconstruction of the element using reinforced concrete with aggregates that were not susceptible to ASR. The

repaired portal frames were inspected in 2003; it was found that the cracking had changed negligibly since the replacement and that the repair was in excellent condition (Blight and Alexander, 2011).

In 2019, another case involved the repair of a portion of the Selby Bridge forming part of the M2 Highway in Johannesburg, which had severely deteriorated. The bridge exhibited excessive vibrations due to the structural integrity of supporting elements being compromised due to ASR deterioration. Figure 2-10 is an image of a piece of concrete that broke off from a core extracted from the column head beneath the bridge. The “white rims” outlining the aggregate are evidence of ASR gel and confirm the ASR diagnosis. The affected section of the highway had to be closed for eight months to allow for the emergency repair of the bridge, which cost R58 million (Strydom, da Silva and Kongolo, 2022).

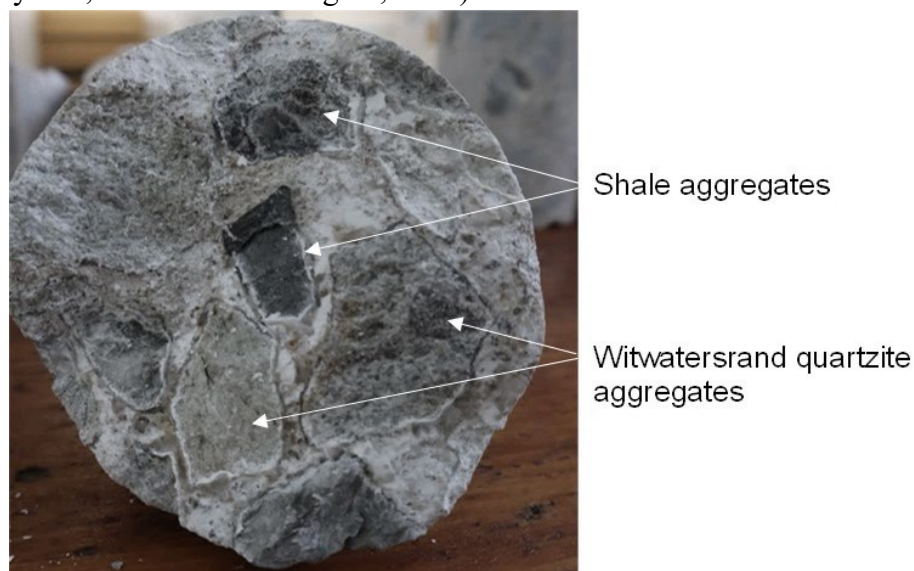


Figure 2-10: “White rims” surrounding aggregates confirm the development of ASR in concrete

The Johannesburg Road Agency (responsible for managing these assets) recognised that other aspects of the Highway would also be subject to ASR deterioration. However, due to the safety of road commuters being at risk and time and budget constraints, only the most critical sections of the highway were repaired.

In both field structures previously discussed, the “double-decker” stretch of the De Villiers Graaff motorway and the portion of the Selby Bridge forming part of the M2 highway, both RC structures are subject to ongoing ASR deterioration. To better understand the structure's condition and likely behaviour of the concrete, emphasis was placed on measuring the effects of ASR over the bulk concrete. This was achieved through full-scale load testing with respect to measuring deflection, rotation and strain development in particular regions. Additionally,

concrete cores were extracted and taken to the laboratory for testing to determine the effect of ASR on concrete properties. However, there was limited insight into strain development on a micro level, owing to ASR deterioration and its subsequent manifestation over the bulk material.

Lastly, in addition to several case studies that have been investigated in Gauteng and the Western Cape Province, it is now recognised that ASR can also occur in the Eastern Cape Province, the Free State Province, Kwa-Zulu Natal as well as Mpumalanga Province (Alexander and Blight, 2017).

2.8.1 Reactive aggregates in South Africa

In South Africa, specific aggregate sources (both coarse and fine) have been identified through service records and laboratory testing as potentially alkali reactive, as outlined in Figure 2-11. Case studies in South Africa indicate that ASR primarily occurs within the aggregate. This can be attributed to planes of weakness generated during the crushing process, facilitating alkali access to reactive silica sites.

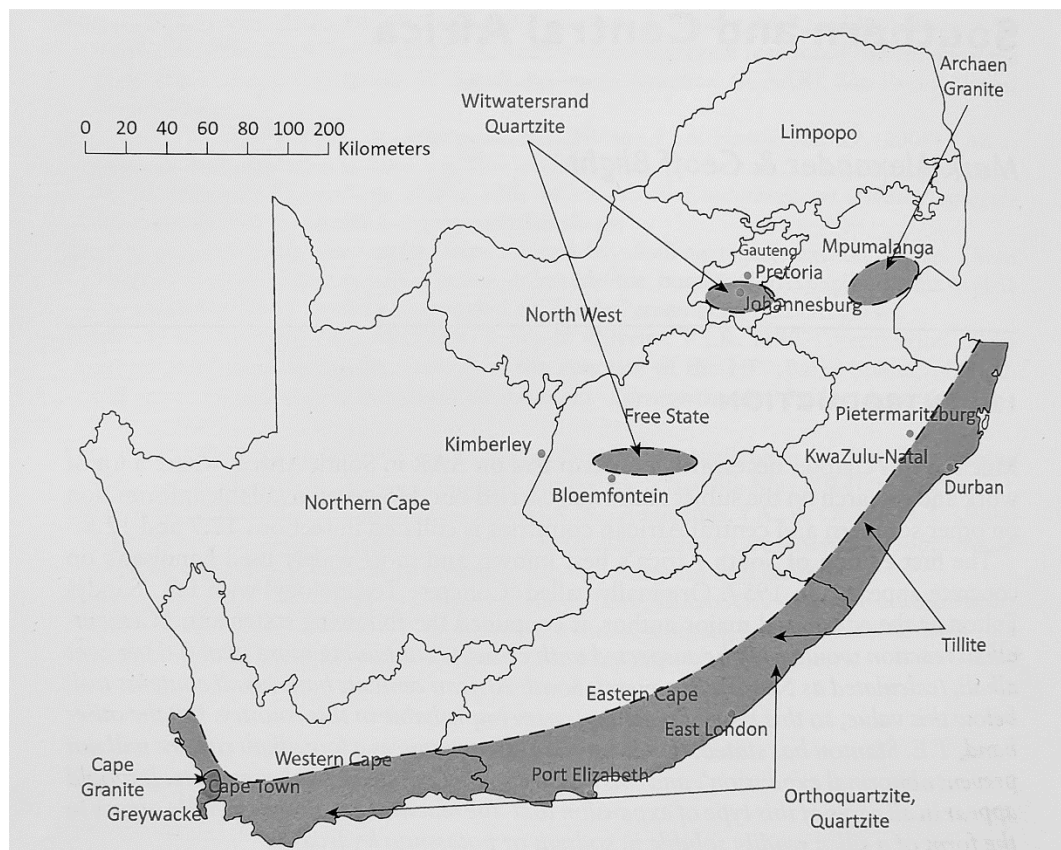


Figure 2-11: Sources of potentially alkali reactive aggregates in South Africa (Alexander and Blight, 2017)

Aggregate susceptibility tests conducted on Witwatersrand quartzite from the M1 "double-decker" stretch of the De Villiers Graaff motorway in 1985 concluded that the aggregate falls into either the "potentially deleterious" or "deleterious" area of the diagram shown in Figure 2-12. This information validates the data provided in Figure 2-11.

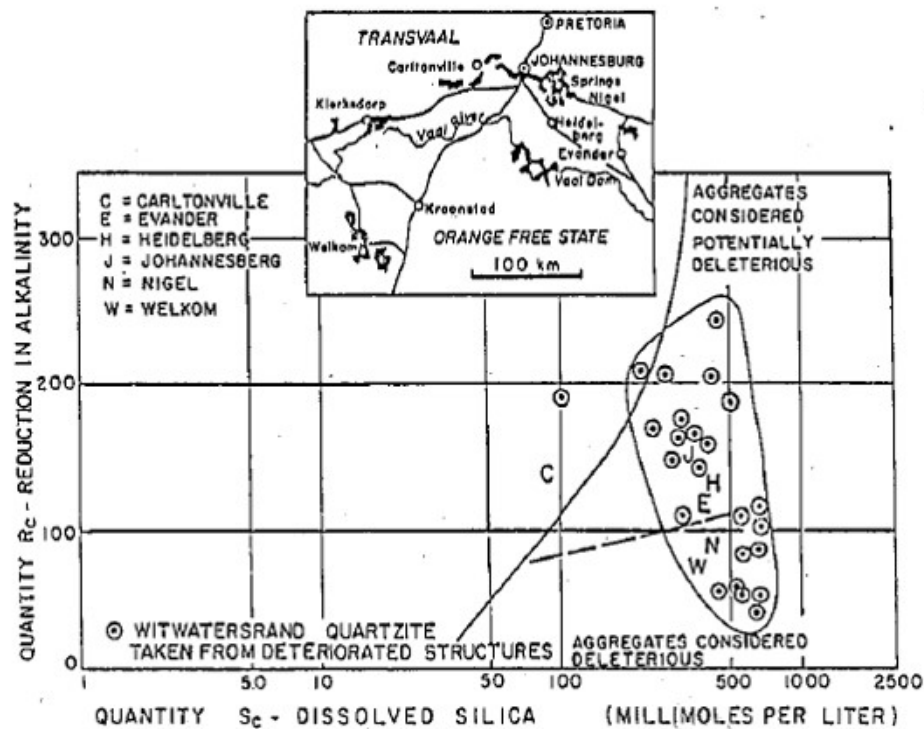


Figure 2-12: Deleteriousness of Witwatersrand quartzite as indicated by ASTM C289 (Blight and Alexander, 1985)

Table 2-7 was developed as a recommended guideline for total alkali levels to mitigate ASR based on the varying alkali reactivity observed in South African aggregates.

Table 2-7: Recommended alkali limits for South African aggregates from different lithostratigraphic areas (Alexander and Blight, 2017).

Rock type	Total Na ₂ O _e (kg/m ³)
Witwatersrand Supergroup quartzite, shale	2.0
Malmesbury Group metasediments	2.1
Dolomite Group chert	2.8
Table Mountain Group orthoquartzite	2.8
Bokkeveld Group arkose	2.8
Natal Group quartzite, sandstone	2.8
Dwyka Formation tillite	2.8
Enon Formation quartzite pebbles	2.8
Quaternary quartzite gravels	2.8
Cape granite	4.0
Archean granite	Not determined
Salem granite	Not determined

It can be seen that Witswatersrand Quartzite has the lowest alkali limit threshold, followed by Malmesbury group metasediments from the Western Cape compared to the rest of the South African aggregates. The range of alkali limits varies from 2.0 Na₂O_e (kg/m³) to 4.0 Na₂O_e (kg/m³), which is a substantially large range. Thus, it is imperative to identify the specific rock type for a given project to minimise the development of ASR in the concrete.

2.9 Management of RC field structures subject to ASR deterioration

Owners and managers must regularly inspect and maintain buildings, bridges, and other infrastructure assets to ensure their safety, reliability, and longevity. The challenges in infrastructure asset management centre on the need for advanced knowledge regarding the condition and performance of these assets. Additionally, reliable data is crucial for planning to ensure that the assets continue to meet expected performance levels well into the future, as highlighted in the asset management framework (AMF) shown in Figure 2-13.

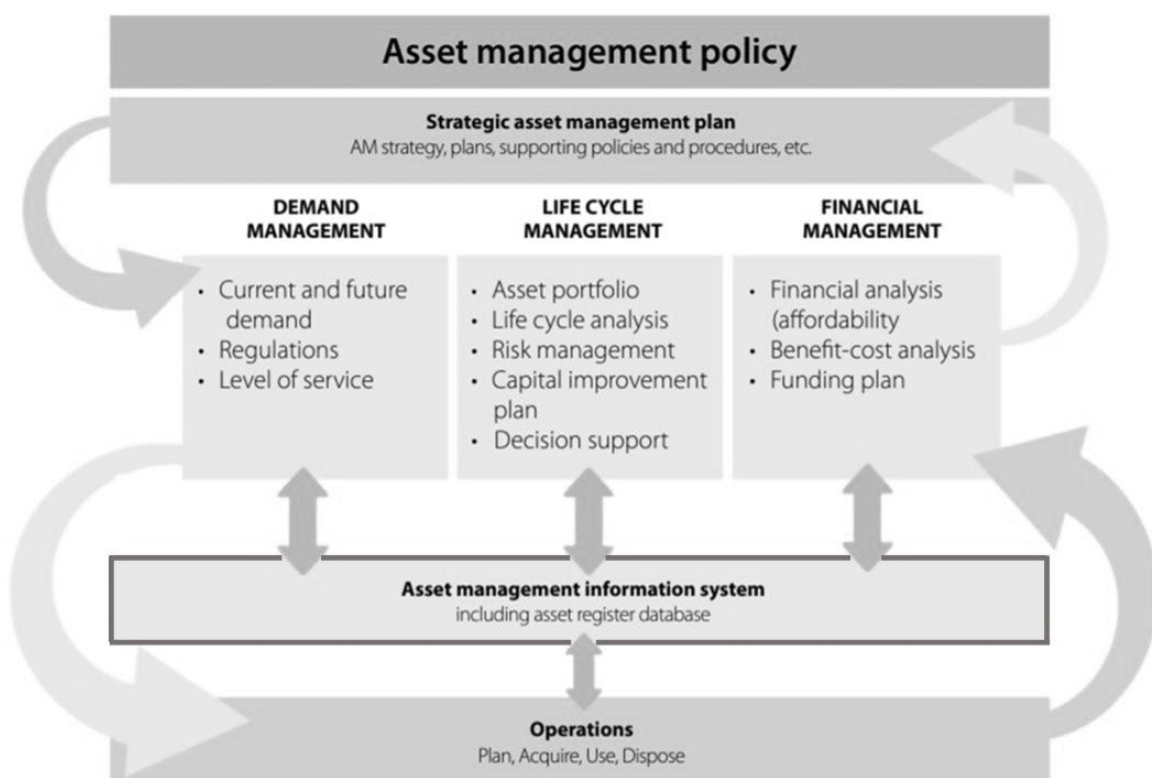


Figure 2-13: Asset management framework (United Nations, 2021)

Ageing infrastructure faces the impact of changing environmental conditions and increased operational loading, leading structures to deteriorate and potentially fail before reaching their intended design lifespan. Recognising the vital role infrastructure plays in sustaining a country's economy and standard of living, ongoing assessment management of infrastructure

is essential to preserve the structural integrity of these assets. This requires a continuous cycle of inspection, monitoring, and repair to safeguard against potential damage and ensure the sustained functionality of the infrastructure. This is particularly crucial in an environment like Johannesburg, where vital RC concrete structures such as bridges and roads are ageing, subject to increasing loads, climate variation, and susceptibility to ASR deterioration.

2.10 Procedures used to diagnose and confirm ASR in RC structures

Correctly identifying and determining the underlying causes of the distress in RC structures is critical to infrastructure safety and long-term operation. Understanding and diagnosing damage to concrete affected by ASR poses challenges due to the similarity in external characteristics with those caused by other factors such as frost attack, drying shrinkage, or sulphate attack. Complicating the diagnostic process is the diverse range of aggregate types, each with its reactivity and properties, along with various environmental factors influencing the reaction outcomes. This complexity makes the diagnosis of ASR damage inherently not straightforward. Sections 2.10.1 to 2.12 present a comprehensive overview of a structured approach to diagnosing and evaluating structures suspected of being subject to ASR damage.

2.10.1 Review of existing documents

Before conducting any scheduled inspection, thoroughly reviewing pertinent documentation related to the structure is essential. This includes but is not limited to, structural design information, construction procedures, previous inspection reports, and any reports detailing remedial work that has been undertaken. Such a review is crucial for establishing the necessary context and expectations before the on-site inspection.

2.10.2 Visual inspection: On-site inspection

Conducting a physical visit to the structure is typically the initial step in assessing and diagnosing potential ASR deterioration. The primary objectives of the on-site visual inspection are to:

- Establish the condition and safety level of the structure;
- Formulate hypotheses on the cause or progress of the deterioration mechanism;
- Locate representative areas for sampling.
- Extract/obtain core samples for further laboratory testing

The inspection strategy should be tailored to the specific type of structure under investigation, guided by a thorough review of existing records. During the visual inspection, each structural element should be examined separately, with a meticulous recording of observations related to the type, extent (severity), and location of defects.

Since ASR is an expansive reaction that takes time to manifest, visible signs of ASR damage typically become apparent after the first five years of a structure's lifetime. Routine visual inspections are essential for noting the first symptoms of ASR damage, which may manifest in various forms on the concrete external surface, as depicted in Figure 2-14. The most commonly reported manifestations include (Godart, DeRooij and Wood, 2013):



Figure 2-14: a) Elements of a bridge deck displaying distress in the form of severe map-cracking b) Wall displaying distress in the form of severe map-cracking (Blight and Alexander, 2011)

- Development of extensive cracking, often in the pattern of “map-cracking”, where concrete is unrestrained by adjacent structures or by internal reinforcement.
- Cracking tends to develop longitudinally parallel to the direction of the dominant restraint, as the transverse cracking is suppressed where compression stresses develop from restraint or loading (e.g. axially loaded columns).
- Cracks filled with ASR reaction product.

Table 2-8 provides a classification system to assess the probability of a structure being subject to ASR presence based on routine inspection results, categorised as low, medium, or high. Features such as cracking patterns, deformations, surface discolouration, exudations, pop-outs with ASR gel deposits, and environmental conditions contribute to this assessment.

Table 2-8: Modified classification system for assessing the probability of the presence of ASR based on routine inspection results (Godart, DeRooij and Wood, 2013).

Feature	Probability of ASR		
	Low	Medium	High
Cracking and crack pattern	None	Some cracking patterns typical of ASR (e.g. map cracking or cracks aligned with primary reinforcement or stress)	Extensive map-cracking or cracking aligned with primary reinforcement or stress; misalignment of facings(differential expansion on each side of the crack)
Deformation and displacement of element	None	Some evidence (e.g. closure of joints, spalls, misalignment between structural members)	fair to extensive signs of volume increase leading to spalling at joints, displacement and misalignment of structural members
Surface discoloration	None	slight surface discoloration associated with some cracks	Many cracks with dark discolouration and adjacent zones of light-coloured concrete
Exudations	None	White exudations around some cracks; the possibility of colourless jelly-like exudations	colourless, jell-like exudations readily identifiable as ASR gel associated with several cracks
Pop-outs with ASR gel deposit	None	Some	Many cracks with dark discolouration and adjacent zones of light-coloured concrete
Environment	Dry and sheltered	Outdoors exposure but sheltered from wetting	Parts of components are frequently exposed to moisture, e.g. rain, groundwater water, and water due to the function of the structure (hydraulic dam)

ASR is one potential mechanism contributing to concrete damage, and it is not advisable to attribute the damage solely to ASR based on visual observation alone. Concrete samples are required for further laboratory examination and testing to confirm the presence of ASR. Due to the heterogeneity of concrete within a large structure, collecting samples from slightly damaged and damaged concrete areas is advisable. Drilling core samples (>150 mm) with a diameter of at least three times the maximum size of the coarse aggregate is preferable. Larger core sizes prove beneficial when conducting residual expansion tests at elevated temperatures and help reduce the leaching effect of alkalis.

2.10.3 Visual inspection: Macro-inspection of cores

Visual and low-magnification inspections of concrete samples extracted from the suspected structure can offer valuable insights into the presence and severity of ASR damage. Macroscopic examinations of the core are typically conducted with the naked eye, aided by a magnifying lens (7x to 10x). Observations should include the size and distribution of

aggregates, voids, compaction, and reinforcement conditions. Several macroscopic features play an essential role in ASR diagnosis and are to be noted during the examination:

- Damp patches on the concrete surface;
- Reactive/ weathering rims (white outer/dark inner rims) around aggregate particles;
- Crack patterns of cement paste and internal cracking of aggregates;
- Presence of any ASR-gel
- The pattern/distribution of the macroscopic feature over the core (through the entire core or only locally)
- Any other signs of irregularities or concrete disintegration

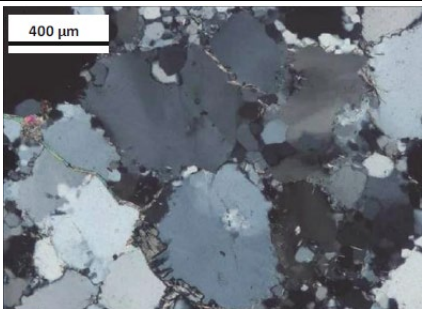
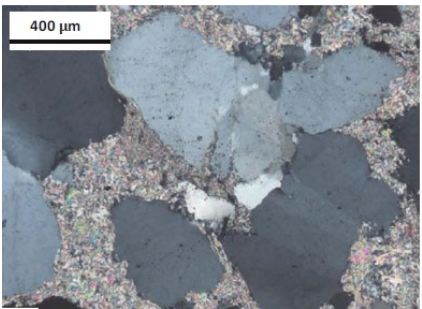
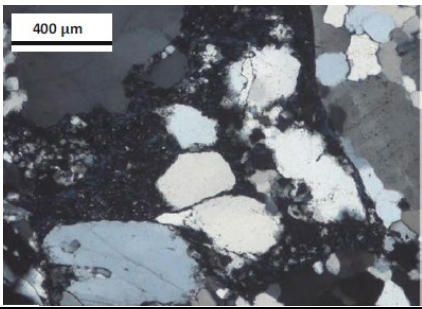
In addition to ASR symptoms, visual inspections may uncover additional features such as crack penetration to the reinforcement and delamination of cover concrete. These findings contribute to understanding the concrete's deterioration status and can help formulate an appropriate repair strategy. Furthermore, the examination identifies areas of interest for subsequent investigations, such as polarising or scanning electron microscopy. This systematic approach enhances the comprehensiveness of the diagnostic process for concrete structures subject to ASR deterioration.

2.10.4 Detailed inspection: Petrographic examination

Petrographic examinations involves analysing thin-section slides, typically around 25-30 μm thick, prepared from concrete samples from field structures. This method is necessary for positively identifying most diagnostic features of ASR. Under cross-polarised light, various aggregate and concrete microstructure components can be adequately characterised.

These examinations enable the identification of the geological nature of aggregate particles, particularly those that have undergone reactions, started to crack, and produced ASR gel. The technique also aids in determining potentially reactive minerals within the damaged concrete and understanding the nature of the cement. In the book "*Petrographic Atlas: Characterization of Aggregates Regarding Potential Reactivity to Alkalis*," three examples of reactive Quartzite found in South Africa are presented in Table 2-9.

Table 2-9: Petrographic analysis of three types of quartzite found in South Africa (Fernandes and Sims, 2016)

Image	Description	Reactivity potential attributed to:
	Coarse-grained, impure quartzite composed of quartz, kyanite, muscovite, sericite, chloritoid, zircon, monazite and opaque minerals arranged in a non-directional heterogranular texture.	Microcrystalline quartz along the boundaries of the larger clasts, strained quartz with sutured boundaries, and domains of quartz mosaics due to recrystallisation.
	Impure quartzite composed of medium to coarse-grained quartz, chlorite and lithic fragments (quartzite). The fine-grained matrix consists of sericite, muscovite and microcrystalline quartz	Microcrystalline quartz in the coarse grains and Subgrains of quartz in the metamorphic rock fragments.
	Coarse-grained, impure quartzite mainly composed of quartz, chlorite, opaque minerals, lithic fragments (quartzite) and chloritoid.	Microcrystalline quartz in the intergranular spaces. Quartz crystals with sutured boundaries and bulging.

The atlas provides examples of various rock types known to be reactive globally. It includes images of rock types, descriptions of geological features and aspects that make the rock type reactive. In the case of the quartzite presented in Table 2-9, all three examples contain impure quartzite composed of microcrystalline quartz, attributing to the rock type's reactivity.

In a case study by Sibbick and West (1992), the severely cracked Padiham Bypass in Northwest England was investigated. Using petrography and explanatory drawings shown in Figure 2-15, they identified vital diagnostic features such as fine cracks radiating from the aggregate particle into the cement matrix and gel in the crack veins. This comprehensive analysis effectively demonstrated that ASR was the underlying deterioration mechanism responsible for the severe cracking exhibited in the pavement.

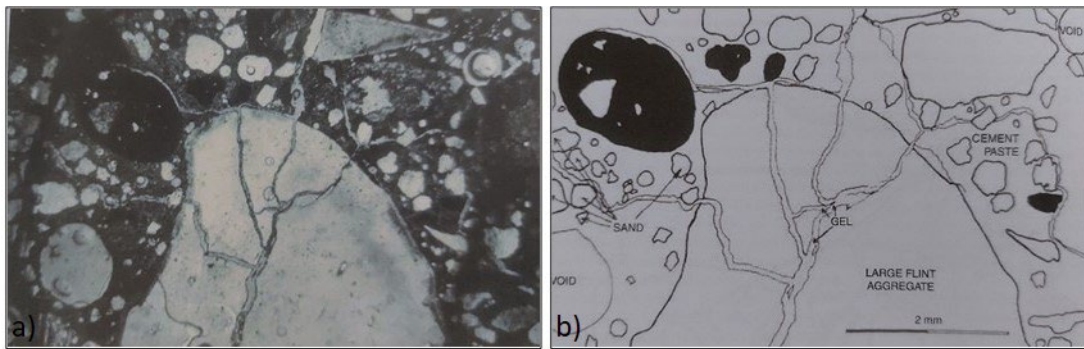


Figure 2-15: a) A photomicrograph and b) a drawing showing a large flint aggregate exhibiting a well-developed branching system of micro cracks with associated gel (Sibbick and West, 1992)

2.10.5 Detailed inspection: Scanning electron microscopy

Visual and petrographic examinations are valuable techniques; however, they do not provide detailed information on the microstructural and morphological phases in concrete or the chemical composition of these phases. To address this limitation, scanning electron microscopy (SEM) examinations and energy-dispersive X-ray (EDX) spectroscopy of concrete phases offer a powerful technique, providing semi-quantitative chemical compositions of the phases of interest (Merz and Leemann, 2013; Shin, Struble and Kirkpatrick, 2015; Leemann, 2017).

SEM offers significantly higher magnification in the range of 1000x to 15,000x, enabling the detection of small amounts of ASR gel, which is not visible with transmitted optical microscopy. The technique produces images that provide insight into the morphology of the ASR gel, as shown in Figure 2-16. The images reveal varying forms of ASR gel, including amorphous, foil-like, and rosette-type structures. EDX provides chemical element data corresponding to the SEM images.

In addition to SEM and EDX, other techniques, such as electron-probe micro-analyser (EMPA) and micro X-ray fluorescence (μ XRF), have been employed for elemental analysis in investigating ASR-induced deterioration in concrete. Both EMPA and μ XRF allow for elemental mapping across a designated area and spot analysis, facilitating the determination of the chemical composition at specific points of interest.

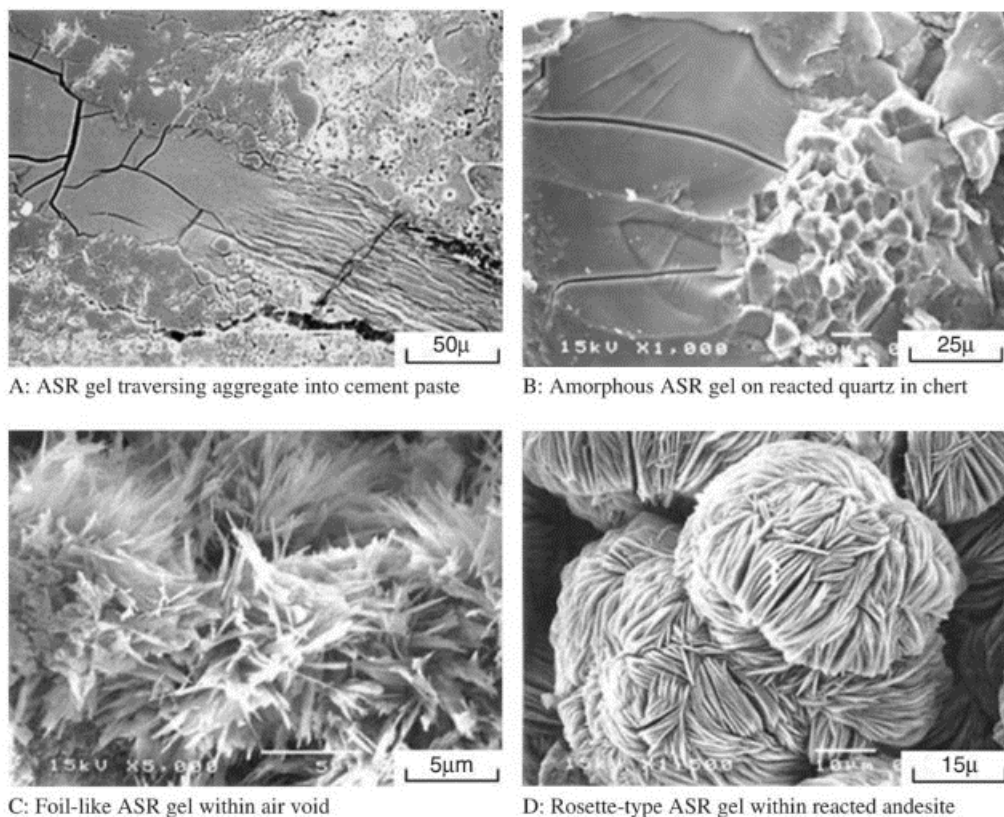


Figure 2-16: SEM images (a, b, c & d) showing various morphology of ARS gel in concrete (Godart and De Rooij, 2017))

2.11 Procedures used to assess the severity of ASR deterioration in RC structures

After confirming the presence of ASR in concrete through the procedure outlined in Section 2.10, the subsequent step involves analysing the severity of the deterioration. Accurate assessments of the condition of deteriorating structures subject to ASR are essential to ensure safety, schedule the repair/replacement of structures and manage the costs associated with public infrastructure (fib Bulletin, 2003). The procedure outlined in Sections 2.11.1 to 2.11.5 aims to determine the extent of ASR deterioration and the rate at which it may escalate over time.

2.11.1 Visual inspection: On-site monitoring of field structures

The initial approach to assessing the severity of ASR involves examining the crack patterns on the surface of the concrete structure. Various methods outlined in the literature, such as "Expansion to date" (Institution of Structural Engineers, 1992), the "LCPC (Laboratoire Central des Ponts et Chaussées)-cracking index" (LCPC, 1997), and the "cracking index method" (Fournier *et al.*, 2010), share a similar underlying principle. These methods summarise the widths of cracks within a measured grid. The crack widths are linked to the severity of the

ASR deterioration, as illustrated in Figure 2-17. Depending on the process rate, measurements should be repeated every 1 to 5 years to establish a useful monitoring scheme.

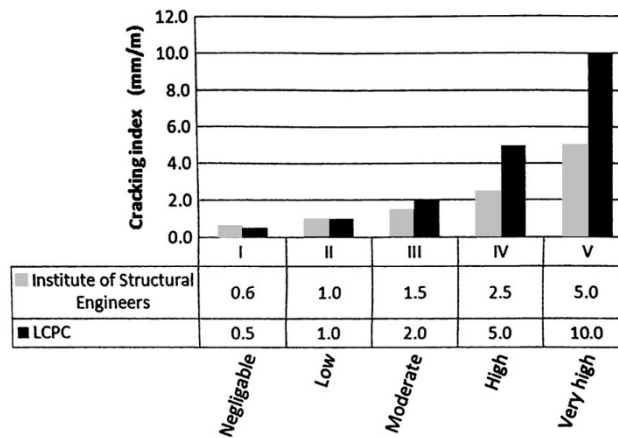


Figure 2-17: Comparison between the Cracking Index developed by the Institution of Structural Engineers and LCPC (Godart and De Rooij, 2017)

An empirical relationship was established by the Council for Scientific and Industrial Research (CSIR) in Pretoria as an approach to approximate the level of ASR expansion from the mean ASR-induced crack width (W_c) in Equation 2-2 (Oberholster, 2009). It is noted that it is an approximation to be used with care, and crack widths used for the approximation should exclude effects of mechanical action, imposed drying shrinkage and thermal expansion. A method to accurately establish the actual ASR expansion in a structure has yet to be developed.

$$\varepsilon_{ASR\%} = 0.312W_c + 0.00236 \quad \text{Equation 2-2}$$

Additionally, the global deformation of structures can be monitored by installing measurement points on the concrete surface. Ideally positioned on the same members used for map cracking, these points supplement information obtained from the same testing area. Humidity and environmental probes provide valuable insights, aiding in interpreting expansion and crack measurements, as prevailing environmental conditions influence both aspects (Olajide, Nokken and Sanchez, 2023).

2.11.2 Laboratory inspection: Determination of Damage Rating Index (DRI)

The DRI is a semi-quantitative petrographic approach increasingly utilised worldwide (Rivard et al., 2002; Bérubé et al., 2005; Rivard and Ballivy, 2005) for assessing damage in concrete sections affected by ASR. Initially proposed by (Grattan-Bellew and Mitchell, 2006), the DRI method was developed to evaluate damage by coarse reactive aggregate particles.

The technique involves the analysis of microscopic damage features associated with ASR, using a stereomicroscope with 16x magnification within a 1 cm² grid drawn on the polished concrete surface (Villeneuve and Fournier, 2012; Sanchez et al., 2016). Each microscopic characteristic has a set weighting factor relative to the distress mechanism, as given in Table 2-10. Ideally, a surface of at least 200 cm² should be used for DRI analysis, with the final DRI value normalised to an area of 100 cm² (Rivard and Ballivy, 2005). Generally, a higher DRI number corresponds to a more significant deterioration of the affected concrete.

Table 2-10: Latest weighing factors for petrographic features noted in the DRI analysis (Villeneuve, 2011)

Petrographic features	Abbreviation	Weighing factor
Cracks in coarse aggregate	CCA	0.25
Opened cracks in coarse aggregates	OCA	2
Crack with reaction product in coarse aggregate	OCAG	2
Coarse aggregate debonded	CAD	3
Disaggregate/corroded aggregate particle	DAP	2
Cracks in cement paste	CCP	3
Cracks with reaction product in cement paste	CCPG	3

Figure 2-18 shows an example of a 1 cm square grid analysis highlighting various petrographic features on the surface of a polished concrete section.

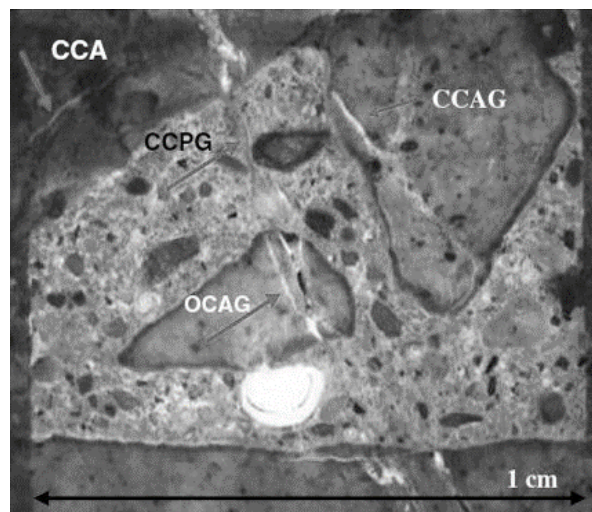


Figure 2-18: Petrographic features to be noted in the Damage Rating Index (DRI) analysis as listed in Table 2-10 (Fournier, Sanchez and Beauchemin, 2015)

The primary goal of the DRI is not to "replace" conventional petrographic procedures of concrete, such as ASTM C 856; instead, the DRI is a complementary petrographic tool aiming to quantify the "damage degree" between different elements of a structure or as a function of time within a specific concrete member. It is crucial to note that, despite being used by several

researchers, there is currently no standard test procedure for DRI. Additionally, no standardised classification exists to separate low, moderate, or high damage levels in the DRI, making the values relative and not absolute measurements. The weighting factors and features considered may be modified for specific cases and deterioration mechanisms, given the wide variation in the DRIs of concrete containing different aggregates.

In a case study conducted in April 2009, six bridge structures along Interstate 395 near Bangor, Maine, in the United States, were inspected for symptoms of ASR-related distress. The bridges displayed varying degrees of ASR symptoms from map cracking in abutments, wing walls, and columns; some preferred alignment (vertical) of cracks was observed in columns. The extent of damage ranged from mild to severe within a given structure, depending on the nature of the exposure. Concrete that was directly exposed to rainfall exhibited very severe cracking in some cases, whereas the damage observed on parts of the same structure that were protected from rainfall by the bridge deck showed considerably less damage as shown in Figure 2-19 (a, b, c and d).

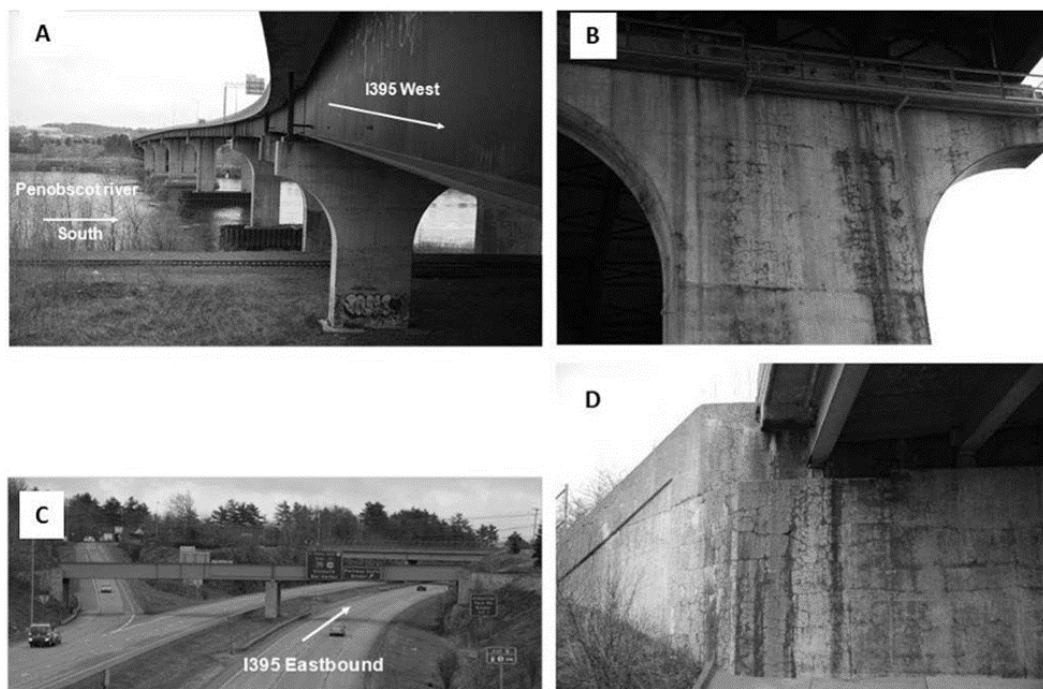


Figure 2-19: I-395 and 5th Parkway bridges. (A): Bridge carrying I-395 over the Penobscot River showing (B) increased cracking on the exposed part of the pier. (C): Bridge carrying 5th Parkway over I-395 showing (D) increased cracking on the wing wall and the exposed part of the abutment. (Thomas et al., 2013)

The presence of ASR was confirmed, and it was revealed that the concrete in all six bridges contained reactive greywacke/argillite in the coarse aggregate. DRI values ranged from 133 to 882, indicating a low to severe degree of ASR damage. Figure 2-20 illustrates the results of the petrographic analysis with regard to DRI values for concrete analysed from three areas with

varied exposure to moisture. It must be noted that different petrographic features abbreviations were used compared to the ones provided in Table 2-10.

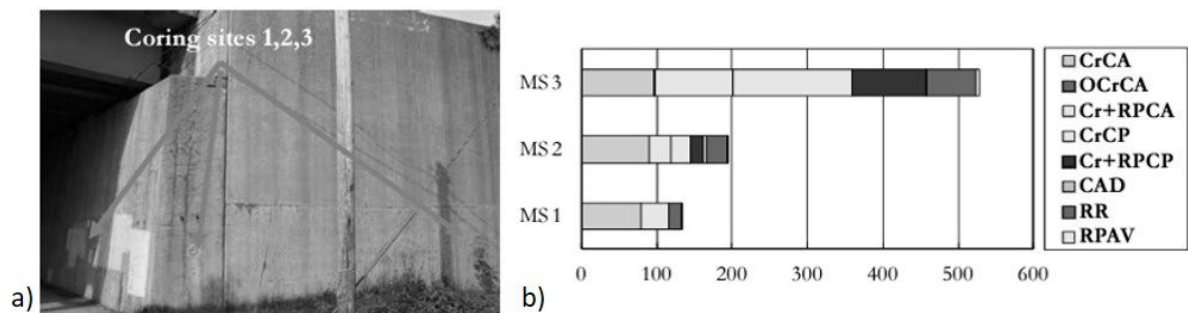


Figure 2-20: a) Coring site b) Associated DRI values obtained for concrete extracted from various coring site locations (Thomas et al., 2013)

In general, it can be seen that the results showed higher DRI values, indicative of higher damage for exposed parts of the concrete structure. This is attributed to the availability of moisture to facilitate ASR deterioration in the concrete. The same trend was confirmed by various other studies, where by higher DRI values correspond with higher damage and low DRIs reflect comparatively little damage (Macdonald, Piersanti and Magnan, 2019; Sanchez, Drimalas and Fournier, 2020; Ahmed, Zahedi and Sanchez, 2022)

2.11.3 Laboratory testing: Determination of alkali metal inventory of concrete affected by ASR

The residual alkali content is important in determining the reserve of alkali hydroxides available in concrete for further reaction and expansion. Conventional methods involve extracting concrete pore solutions using techniques such as expression or dissolution. The amount of soluble alkali in concrete depends on the extraction method, whether diluted acid-soluble or water-soluble, hot or cold extraction, etc. A brief overview of these methods is presented below:

- i. **Pore Water Expression (PWE) Method:** This method employs high-pressure equipment to compress the hardened cementitious material sample, resulting in the expression of the pore solution, as illustrated in Figure 2-22. PWE is considered the reference method for comparison with other techniques (Kagimoto, Sato and Kawamura, 2000; Bérubé and Fournier, 2004; Cyr et al., 2008; Pu Q et al., 2012).

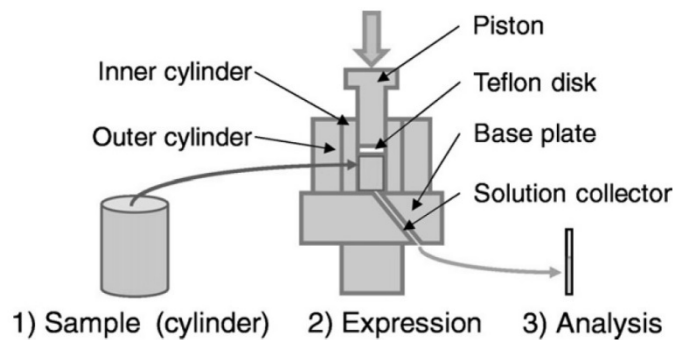


Figure 2-21: Schematic of pore water expression (PWE) test set-up (Cyr et al., 2008)

- ii. **In Situ Leaching (ISL) Method:** This method involves drilling small cavities in a water-saturated sample surface, as shown in Figure 2-22, adding a known amount of deionised water, and analysing the solution after equilibration. Equilibrium is reached when the pH of the solution inside the cavity is constant, which depends on the sample and takes one or two weeks (Sagüés, Moreno and Andrade, 1997; Li, Sagüés and Poor, 1999; Cáseres et al., 2006).

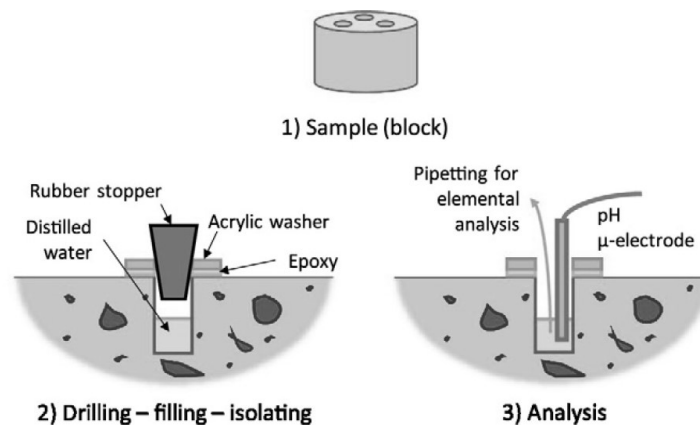


Figure 2-22: Schematic of the in situ leaching (ISL) arrangement (Sagüés, Moreno and Andrade, 1997)

- iii. **Ex Situ Leaching (ESL) Methods:** All ESL methods involve grinding concrete to a powder, leaching with deionised water, separating the liquid, and analysing the extracted solution, as summarised in Figure 2-23. Three types of ESL methods are discussed below:

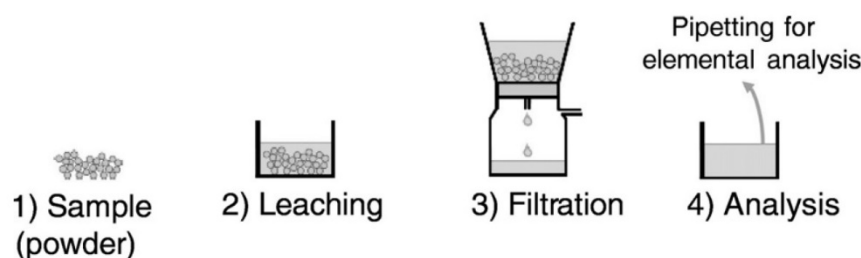


Figure 2-23: Schematic of the typical four steps used in ex-situ leaching (ESL) methods (*Plusquellec et al. 2017*)

- **Cold Water Extraction (CWE) Method:** The sample is pulverised to obtain a particle size below 80 μm . A 20 g sample is mixed with 20 ml of deionised water, stirred, and immediately filtered (Bérubé M, Duchesne J and Rivest M, 1996).
- **Hot Water Extraction (HWE) Method:** The sample is pulverised to obtain a particle size below 160 μm . A 10 g sample is mixed with 100 ml of deionised water, boiled for 10 min, and left to stand overnight ((Bérubé M.-A *et al.*, 2002).
- **Espresso Method:** Recently modified, this method involves placing a 10 g sample in a Büchner filter, adding 300 ml of deionised boiling water, and conducting vacuum filtration (Fournier, Sanchez and Beauchemin, 2015).

In a review by Plusquellec et al. (2017), various methods for determining pH and free alkali metal content of concrete pore solutions were considered, and comparable results were noted among the techniques outlined in Table 2-11. Despite slight variations, the ESL methods yielded similar results. Thus, the authors recommended cold water extraction as the easiest and fastest ESL method.

Table 2-11: Summary of methods for determining the free alkali metal content in concrete (*Plusquellec et al. 2017*)

Method	Specimen	Extraction time	Main advantages	Main drawbacks
Pore water expression	Cylinder (no pieces)	0.5–4 h	- No dilution effect	Not applicable on relatively dry and dense samples Possible pressure dependence of the results
In situ leaching	Piece of concrete	1–2 months	Spatial distribution possible	Not applicable on non-saturated samples Potential alkali leaching during water saturation Potential carbonation (long equilibration time)
Cold water extraction	Powder < 80 μm	5 min	- Applicable on relatively dry samples	Dissolution of solids (low)
Hot water extraction	Powder < 80 μm	≈ 12 h	- No water saturation is required	Dissolution of solids (medium)
Espresso	Powder < 160 μm	5 min		Dissolution of solids (high)

Plusquellec et al., (2018) investigated concrete's free alkali metal content from an ASR-affected dam. Na^+ , K^+ , and Ca^{2+} profiles were obtained using CWE, as shown in Figure 2-24. W, WA, A and S refer to concrete taken from various locations of the concrete dam. The reference surface refers to the exterior surface of the core (dam wall).

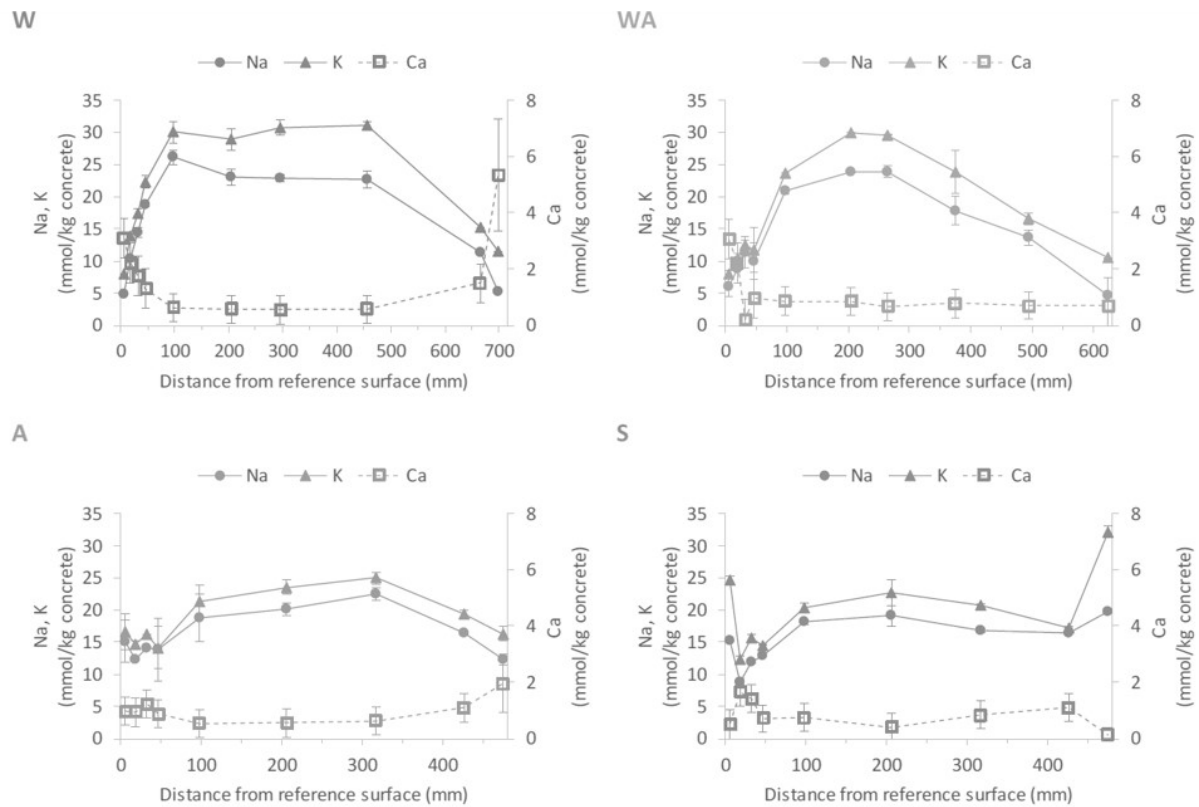


Figure 2-24: Na^+ , K^+ , and Ca^{2+} profiles (mmol/kg of concrete) obtained after CWE on cores W, WA, A, and S. The data are not corrected for potential alkali metal release from the aggregates. (Plusquellec et al., 2018)

The study revealed alkali metal leaching on external surfaces, with an observed decrease in the free alkali content between the bulk and surface of the cores. Kagimoto, Sato and Kawamura (2000) observed a similar trend in alkali metal leaching to a depth of approximately 100 mm on a 14-year-old concrete structure. There have, however, been very few studies on alkali metal leaching in actual structures (Grattan-Bellew and Danay, 1992; Kagimoto, Sato and Kawamura, 2000; Bérubé M.-A *et al.*, 2002). The reason is attributed to the difficulty of extracting pore water from old concrete by expression under pressure.

For all four profiles, the Na^+ and K^+ content is not the same value; however, the profiles tend to follow the same trend over the length of the concrete cores. Furthermore, it can be seen that as the calcium content increases, the alkali content decreases and vice versa. This result is attributed to the concept of cation exchange with the calcium replacing the alkali in the initially

alkali-rich reaction products. This exchange occurs as a portion of the ASR gel migrates away from the aggregate particle and comes into contact with the calcium-rich cement paste (Hansen, 1944; Knudsen and Thaulow, 1975b).

Furthermore, using the measured and calculated pore solution values, a simulated pore solution composition for a specific concrete can be established, which can be used in potential residual expansion tests. Figure 2-25 shows the alkali metal release, based on the CWE measurements, of an ASR-affected dam. The calculated alkali metal content of the cement is 1.28%, and an assumed proportion of free alkali metals $k_{\text{free}} = 50\%$.

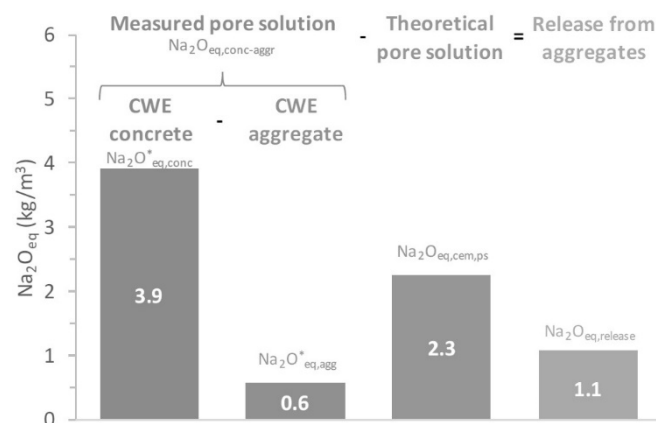


Figure 2-25: Calculation of the alkali metal release based on the CWE measurements, considering $\%\text{Na}_2\text{O}_{\text{eq}} = 1.28\%$ and $k_{\text{free}} = 50\%$ (Plusquellec et al., 2018)

2.11.4 Laboratory testing: Determination of mechanical properties of concrete affected by ASR

Understanding the impact of ASR deterioration on the mechanical properties of affected concrete is crucial while assessing the condition of ageing infrastructure.

i. Compressive strength

ASR-induced cracks are initially formed within the aggregate particles and only extend to the cement paste at later stages. Joo and Takahashi, (2024), found that distributed microcracks had a limited effect on reducing compressive strength, as stress could still be effectively transmitted across these cracks. Moreover, when microcracking was the dominant damage pattern, the mechanical contribution of the ASR gel increased. This was due to the solidified gel more readily filling the microcracks, thereby aiding in the partial recovery of ASR-induced damage.

In contrast, wider macrocracks significantly reduced mechanical properties, as they allowed damage to propagate more easily and were less likely to be filled by the gel.

Subsequently, ASR development causes a progressive compressive strength loss of affected concrete (Sanchez *et al.*, 2017). The Institution of Structural Engineers (1992), CSA (2000b) and Siemens *et al.* (2002) have reported reductions of up to 60% in the compressive strength of concrete affected by ASR. On the contrary, several researchers have found that the strength of reinforced elements is not significantly reduced by ASR deterioration (Chana and Korobokis, 1992; Bach, Thorsen and Nielsen, 1993; Ahmed, Burley and Rigden, 1998; Fan and Hanson, 1998; Monette, Gardner and Grattan-Bellow, 2000). This was particularly true for large-scale laboratory specimens and field structures (Blight *et al.*, 1983; Imai *et al.*, 1983; Boenig, 2000; Deschenes, 2009; Larson, 2010), where the reinforcement cage restrains the concrete expansions, effectively “prestressing” and confining the deteriorated concrete to maintain the structural integrity. Furthermore, the compressive strength can vary according to the development of the microcracking planes, as microcracking develops anisotropically under constraints (Godart, de Rooij and Wood, 2013). The reduction in compressive strength can lead to the transfer of load to steel, subsequently leading to the yielding of the material.

ii. Elastic modulus

The stiffness of the aggregate particles was found to influence the reduction of the modulus of elasticity in various ASR-affected concrete. Institution of Structural Engineers (1992) reported reductions up to 60% for elasticity modulus. The stiffness damage test has been shown to provide a more sensitive measure of the development of mechanical damage from microcracking that leads to the degradation of stiffness and strength (Chrisp, Waldron and Wood, 1993).

The Stiffness Damage Test (SDT) is a mechanical and cyclic test procedure used to assess the condition of concrete affected by internal swelling reaction (ISR) mechanisms such as ASR (Sanchez *et al.*, 2015). The SDT was initially developed by (Walsh, 1965), who observed a good correlation between the crack density and the cycles of loading/unloading (i.e., stress/strain relationship) of rock specimens. In 1987, Crouch (1987) adopted this procedure for concrete specimens. The test procedure, which is based on five compression cycles of concrete specimens (cylinders or cores), was initially performed using fixed loads of 5.5 MPa (Crisp, Wood and Norris, 1989; Chrisp, Waldron and Wood, 1993) or 10 MPa (Smaoui *et al.*,

2004), at the loading rate of 0.10 MPa/s. Selected outcomes of the test included the modulus of elasticity, dissipated energy (hysteresis area), plastic deformation (plastic strain) and the non-linearity index (NLI; the ratio of secant modulus at half of the maximum load and the secant modulus at the peak load) of the first or over the four last cycles (Sanchez *et al.*, 2016).

Sanchez *et al.*, (2015) proposed that the SDT should be performed using a percentage of the compressive strength of the concrete under analysis instead of a fixed load, as illustrated in Figure 2-26. The proposed percentage is 40% of the compressive strength. The specimen is conditioned in water before conducting the test. Three outputs are calculated using the stress-strain curves from the stiffness damage test: hysteresis area, modulus of elasticity and plastic deformation.

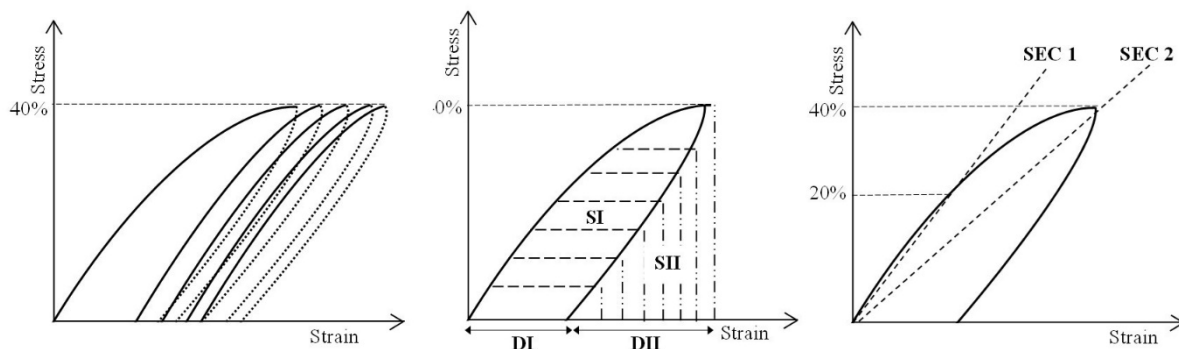


Figure 2-26: Calculation of the stiffness damage index (SDI), Plastic Deformation Index (PDI) and Non-Linearity Index (NLI) from a typical stress-strain graph

The hysteresis area measures the energy to close the macro and microcracks disseminated throughout the concrete specimens. The hysteresis area values for the first cycle are typically about 2 to 3 times larger than the average values obtained over the last four cycles. The plastic deformation measures the permanent strain the ASR-affected concrete specimens undergoes when subject to compressive load. Despite being considered a non-destructive test, ASR-affected concrete will not fully recover to its original condition upon unloading, as the crack surfaces are rough, and some sliding across those surfaces occurs during the test, leading to plastic deformation. This parameter is linked to the hysteresis area and the crack's closure process.

Furthermore, the authors suggested using indices as parameters of the test procedure, namely the Stiffness Damage Index (SDI) and Plastic Deformation Index (PDI). The SDI and PDI represent the dissipated energy to total energy ratio and plastic deformation to the total deformation in the system (Sanchez *et al.*, 2016; Zhu Y *et al.*, 2021). Figure 2-26 illustrates

how $SDI = SI/(SI + SII)$ and $PDI = DI/(DI + DII)$ are determined over the five cycles, where SI is the irreversible deformation energy, SII is the elastic deformation energy, and DI and DII represent the plastic deformation and elastic deformation, respectively, over the loading-unloading process). Moreover, the modulus of elasticity (average secant modulus of the 2nd and 3rd cycles) along with the NLI (Sec 1/Sec 2 in the first cycle and the average Sec 1/Sec 2 value of the last four cycles).

Furthermore, Figure 2-28 is a set of stress-strain results obtained by Sanchez et al. (2014), which demonstrated the decrease in stiffness as a function of increased expansion levels of concrete subject to ASR deterioration. Studies have shown that the higher the ASR-induced expansion level with respect to damage, the lower the modulus of elasticity and the higher the SDI and PDI indices (Monette, Gardner and Grattan-Bellow, 2000).

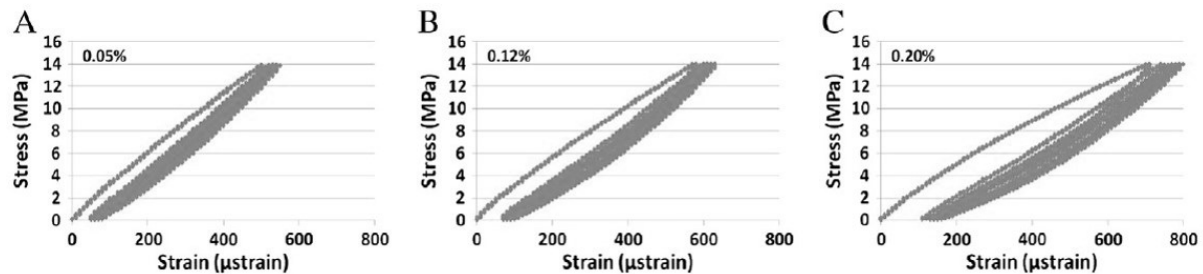


Figure 2-27: Typical stress-strain curves obtained for test specimens cast from 35 MPa concrete for various expansion levels (Sanchez et al., 2014)

Figure 2-28a illustrates a concave curve that produces an NLI parameter “greater than one”, this result has been correlated to a predominant cracking network perpendicular to the loading direction. On the other hand, a convex curve (Figure 2-28b) produces an NLI parameter “lower than one”, indicating a cracking network parallel to the loading direction (Chrisp, Wood and Norris, 1989; Chrisp, Waldron and Wood, 1993).

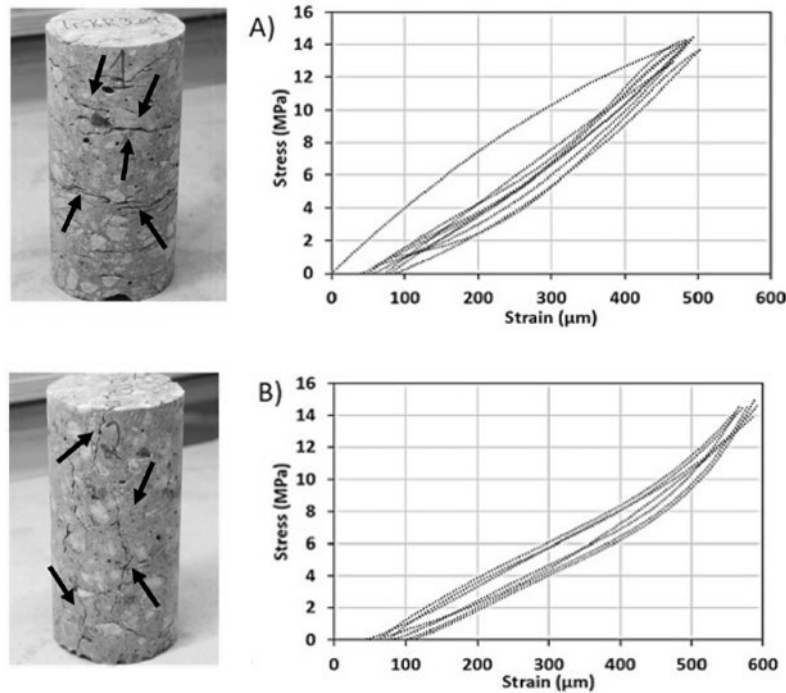


Figure 2-28: a) predominant crack orientation perpendicular to load resulting in a concave SDT graph b) predominant crack orientation parallel to load resulting in a convex SDT graph (*Sanchez et al., 2014*)

Thus, it can be seen that conducting a SDT provides more information on the extent of internal damage, predominant cracking direction, and modulus of elasticity owing to ASR deterioration than a standard modulus of elasticity test.

2.11.5 Laboratory testing: Potential residual expansion of concrete subject to ASR deterioration

In assessing ASR development, it is important to comprehend and measure the mechanics governing deterioration, such as the generation of swelling pressures due to ASR gel formation and subsequent strain development. For in-service structures affected by ASR, in-situ monitoring of actual structures typically takes 2-3 years to yield meaningful information, subject to mechanical, thermal, and climatic variations. However, evaluating strain development in extracted cores can determine the likely "residual expansion" of concrete. This measurement is a faster (6 months to a year) indicator of the potential continuation of mechanisms, leading to further deterioration. Furthermore, when combined with data about the temperature and moisture conditions of the structure, it may be used as input for modelling purposes.

i. Test methods used to determine residual expansion

Various test methods, such as LPC N°44 (LPC, 1997), the Swiss (Leemann, Merz and Cuchet, 2021), and the Quebec method (Fecteau and Fournier, 2014), have been developed to determine potential residual expansion. These methods incorporate parameters like elevated temperatures, a fixed relative humidity value and alkali availability to simulate an ideal environment conducive to ASR. The goal is to quantify the concrete's "potential" future expansion leading to the most unfavourable conditions.

Previous studies have depicted the potential residual expansion of field concrete in an S-type curve, as shown in Figure 2-29. This trend is divided into three phases:

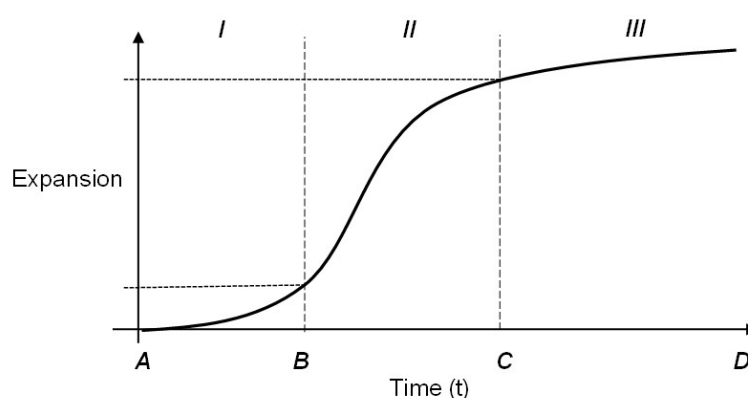


Figure 2-29: Schematic illustrating S-type residual expansion trend for field-tested concrete

This trend can be further broken down into three distinct phases:

- **Phase I** – From point A to B, there is minimal or zero expansion due to the initiation of the ASR mechanism and formation of new ASR gel. Existing ASR gel may exhibit initial expansion based on moisture uptake and the state of stress of the material in the structure.
- **Phase II** – Rapid expansion occurs from point B to C, attributed to the continued development of ASR gel and swelling of existing ASR gel.
- **Phase III** – A gradual slowing down of expansion occurs from point C to D, leading to a plateau. This is due to the exhaustion of constituents (either alkaline in the pore solution, reactive silica, or moisture) required for ASR.

Several parameters influence the behaviour of a structure affected by AAR, which has not been considered in the residual expansion test methods. Some of these parameters and limitations are listed below:

1. The field concrete is subject to a given stress state; this stress is relieved in the residual expansion test;
2. The environmental exposure of the field concrete is not matched to the highly triggered exposure of 38 C and 100% relative humidity of the residual expansion test;
3. The residual expansion test does not account for the effect of rebar.
4. The maximum expansion measured in the above test methods will likely be underestimated since the core suffers significant alkali leaching. Consequently, the expansion tends to level off after a specific time, not due to the consumption of relative silica or alkalis (Rogers and Hooton, 1993).

It is essential to note the influence of these parameters, especially when attempting to extrapolate laboratory results to predict the likely behaviours of field structures.

Research by Scrivener (2019) has shown that concrete immersed in fabricated pore solution, mimicking the original structural concrete, displays an increasing linear residual expansion trend, as shown in Figure 2-30. It can be seen that the expansion curves for concrete submerged in an alkali pore solution are sustainably greater than concrete submerged in a fixed NaOH solution or an environment with 100% RH, addressing concerns about alkali leaching effects in existing testing approaches. Similar expansion curves that did not end in a plateau were reported by Catherine Larive (1998), Shehata and Thomas (2000), and Carles-Gibergues and Cyr (2002).

Figure 2-30: Influence of test environment with respect to additional free alkalis when conducting residual expansion tests on concrete (Scrivener, 2019)

This approach provides a more realistic assessment of a material's likely future behaviour. It challenges the notion of ASR-induced residual expansion reaching a plateau after a certain period, as suggested by existing test methods.

2.12 Structural severity of structure subject to ASR deterioration

The Institution of Structural Engineers (2002), UK, developed an approach to produce structural severity ratings that can assist with managing and monitoring structures affected by ASR. The structural element severity rating incorporates three inputs, an expansion index I to V, based on expansion results (ranging from 0 % to >0.25 % expansion). This value would be determined by conducting a residual potential expansion test on the concrete extracted from the structure. The environment in which the element is situated is based on moisture availability

and RH%, which is dry, intermediate, and wet. Lastly, in the detailing reinforcement of given structural elements under consideration, Class 1 refers to an element that has a three-dimensional cage of very well-anchored reinforcement in contrast to Class 3, where a two-dimensional cage of reinforcement in one or two faces, no through ties, no links or low cover on the element.

It can be seen in Table 2-12 that structural elements located in wet environments with minimal restraint (Class 3) in the form of reinforcement are subject to the greatest risk of future expansion due to ASR deterioration, compared to the same element located in a dry environment. Such elements would require immediate attention to determine the extent of damage owing to ASR and subsequent remedial strategies to ensure the safety and reliability of the element.

Table 2-12: Structural element severity rating (Institution of Structural Engineers, 1992)

Site Enviro nment	Reinforc ement detailing class	Expansion Index									
		I		II		III		IV		V	
		$\varepsilon \text{ ASR}\% < 0.06\%$		$0.06\% \leq \varepsilon \text{ ASR}\% < 0.10\%$		$0.10\% \leq \varepsilon \text{ ASR}\% < 0.15\%$		$0.15\% \leq \varepsilon \text{ ASR}\% < 0.25\%$		$\varepsilon \text{ ASR}\% \geq 0.25\%$	
		Consequence of failure: Slight or Significant									
		Slight	Sign.	Sligh t	Sign.	Slight	Sign .	Slight	Sign.	Slight	Sign.
Dry	1	N	N	N	N	N	N	N	N	N	N
	2	N	N	N	N	N	N	N	N	N	Mi
	3	N	N	N	N	N	N	N	Mi	Mi	Mo
Interm ediate	1	N	N	N	Mi	Mi	Mo	Mi	Mo	Mi	Mo
	2	N	N	Mi	Mo	Mi	Mo	Mo	Mo	Mo	S
	3	N	Mi	Mi	S	Mo	S	S	VS	S	VS
Wet	1	Mi	Mi	Mi	Mo	Mi	Mo	Mo	S	Mo	S
	2	Mi	Mi	Mo	S	Mo	S	S	S	S	VS
	3	Mi	Mo	Mo	VS	S	VS	VS	VS	VS	VS

*Structural severity ratings: N = Negligible; Mi = Mild; Mo = Moderate;

S = Severe; VS = Very Severe

Using the expansion index, environment and reinforcement class in Table 2-12, the engineer can assign a severity rating to each element in a structure and determine the management procedure going forward. Negligible (N) implies that routine inspections must be continued at a regular frequency, in contrast to very severe (VS), where the element requires immediate action and specialised and detailed studies are needed.

2.13 Part II: Digital Image Correlation (DIC)

In recent years, full-field displacement and strain measurements have become a major focus in the development of experimental techniques in solid mechanics (Moulart *et al.*, 2007; Dijkstra and Broere, 2010). Several methods are proposed for full-field displacement and strain measurements, such as speckle interferometry, holographic interferometry, geometric moiré, speckle photography, and image correlation (Grédiac, 2004). All these techniques have been widely applied in experimental mechanics. However, it is not easy to determine the optimum one for a particular use; it is often a trade-off between strain measurement accuracy, efficiency and the cost of these techniques. DIC has proven to be suitable for solid mechanics as it has a relatively low cost, maintains high accuracy, and is accessible and straightforward to conduct (McCormick and Lord, 2010).

DIC is a non-contact experimental method involving digital image acquisition and processing to yield information about shape, motion and deformation over the region of interest (Sutton, Orteu and Schreier, 2009). The technique was developed in the 1980s at the University of South Carolina (Peters and Ranson, 1982). DIC involves using greyscale values in images of the undeformed and the deformed objects, which are then numerically correlated to obtain surface deformation estimates.

2.14 Basic concepts of DIC

Depending on the complexity of the specimen or the expected deformation pattern, DIC can be broadly divided into three categories: Two-dimensional (2D) DIC, Three-dimensional (3D) DIC and Digital Volume Correlation (DVC).

2D DIC, in particular, is employed when the subject under investigation is planar and the out-of-plane deformation is known or assumed to be negligible. This involves using a single camera placed perpendicular to the plane of the deforming body for image acquisition. The crux of DIC lies in registration techniques, where area- or feature-based algorithms are employed to track the points of the object during deformation formation. Area-based algorithms compare the intensity pattern similarities to achieve correspondence and are commonly used due to their simplicity, high accuracy and spatial resolution (Wang, Kieu and Le, 2014).

When performing DIC, artificial patterns such as dots, grids or random speckles are adhered to the surface of the deforming body. These patterns are then assumed to trace the deformation of the underlying specimen. An area of interest (AOI) is manually specified, as shown in Figure 2-31. The correspondence of a single pixel in the reference image cannot be made directly with the deformed image, as the exact grey-scale value can be located in numerous locations. The area is subsequently divided into sub-regions, which are then checked for correspondence, assuming the motion within this sub-region is constant. The sub-regions are known as subsets; typically, square subsets are employed in DIC.

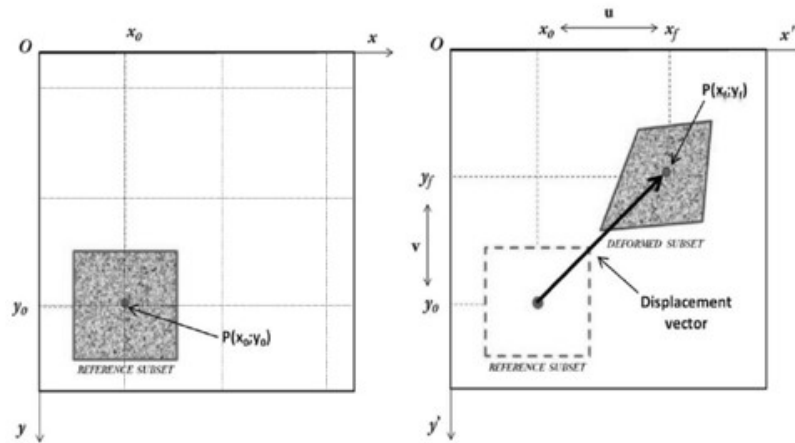


Figure 2-31: Schematic of a reference subset before deformation and the corresponding target subset after deformation

The displacements are computed at each point of the virtual grids to obtain full-field deformation. The imposed green square is the subset (a set of pixels) for tracking the movement of its centre point $P(x, y)$ from the reference image (before deformation) to deformed images $P'(x', y')$. The tracking of the subset is conducted using selected correlation functions such as cross-correlation (CC) or normalised cross-correlation (NCC) (Pan *et al.*, 2009). Subsequently, the strain fields are derived by smoothing and differentiating the displacement fields.

2.15 Application of DIC to obtain displacement and strain measurements in cementitious materials

DIC has been widely used to study the deformation of engineering materials, including cement-based materials and concrete. The following studies illustrate how the DIC technique has been employed to obtain full-field displacement and strain measurements in cementitious materials.

- **Zhang, 2014**

Zhang (2014) employed DIC as a strain measurement technique to improve the ATSM C1260 Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method). Recycled glass aggregates were used as the reactive aggregate source in the four mortar batch mixtures. For each respective batch, during the 14-day accelerated ASR test, digital images of the surface of the mortar bars were captured every 30 minutes. Figure 2-32 shows the test setup used. The images were then processed to determine the in-plane strain field, and the results were compared to the physical measurement of length change.



Figure 2-32: Experiment setup: 1-computer, 2-plexiglass container, 3-dial gauge, 4-reservoir, 5-immersion circulator, 6-tubing, 7-water drip bottle, 8-camera, and 9-lighting (Zhang, 2014)(Zhang, 2014)

The DIC technique results closely matched the real strain measurement using the dial gauge. Figure 2-33 illustrates the strains from the DIC method (MATLAB Strain) employed versus the true strain (Demec) readings. Specimen 1 and 2 reflect the day-by-day physical strain measurements of two specimens stored in the plexiglass container. The dial gauge represents the real strains of the specimen, which was used to capture images. The strain plots for the four curves exhibit similar trends over the 12-day duration.

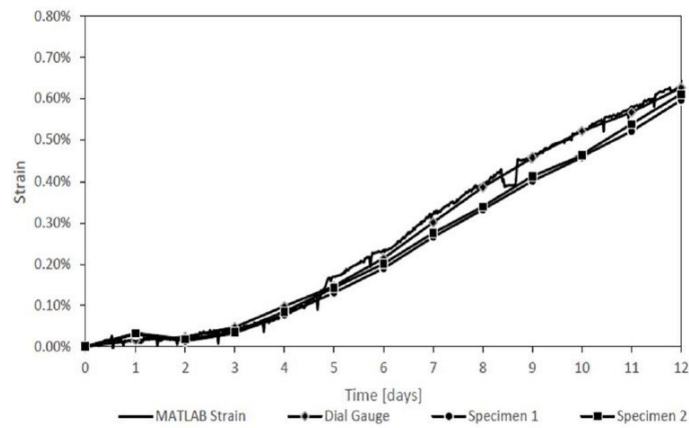


Figure 2-33:Vertical strain from MATLAB analysis and true strains of specimens in Batch 1 (Zhang, 2014)

The study concluded that DIC was an effective technique for analysing the surface strains of mortar subject to alkali silica reaction. The strain readings produced using DIC closely matched the real strain measurements of the dial gauge.

- **Chen *et al.* 2018**

Chen et al. (2018) used three-dimensional DIC to characterise the non-uniform full-field displacement and strains in concrete, focusing on the effect of aggregate size and volume on concrete shrinkage. In Figure 2-34, the images on the top illustrate the selected region of interest (ROI) on selected specimens containing one coarse aggregate particle and mortar. The images on the bottom show the maximum and minimum principal strain, respectively, determined using DIC.

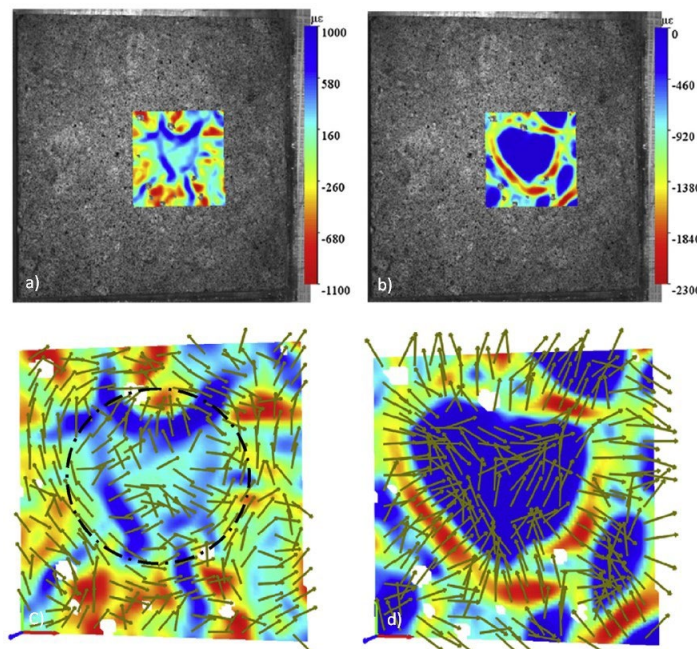


Figure 2-34:Directions of maximum and minimum principal strain for two respective regions of interest (Chen et al., 2018)

In Figure 2-34a, the blue area represents the mortar around the coarse aggregate. The analysis confirms that the expansive maximum principal strains are near the coarse aggregate, as illustrated by the black circle in Figure 2-34c. In Figure 2-34b, the blue area represents the coarse aggregate, and the other colours (red, green, yellow) represent the mortar. When the mortar shrinks, the coarse aggregate restrains it. Thus, the direction of the minimum principle strain in the mortar was perpendicular to the aggregate, as seen in Figure 2-34d.

- **Teramoto *et al.*, 2018**

Teramoto et al. (2018) used DIC to investigate the effects of ASR-induced macroscopic expansion and internal crack propagation in concrete specimens containing reactive aggregates.

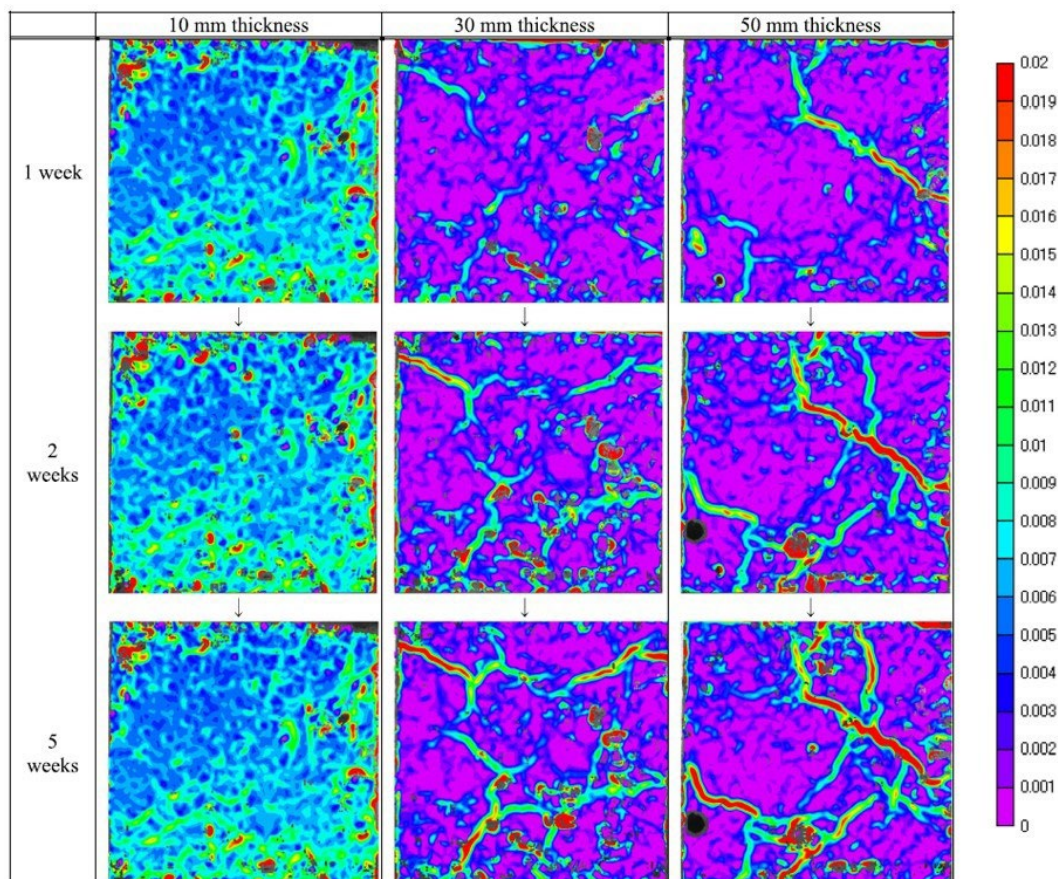


Figure 2-35 illustrates that DIC effectively detected ASR-induced micro-cracking in concrete.

Figure 2-35: Development of ASR strain in specimen with varying concrete thickness (Teramoto et al., 2018)

It can be seen that the use of 50 mm thick specimens was most effective at facilitating and capturing ASR-induced micro-cracking in concrete. Lastly, it can be seen that maximum strains

of 0.02% were observed in the crack veins, owing to ASR gel swelling pressure, after five weeks.

In all of the studies noted, DIC was successfully employed to obtain the necessary full-field displacement and strain measurements across various phases in cementitious materials. Each study required a specific setup, with respect to camera position, specimen placement, and region of interest, to employ DIC. Lastly, the results obtained from DIC correlated closely to traditional strain measurement techniques, such as using a Demac gauge.

2.16 Closure – Need for further research

Based on the critical review of the literature, it is evident that the issue of ASR deterioration is not limited to RC structures based in Johannesburg, South Africa. Given the crucial role of key infrastructure in society, closure or demolition is not a feasible solution. Therefore, the focus has shifted towards proactive maintenance and rehabilitation to prolong the service life of RC structures.

Accurate prediction of ASR-affected RC structure behaviour requires the extraction and analysis of samples from the actual infrastructure. Assessing the residual potential expansion of concrete offers valuable insights into the generation of swelling pressures caused by ASR gel formation and subsequent strain development. While current test methods effectively quantify overall bulk expansion, they fail to measure localised expansion and resultant cracking.

ASR is a localised mechanism primarily occurring at reactive silica sites with sufficient moisture. It leads to a non-uniform distribution of ASR gel production and swelling pressures within concrete. Concrete's inherent heterogeneity makes this complexity worse, resulting in varying strains across phases and interfaces, notably between aggregates and cement paste. Consequently, conventional bulk residual expansion measurements offer only a cumulative understanding, limiting insight into maximum strains at the aggregate-cement paste interface governing ASR mechanisms.

Exploring strain development at both micro and macro scales offers valuable insights into how localised deformations manifest across the bulk of concrete and their correlation with ASR-induced damage. This approach aimed to provide a more comprehensive understanding of ASR deterioration mechanisms and aid in devising effective mitigation strategies.

Chapter 3 Field investigation and experimental methodology

3.1 Introduction

This chapter presents details of the field investigation and laboratory-based experiments carried out in this study to achieve the Aims as stated in Chapter 1. The experimental programme was designed in three sequential phases, an overview of which is provided in Figure 3-1. This approach was adopted as the findings gained at the end of each phase of the study informed subsequent steps in the research.

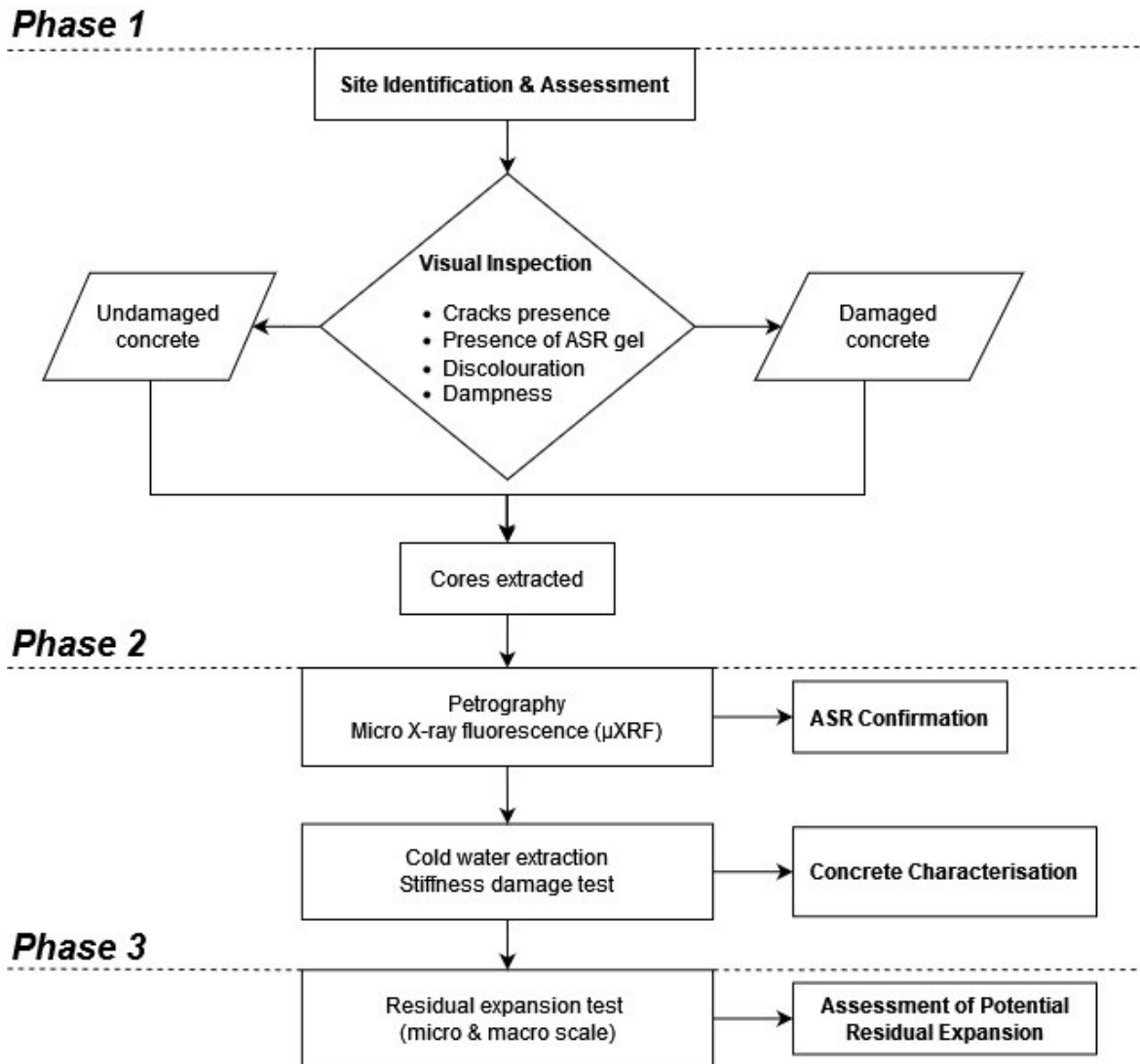


Figure 3-1: Schematic overview of the field investigation and testing methodology for this study

Phase 1

The programme's first phase involved identifying a field structure already subject to ASR deterioration. The decision to use concrete obtained from a field structure rather than manufacture and test concrete subject to ASR deterioration in the laboratory was based on the following reasoning.

To make reliable predictions of the behaviour of an existing RC concrete structure affected by ASR deterioration, it is essential to obtain samples from the actual structure in question for testing. This is because the concrete from field structures are affected by various conditions that are impossible to simulate or replicate using laboratory-casted concrete. Such as the condition of the concrete as a result of how the concrete was cast, the state of stress due to the loading on the structure, availability of moisture (both internally and externally) and environmental factors such as the climate in a particular region. Subsequently, it would be difficult to replicate the extent of ASR deterioration and damage to the concrete from the field structure, such as the orientation and distribution of cracks and stiffness of the material. Thus, by using concrete from a field structure, it is possible to conduct the necessary tests and make reliable recommendations about the likely future behaviour of the concrete.

3.2 Factors considered in the selection of an in-service RC structure

The selection of an in-service RC structure in this study was crucial to determining the potential residual expansion of concrete subject to ASR. Two factors were considered during this process: firstly, the region was chosen based on the presence of reactive aggregates. This was important because not all aggregate types used in the construction of RC structures are reactive. Selecting a region with a known history of reactive aggregates ensured a high probability of the concrete being prone to ASR deterioration. The second factor was that the in-service structure exhibits a degree of visual distress due to ASR deterioration. Case studies have shown that the rate of ASR development may vary depending on the type of reactive aggregate and environmental conditions. Therefore, visible distress on the structure ensured a high probability that the ASR reaction had been initiated. The extent of ASR deterioration was not a significant factor in the selection of an in-service structure.

3.2.1 Factor A - Johannesburg region

As mentioned in Section 0, one of the conditions required for ASR to initiate is the presence of reactive forms of silica in the aggregates used in the concrete; the underlying geology (with respect to the mineralogy and the mechanics of the formation of the aggregates) of a particular region primarily determines this characteristic. There is now a known history of aggregates prone to ASR in the Johannesburg region; the underlying geology contains potentially reactive aggregates such as quartzite and shales associated with the gold-bearing reefs (Section 2.8). Furthermore, prominent RC structures based in the Johannesburg region, such as the “double-deck” stretch of the De Villiers Graaff motorway (constructed in 1962), are known to be affected by ASR (Blight and Alexander, 2011). However, at the time of construction there was no known knowledge on potentially reactive aggregates, in the region.

3.2.2 Factor B - Visual evidence of map cracking and ASR gel

As discussed in Section 2.4, ASR is an expansive reaction between alkalis and aggregates in the concrete; this reaction takes time (Stanton, 1940). Newly built structures will typically not show signs of visual distress owing to ASR in the first five years of the structure’s lifetime (unless the structure contains highly reactive aggregates). If ASR has been initiated within the concrete, the first symptoms are usually noted from visual inspections. The most common observations are variable expansion, development of extensive cracking, surface discolouration, gel exudations and occasional pops-outs (Godart and De Rooij, 2017).

3.3 Diagnosis and confirmation of ASR in concrete extracted from field structures

Once a suitable in-service RC structure was identified, several steps were taken to characterise the concrete and positively diagnose the concrete as being subject to ASR deterioration. Visual observations alone are insufficient to diagnose a structure subject to ASR deterioration. Thus, it was essential to combine visual assessments and other techniques, such as petrography and scanning electron microscope analysis, to determine the underlying cause of damage to the structure (Godart and De Rooij, 2017).

3.3.1 Review of existing documentation and records

Prior to carrying out any assessments or laboratory testing, a review of existing documents (structural drawings, reports, remedial work) on the structure was undertaken. Information that was of interest included structural design information and construction details (the placement of reinforcement, concrete mix design used and drainage systems). This knowledge was useful as it provided a context for what to expect when carrying out further investigations on the structure. It also explained possible damage observed on some aspects of the structure (for example, the location of construction joints, areas exposed to moisture and high-stress levels).

3.3.2 On-site visual assessment of in-service RC structure

Once a suitable in-service RC structure was identified, a visual on-site evaluation was undertaken in accordance with the Guide for Conducting a Visual Inspection of Concrete in Service (ACI 201, 2008).

The visual inspection results were then used as input to evaluate the probability of the field structure's ASR presence as either low, medium, or high, using Table 3-1. Depending on the outcome of the probability assessment, a decision was taken whether to continue with the experimental programme by extracting cores for further laboratory testing or select another field structure that is already subject to ASR deterioration.

Table 3-1: Classification system for assessment of the probability of the presence of AAR based on routine inspection results (modified after CSA A846-00:2000 from (Godart, DeRoosj and Wood, 2013))

<i>Feature</i>	<i>Probability of AAR</i>		
	<i>Low</i>	<i>Medium</i>	<i>High</i>
Cracking & crack pattern	None	Some cracking patterns typical of AAR (e.g. map cracking or cracks aligned with primary reinforcement or stress)	Extensive map-cracking or cracking aligned with primary reinforcement or stress; misalignment of facings (differential expansion on each side of the crack)
Deformation and displacement of elements	None	Some evidence (e.g., closure of joints, spalls, misalignment between structural members	Fair to extensive signs of volume increase leading to spalling at joints, displacement and misalignment of structural members
Surface discoloration	None	Slight surface discolouration associated with some cracks	Many cracks with dark discolouration and adjacent zones of light-coloured concrete
Exudations	None	White exudations around cracks, possibility of colourless jelly-like substance	Colourless, jelly-like exudations readily identifiable as ASR gel associated with several cracks
Pop-outs with ASR gel deposit	None	Some	Many
Environment	Dry and sheltered	Outdoor exposure but sheltered from wetting	Parts of components frequently exposed to moisture, e.g., rain and groundwater, water due to the natural function of the structure.

3.3.3 On-site sample collection: number of specimens, specimen type and size

Once the in-service RC structure was characterised as exhibiting a medium to high probability of being subject to ASR deterioration, samples of the concrete were required to confirm the presence of ASR within the concrete. Owing to the heterogeneity of concrete within a large structure, various areas were identified on the in-service RC structure, which displayed varying degrees of ASR deterioration –“severely damaged” and “slightly damaged” areas. Factors such as element exposure conditions, access to moisture, and visual symptoms of likely ASR deterioration (map-cracking, colourless, jelly-like exudations) were considered when establishing areas that were “severely damaged” and “slightly damaged”. The characterisation of the two regions (“severely damaged” and “slightly damaged”) noted are relative and not absolute. For example, a selected area may be classified as slightly damaged even if the concrete surface displays map cracking. However, it was relative to a damaged area that displayed the same symptom of distress, map cracking, but to a more significant extent.

The extraction of twelve cores from different parts of the structure allowed for a representative assessment of the extent of ASR deterioration within the concrete (more details are provided in section 4.4). Cores were extracted at a height of approximately 1 meter from the ground in designated areas, which ensured accessibility for the coring process while allowing uninterrupted traffic flow beneath the Railway Bridge.

A reinforcement scan was conducted on the respective “severely damaged” and “slightly damaged” surfaces before coring the concrete to ensure no steel was damaged or extracted as part of the core. A minimum of three concrete core specimens were selected from identified “severely damaged” and “slightly damaged” areas. The minimum core diameter of 100 mm and the maximum core barrel length of 300 mm were specified to meet the minimum specimen requirements to conduct the bulk expansion tests (ASTM C 1293, 2002). Coring was carried out under “wet” conditions (using water); this method implied that any ASR gel present on the exterior core surface may have been washed away. This results may have only affected the DRI assessment results which considered the presence of ASR gel (reaction product) in the aggregate or cement paste. Following coring, the holes in the in-service RC structure were filled with cementitious grout. The concrete cores were labelled and transported to the laboratory for further testing.

3.3.4 Assessment of as-received cores

Concrete cores were first examined in an “as-received condition”. A macroscopic inspection of the cores was performed with the naked eye. A core log was completed, and observations such as the size and distribution of aggregates, compaction, voids, and the presence and condition of reinforcement (if cored by accident) were recorded.

A 2D image of the circumferential surface of each concrete core was created. This process involved placing a concrete core, standing vertically on a rotating platform at a fixed speed of one revolution of the core per minute. A camera mounted at a fixed distance (500 mm) from the concrete core was used to capture a video as the core rotated through 360 degrees. VideoLAN Client (VLC) media player software and scene video filter application were used to convert the video into a series of still images (with a resolution of 2.08 megapixels). Images were captured at 0.35 frames per second and manually stitched together (without the use of software) to create a 2D image of the circumferential surface of each concrete.

The Damage Rating Index (DRI) method was modified to evaluate the extent of damage in the concrete and provide a means of relative degrees of damage in various elements of the structure. The DRI method uses concrete samples extracted from either a structure or collected after completion of laboratory testing, which are then prepared for petrographic evaluation. The concrete specimens are generally cut longitudinally along the core length, providing maximum surface area for examination (to allow for a representative assessment of the concrete). The surface is then polished and scribed with a square grid of 1 cm. The sample is then observed using a stereomicroscope at 16 times magnification. The presence of each of the petrographic features listed in Table 3-2 (related to the occurrence of ASR in concrete) is counted in each of the squares examined, and those counts are multiplied by the respective “weighting factors”. The weighted counts are then summed together, and the value is normalised to 100 cm² to produce a gross DRI value for the specimen.

Table 3-2:Weighting factors for the Damage Rating Index (Villeneuve V and Fournier B, 2012)

Petrographic feature	Weighting factor
Closed (sealed) cracks in aggregates (CCA)	0.25
Open cracks in aggregates (OCA)	2
Opened crack with reaction product in aggregate (OCAG)	2
Disaggregated/cored aggregate particle (DAP)	2
Coarse aggregate de-bonded (CAD)	3
Cracks in cement paste (CCP)	3
Crack with reaction product in cement paste (CCPG)	3

In this study, the approach was adapted to characterise the 2D images of each concrete core's circumferential surface as opposed to the traditional approach of cutting up and polishing the extracted concrete cores. This was done due to the limited number of concrete cores for the study. The modified approach involved using a grid of 2 cm squares, as the examination was done using the naked eye which was overlaid on the 2D images circumferential surface images. In addition, the identification of various petrographic features in each grid was not possible using a 16 times

magnification due to the resolution of the 2D image. The examination involved counting the number of petrographic features noted in Table 3-2 for each grid square, as seen in Figure 3-2.

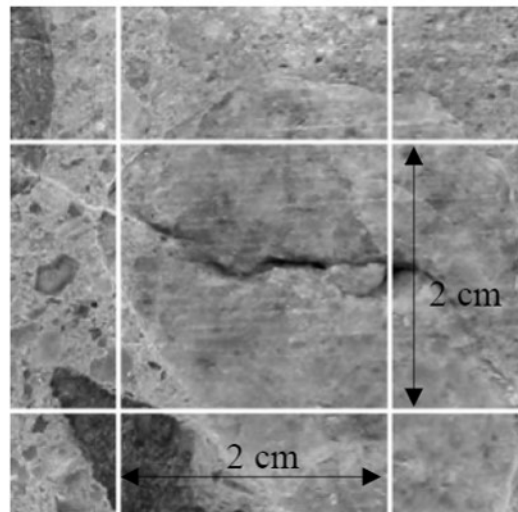


Figure 3-2: Photographic image of core surface scribed with a 2 cm grid illustrating a crack in the aggregate

The implication of using the modified DRI approach was that lower DRI values would be obtained for each concrete core. By using a smaller grid area, a single feature, i.e. a crack, would be counted multiple times in the smaller area, in comparison to a larger area, where the crack would only be counted once, leading to a low DRI value. Also, the aggregate size would have an impact on the final result. Furthermore, using the naked eye would lead to fewer petrographic features being observed compared to a stereomicroscope at 16 times magnification, leading to a lower DRI value.

After these features were recorded for the sample, the total is multiplied by the appropriate weighting factor (representing the relative importance in the overall deterioration process, Table 3-2, to produce a gross DRI value. Due to the varying length of concrete cores extracted from the in-service structure, the total area examined varied for each respective 2D image. Thus the DRI value was normalised to a standard area of 100 cm² to produce a final DRI for the sample.

In addition, colour-coded graphics showing petrographic features on the circumferential surface of the concrete core were developed. Furthermore, the number of petrographic features along the depth of each concrete core and the number of petrographic features relative to the orientation of each core with respect to the structure and the circumference of the concrete core were analysed.

Phase 2

The programme's second phase involved the preparation of the extracted concrete cores for further examination and testing. Two techniques, petrography and micro X-ray fluorescence spectrometry, were used to confirm whether ASR was the underlying deterioration mechanism that led to distress in the RC railway bridge. Lastly, the free alkali content and stiffness damage test was conducted to characterise the concrete taken from various locations of the in-service RC structure

3.4 Specimen preparation

Given the twelve (limited number of) core specimens available for this study, careful consideration was given to using concrete cores to optimise the number of tests conducted. Table 3-3 offers an overview of the tests undertaken and which cores were selected to ensure a representative sample from the various locations of the RC structure.

Table 3-3: Overview of tests conducted on the concrete cores from various locations of the RC railway bridge

Location		Abutment				Pier		Inner wall				Pier	
		1	2	3	4	5	6	7	8	9	10	11	12
i	Petrography	×	×				×	×			×		×
	Micro X-ray fluorescence			×									
ii	Micro expansion	×	×				×	×			×		×
iii	Residual Bulk expansion	×	×				×	×			×		×
iv	Free metal content	×	×				×	×			×		×
v	Compressive strength					×							
vi	Stiffness damage index			×						×		×	
vii	Spare (kept aside in case a particular test needed to be repeated)				×				×				

Six of the twelve cores (cores 1, 2, 6, 7, 9 and 12) were prepared in the same manner. Four factors were taken into consideration when selecting the appropriate cores:

- the concrete core was continuous in length (i.e. not split into two parts at various depths),
- the minimum size of a specimen required to run a particular test,
- ensuring that the sample was taken from approximately the same depth from each of the respective concrete cores,
- at least three concrete specimens were assessed for a given test (one from slightly damaged area, one from moderately damaged area and one from severely damaged area). Except, only one specimen, from the slightly damaged area (instead of three) was tested for the

compressive strength. This results was subsequently used for the stiffness damage index test.

Error! Reference source not found. illustrates the preparation of a typical concrete core, showing the number of concrete discs cut from each core and its location relative to the exterior concrete surface, designated A to E. Two factors determined the thickness of each respective disc. Firstly, the specimen's minimum disc thickness (length) required for a particular test, in this instance, the potential residual expansion test & DIC specimens. The residual expansion test required a minimum specimen length of 200 mm. In the case of the DIC specimens, the decision was taken to use a 10 mm thick disc. This was based on the premise that the study would consider the 2D effect of ASR expansion, thus limiting the thickness (z - component) of the concrete disc, respective to the surface area (xy - component) of the concrete disc under investigation.

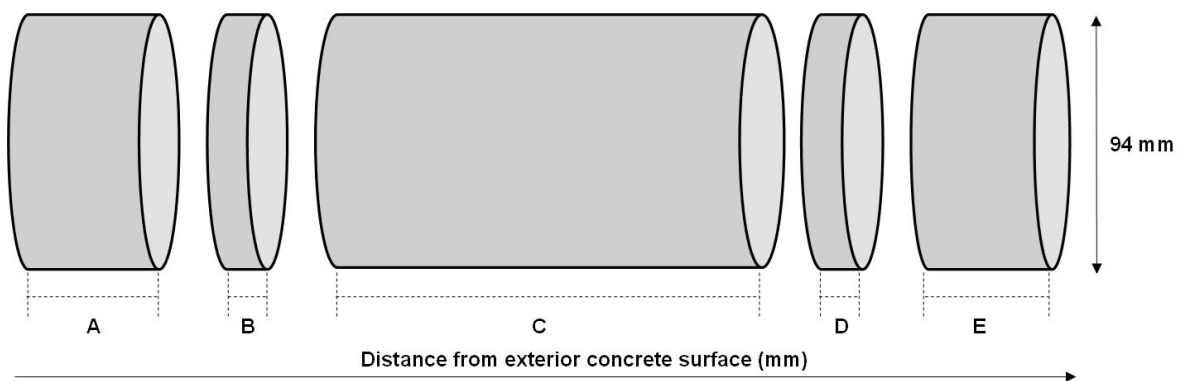


Figure 3-3:Preparation of a typical concrete core for various tests

Secondly, obtain representative samples from both the exterior and interior of the concrete core, with respect to petrography and free alkali metal content. Table 3-4 illustrates each concrete disc's thickness and the test carried out on each concrete disc.

Table 3-4:Thickness and associated test conducted on various concrete discs labelled A to E

Section	Disc thickness (mm)	Test conducted
A	30	Petrography/ free metal content
B	10	Digital image correlation
C	200	Bulk expansion
D	10	Digital image correlation
E	30	Petrography/ free metal content

Three of the twelve cores (3, 10 and 11) were prepared for the stiffness damage index test. Efforts were made to ensure the specimens were cut 4 cm from the exterior concrete surface. This was

done to characterise the interior concrete's stiffness and exclude any surface cracks effects on the test results.

3.5 Petrographic assessment

The visual inspection of the RC railway bridge and the assessment of the “as-received” cores indicated the presence of ASR in the concrete. However, petrographic analysis of the concrete was necessary to confirm that ASR was the primary cause of the underlying damage. Petrography is the generally accepted practice for diagnosing ASR and was performed in accordance with ASTM C 856-11 (2011), Standard Practice for Petrographic Examination of Hardened Concrete.

When selecting specimens for thin section preparation, the focus was on areas that contained cracks in the coarse aggregate, the cement matrix, and, in particular, the interface between the various phases, as illustrated in Figure 3-4. The aim of examining the interface was to identify the presence of ASR gel and note any modified properties of the cement paste and aggregate in the close vicinity. Additionally, the specimens were taken from cores at different locations on the RC railway bridge to ensure a representative sample.

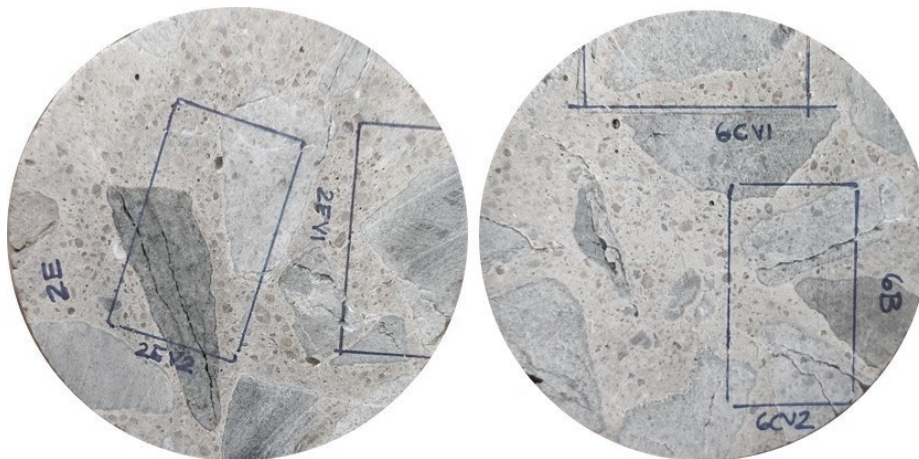


Figure 3-4: Concrete discs with a selection of marked-off areas as the focus area in preparation of thin section slides

Thin sections were prepared following procedures detailed in Humphries (1992) from extracted concrete cores taken from various locations of the RC railway bridge. The selected concrete and aggregate pieces were cut and impregnated with clear epoxy resin to reinforce the sample material, seal off porosity and reduce preparation artefacts. To reduce the influence of microstructure alterations, the specimens were lapped plane, then mounted on a 27×48 mm sized carrier glass with clear epoxy resin to produce $25 \times 43 \times 0.030$ mm thin sections. Thin sections were assessed

using an Olympus BX41 microscope with a digital Olympus DP74 camera attached in plane-polarised light (PPL) and crossed-polarized light (XPL) as shown in Figure 3-5. This particular microscope had a motorised microscope stage, and a full scan of each respective thin section was performed.



Figure 3-5: Olympus BX41 microscope with a digital Olympus DP74 camera attached

This technique was used to identify various rock types based on the microstructure and texture/fabric with respect to the degree of crystallisation, grain size, shape, contacts and undulatory extinction (MacKenzie, Donaldson and Guilford, 1982; Vernon, 2011). These properties and qualities contribute to the variable alkali-reactivity behaviour in concrete between different rock types, as well as for a single given rock type from one locality to another, laterally within a single geological formation, or from various levels within a single quarry (Oberholster, 1996; Broekmans, Fernandes and Nixon, 2009).

In addition, the technique was also used to identify diagnostic features of ASR damage in the concrete, such as ASR gel and cracks radiating from reactive aggregates to the surrounding cement paste.

3.6 Micro X-ray fluorescence (μ -XRF)

Micro X-ray fluorescence (μ -XRF) is a non-destructive, elemental analysis technique which uses direct X-ray excitation to induce characteristic X-ray fluorescence emission from a specimen. This technique aimed to identify and characterise the presence of ASR gel in the concrete, which was not clearly identified from the petrographic analyses. This instrument was chosen over TIMA (TESCAN Integrated Mineral Analyser) and QEMSCAN (Quantitative Evaluation of Minerals by Scanning Electron Microscopy), as the instrument allows for a much larger area to be scanned 10 x 10 cm in comparison to a limited area of a thin section 25 × 43 mm.

The Bruker M4 Tornado Micro-XRF (Figure 3-6) was used to scan the concrete discs, and the resulting data was analysed using the HyperMap software. The specimens were subjected to a chamber pressure of 2 millibars, and the beam conditions during scan analyses were 50 kV accelerating voltage and 600 μ A emission current. The Bruker M4 Tornado Micro-XRF has a relatively large specimen chamber (600 mm x 350 mm x 260mm), which allowed for the analysis of 1 cm thick concrete discs cut from selected concrete cores. The only requirement was that the specimen scan surface be as flat as possible to obtain accurate and reliable results.



Figure 3-6: A flat concrete disc is placed on the Bruker M4 Tornado micro-XRF stage in preparation for the scan.

Additionally, the technique was employed use Energy Dispersive X-ray (EDX) Spectroscopy to produce elemental maps of the concrete thin sections, focusing on identifying the specific elements present in the aggregates. Feldspars and micas present in some aggregates are possible sources of alkali in addition to that derived from cement (Lagerblad, 2012). Constantiner and Diamond (2003) have shown that up to 10% of the alkali content may be derived from the aggregates in rare cases.

If these aggregates are embedded in a cement matrix, they release alkali ions into the pore solution, and if ASR-reactive aggregates are present, these released alkali ions that can participate actively in ASR.

3.7 Concentration of free alkali metals

The cold water extraction (CWE) method (Bérubé M, Duchesne J and Rivest M, 1996) was used to determine the free alkali metal content per mass of concrete. The free alkali content of the concrete was assessed at two depths for a given concrete core: close to the exterior concrete surface in the region of 10-30 mm and further along the depth of the concrete core in the region of 250-280 mm from the exterior concrete surface. This was to determine if there was any variation in the quantity of free alkali metals in the concrete close to the exterior surface compared to the interior bulk concrete.

Aggregates embedded in the concrete matrix can also release alkali metals during CWE, as they are part of the mortar that is crushed and milled. This can lead to an overestimation of the concrete's free alkali metal content. To correct the results obtained for the concrete, the contribution of the aggregates must be known, so CWE was also performed on the coarse aggregates extracted from the concrete.

Firstly, the respective concrete discs were mechanically crushed (using a hammer) to separate the coarse aggregates from the mortar. The coarse aggregates were soaked in diluted hydraulic acid solution to remove residual mortar on the aggregate surface and rinsed with deionised water. The CWE requires the use of fine powder ($< 80 \mu\text{m}$), so all the samples (both mortar and aggregate) were ground with a vibratory disc mill (RS200 from Dickie & Stockler) at 960 rpm for 60 seconds for the aggregate samples and 120 seconds for the mortar samples. The material was passed through an $80 \mu\text{m}$ sieve, and the powders were stored in desiccators with soda lime until further analysis.

For CWE, 20 g of powder with a maximal grain size of $80 \mu\text{m}$ (for both the mortar and the aggregate alone) was mixed with 20 g of deionised water and stirred for 5 minutes. The suspension was then filtrated for approximately one minute using a water pump and a Grade 40, $8 \mu\text{m}$ particle retention Whatman cellulose filter. Finally, each 1 ml of filtrate solution was diluted ten times with deionised water and acidified with HNO_3 to obtain a final $[\text{HNO}_3]$ concentration of 0.1 mol/l. A

Spectroquant Pharo 300 machine was used to analyse the solution and determine the Na⁺ and K⁺ concentrations. The method was applied three times for each sample.

3.7.1 Free alkali metal content

Based on the Spectroquant Pharo 300 results, the free alkali metal content (not yet corrected for the release from the aggregates during CWE) was calculated using Equation 3-1 :

$$X_{conc} = [X]_{ps+extr} \cdot \frac{M_{ps} + M_{extr}}{M_{sample}} \quad \text{Equation 3-1}$$

where:

X = either Na or K

X_{conc} = alkali metal content of the concrete (mmol/kg)

[X]_{ps + extr} = concentration of X in the filtrate obtained from CWE (mmol/l)

M_{ps} = amount of pore solution in the sample (g)

M_{extr} = amount of extraction solution (deionised water or methanol) added during CWE (g)

M_{sample} = mass of the sample (g)

3.7.2 Free alkali metal content taking into account release from aggregates

To determine the original concentration of the alkali metals in the pore solution, [X]_{ps} was calculated using Equation 3-2:

$$[X]_{ps} = [X]_{ps+extr-aggr} \cdot \frac{M_{ps} + M_{extr}}{M_{ps}} \quad \text{Equation 3-2}$$

where:

[X]_{ps+extr-aggr} = concentration of X in the filtrate obtained from CWE (mmol/l) and corrected for the release of X from the aggregates during CWE.

$[X]_{ps+extr-aggr}$ was determined using Equation 3-3:

$$[X]_{ps+extr-aggr} = [X]_{ps+extr} - \frac{[X]_{aggr} \cdot M_{sample} \cdot A/100}{M_{extr}} \quad \text{Equation 3-3}$$

where:

$[X]_{aggr}$ = concentration of X in the filtrate after performing CWE on the aggregates alone

A = % of aggregates (A)

3.7.3 Calcium concentration

The CWE method using de-ionised water does not allow Ca^{2+} concentration determination in the sample's pore solution due to the dissolution of portlandite during the process, as shown by Plusquellec et al. (2017). To circumvent this issue, water was replaced with methanol when conducting the CWE method to determine the Ca^{2+} concentration.

3.7.4 Determination of concrete pore solution composition based on free alkali metal content

The respective sodium and potassium metal contents were used as inputs to determine the composition of the concrete pore solution. This step was necessary as these results were used to determine the composition of the fabrication pore solution needed for the residual potential test. The Na^+ and K^+ metal contents (mmol/kg) were converted to $Na_2O_{eq,conc}$ (kg/m³) using Equation 3-4:

$$Na_2O_{eq,conc} = \frac{M(Na_2O)}{2} \cdot (Na_{conc} + K_{conc}) \cdot \rho_{concrete} \quad \text{Equation 3-4}$$

where:

$Na_2O_{eq,conc}$ = alkali metal content in kg/m³,

$M(Na_2O)$ = molar mass of Na_2O (0.06198 kg/mol),

Na_{conc} and K_{conc} = the Na^+ and K^+ contents of the sample expressed in mol/kg,

$\rho_{concrete}$ = concrete density in kg/m³ (assumed to be 2 400 kg/m³)

3.8 Stiffness Damage Test (SDT)

In preparation for the STD, the specimens were first faced on both ends and then kept moist by being placed in water at 23 °C for a month before testing. The stiffness damage test was conducted indirectly using ASTM C469-02 (2017). The testing process involved subjecting the specimens to five cycles of compression loading and unloading in a compression machine at a controlled loading rate of 0.10 MPa/s. A compressive load of 67 kN was used, which corresponded to 30% of the concrete's compressive strength. During testing, deformation data was obtained from an LVDT connected to a datalogger. The Amsler machine was also connected to a datalogger that recorded the load applied to the specimen throughout the test, as illustrated in the test setup in Figure 3-7.

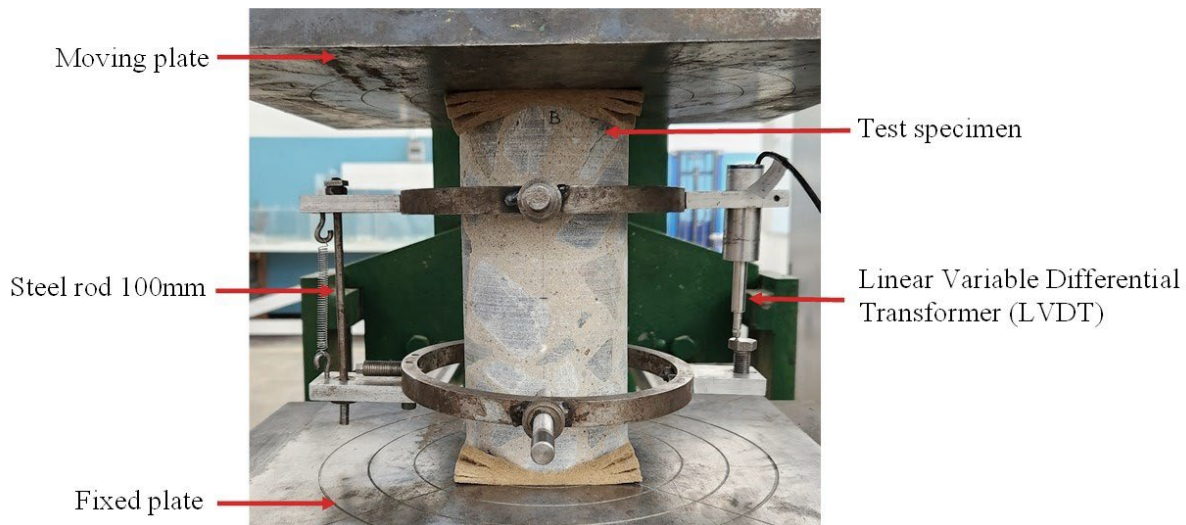


Figure 3-7: Test setup used for stiffness damage test

The load-deformation data obtained was used to calculate and plot the stress-strain curve for each specimen. Six indices were determined as outputs of the test procedure; their description is provided in Table 3-5, along with the schematic shown in Figure 3-7. The area of the load cycle hysteresis loop was calculated by subtracting the area under the unloading curve from the area under the loading curve.

Table 3-5: Output parameters calculated from stiffness damage test

Index	Description
Modulus of elasticity	Average secant modulus of the 2 nd , 3 rd , 4 th and 5 th cycles
Dissipated energy	Hysteresis area
Plastic deformation	Plastic strain
Stiffness Damage Index (SDI)	Ratio of dissipated energy to total energy in the system over the five cycles. (i.e., $SI/(SI + SII)$ and) SI is the irreversible deformation energy, SII is the elastic deformation energy (as illustrated in Figure 3-7)
Plastic Deformation Index (PDI)	Ratio of plastic deformation to total deformation in the system over the five cycles. (i.e., $DI/(DI + DII)$ DI and DII represent the plastic and elastic deformation over the loading-unloading process. (as illustrated in Figure 3-7)
Non-linearity index (NLI)	The ratio Sec 1/Sec 2 in the first cycle and the average Sec1/Sec2 value of the last four cycles. (as illustrated in Figure 3-7)

Furthermore, to evaluate the impact of moisture on the stiffness of the concrete, specimens were subjected to a one-month drying period at room temperature (23 °C), followed by the performance of an SDT. Subsequently, the specimens were immersed in water for another month, after which the SDT was repeated. The cyclic wet/dry effects on the stiffness damage index (SDI) of concrete were considered significant because they directly influence the long-term durability, structural integrity, and service life of concrete infrastructure, especially in environments exposed to fluctuating moisture conditions.

This cyclic exposure of being submerged in water and subsequent drying was repeated three times. There were possible risks of ion leaching due to concentration gradient from exposed surfaces; however, the focus of this test was not on the creation of new ASR gel (which would have been impeded by the leaching of alkalis).

Phase 3

The programme's third phase included determining the residual potential expansion on the macro and micro scales of the concrete obtained from an in-service RC structure subject to ASR deterioration.

1.1 Residual potential expansion of field concrete – macro scale

The residual expansion of the bulk concrete cores was determined using the procedure outlined by Laboratoire de Matériaux de Construction in Lausanne (LMCL). The test method consists of storing samples in a simplified pore solution at 38 °C and measuring the expansion of the specimen subject to ASR with time. The test method is divided into three steps:

i. Manufactured pore solution

The CWE method was used to determine the alkali content per volume of concrete being tested. The detailed procedure for this test method is outlined in Section 3.7. A synthetic pore solution was manufactured using KOH and NaOH pellets dissolved in water at room temperature (23 °C).

ii. Preparation of sample

Specimens were cut to 200 mm in length and faced on both ends. Steel studs were glued onto the concrete specimens along the length of the specimen with epoxy resin, and four axes (A, B, C & D) were created, each 90° to each other, using the arrangement shown in Figure 3-8.

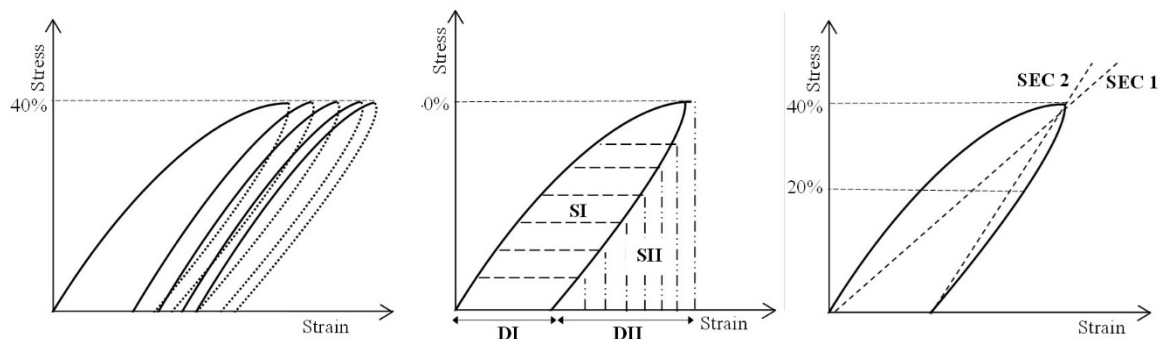


Figure 3-8: Calculation of the stiffness damage index (SDI), Plastic Deformation Index (PDI) and Non-Linearity Index (NLI) from a typical stress–strain graph

Furthermore, to evaluate the impact of moisture on the stiffness of the concrete, specimens were subjected to a one-month drying period at room temperature (23°C), followed by the performance of a Stiffness Damage Test. Subsequently, the specimens were immersed in water for another month, after which the SDT was repeated. This cyclic exposure of being submerged in water and subsequent drying was repeated three times.

Phase 3

The programme's third phase included determining the residual potential expansion on the macro and micro scales of the concrete obtained from an in-service RC structure subject to ASR deterioration.

3.9 Residual potential expansion of field concrete – macro scale

The residual expansion of the bulk concrete cores was determined using the procedure outlined by Laboratoire de Matériaux de Construction in Lausanne (LMCL). The in-house test method consists of storing samples in a simplified pore solution at 38 °C and measuring the expansion of the specimen subject to ASR with time. The test method is divided into three steps:

i. Manufactured pore solution

The cold water extraction (CWE) method was used to determine the alkali content per volume of concrete being tested. The detailed procedure for this test method is outlined in Section 3.7. A synthetic pore solution was manufactured using KOH and NaOH pellets dissolved in water at room temperature (23 C°).

ii. Preparation of sample

Specimens were cut to 200 mm in length and faced on both ends. Steel studs were glued onto the concrete specimens along the length of the specimen with epoxy resin, and four axes (A, B, C & D) were created, each 90° to each other, using the arrangement shown in Figure 3-9.

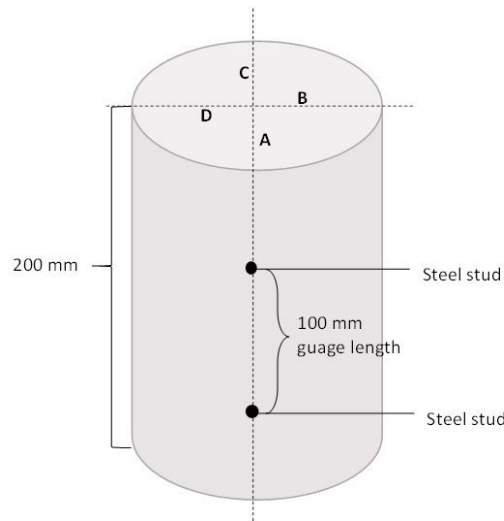


Figure 3-9: Placement of steel studs on concrete specimen

iii. Expansion measurement

At the start of the test, samples were weighted, and reference strain readings were taken. A 100 mm Demec gauge was used with an accuracy of 2.00×10^{-5} . Samples were immersed in the fabricated pore solution for 30 minutes, after which they were removed, and weight measurements were taken. Thereafter, specimens were placed in a sealed container and covered by the pore solution maintained at a constant temperature of $38^\circ\text{C} \pm 2^\circ\text{C}$. Mass and strain measurements were systematically recorded at seven-day intervals over the course of the four-month testing period. Although such tests can extend up to one year, the duration in this study was shortened due to time constraints.

3.10 Residual potential expansion of field concrete – microscale

Two approaches were used to quantify and assess the development of microstrain of concrete subject to ASR: DIC and Demec gauge.

3.10.1 Digital Image Correlation (DIC)

A Good Practices Guide for Digital Image Correlation by Bigger et al. (2018) was used to design, execute, and analyse DIC measurements. The document is not a standard but provided insight on

DIC measurement setup, image correlation, and basic post-processing of DIC data for strain computations.

Firstly, a customised DIC rig was manufactured to accommodate a camera lens, test specimen and specific testing requirements. This specialised rig was designed to facilitate the precise positioning of a single specimen, which allowed for the capturing of an image. After each photo was taken, the specimen could be easily replaced, ensuring a seamless transition to the next specimen. Figure 3-10 illustrates a schematic of the DIC rig, showcasing its various components.

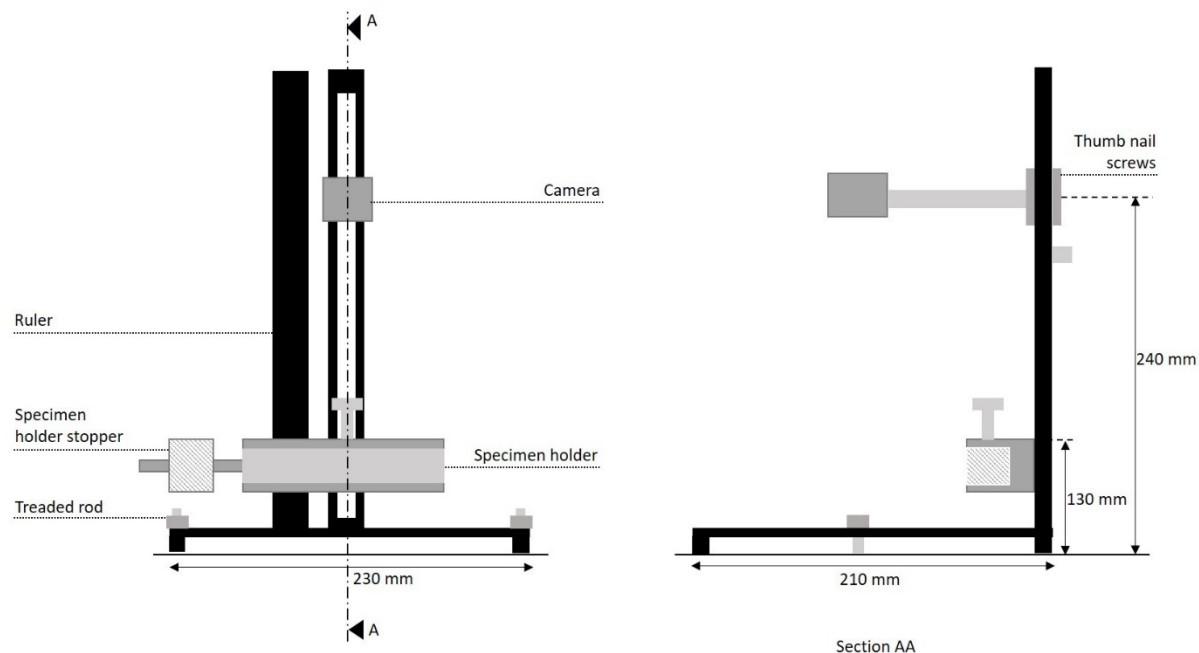


Figure 3-10: Customised DIC rig for local expansion measurements of concrete subject to ASR (front view & side-view)

To ensure stability during testing, the rig was securely fixed to a concrete table using threaded rods embedded in a cement paste. This robust mounting system provided a reliable foundation for the DIC rig, ensuring accurate and consistent results. The optical setup consisted of a Basler Premium Lens with C-mount, a fixed focal length of 12.0 mm and a resolution of 5 megapixels. The optimal focal distance (distance between camera and specimen) was determined using the lens selector tool on Basler's website. This ensured that individual speckles particles were in focus in the captured images.

Fluorescent lights were secured on stands on the DIC rig's left and right sides. The lights were positioned 60 mm from the edge of the DIC rig, located 270 mm above ground and titled at an angle of 30 degrees (where zero degrees was the fluorescent lights position, when the light shining was parallel to the specimen in the holder). They were incorporated into the DIC set-up for two reasons. Firstly, a light source should be provided when capturing an image of the concrete discs. Secondly, the effect of the light waves showing on the image should be eliminated if only one light source was used.

3.10.2 Specimen preparation

For each 10 mm concrete disc, various steps were taken to prepare a test specimen for DIC purposes, as outlined in Figure 3-11. The concrete disc with the flattest surface was selected and polished to achieve a smooth finish. Subsequently, the surface was brushed and wiped down with water to remove any remaining dust particles that had resulted from the polishing process. Following this preparation, a precise 20 mm grid was inscribed onto the surface, and a reference image was taken to note the relative position of aggregates in relation to the cement paste.

For each disc the region of interest was a 20 mm by 20 mm square, positioned in the centre of the disc for the DIC analysis. This size square was selected as it allowed for micro-scale level strain measurements across multiple phases, particularly at the interfacial transition zone (ITZ) between cement paste and aggregate in concrete. Special effort was taken to ensure that the largest aggregate particles (size being in the order of 60 mm) were not solely present in the respective concrete disc region of interest.

A uniform application of Plascon brilliant sheen, polyvinyl acetate (PVA) acrylic white plain was carefully applied across the entire concrete surface. After allowing the first coat to dry for 24 hours, a second coat was applied. The original 20 mm grid was then redrawn in the same position as the initial grid on the disc's surface. The creation of a distinctive speckling pattern involved the use of matte black spray paint. The specimens were laid flat on a table and the aerosol can, positioned 1 000 mm away, was applied for a minute.

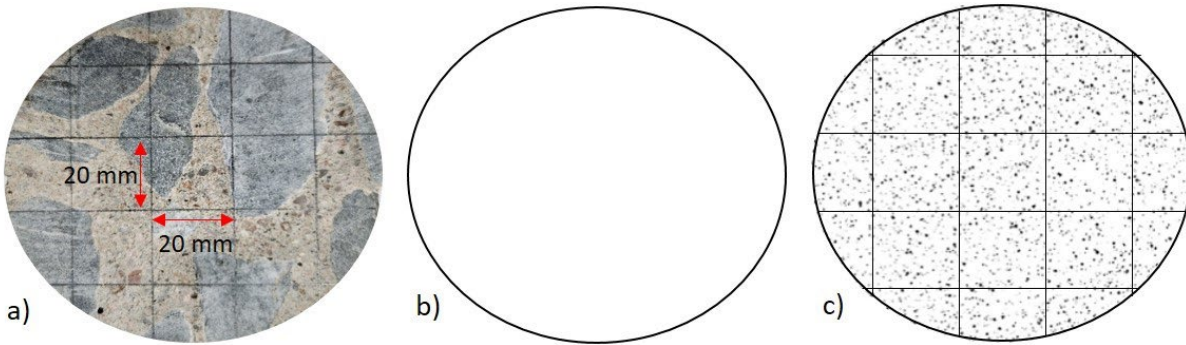


Figure 3-11: Specimen preparation for DIC application

This technique resulted in a randomised speckle pattern, with speckle diameters ranging from approximately 57 μm to 187 μm , as illustrated in Figure 3-12. Effort was made to ensure that the speckle size was relatively small, as large speckles cannot be tracked when considering the magnitude of expected ASR strains. After the speckle application, the specimens were left to dry for twenty-four hours.

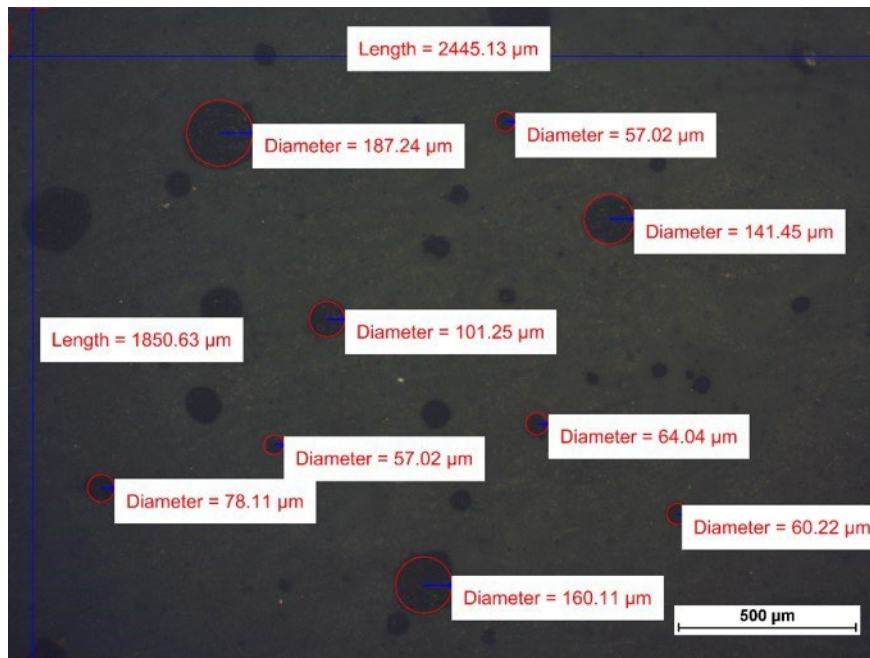


Figure 3-12: Measured diameters of a selection of speckles, viewed using a microscope under reflected light

Steel studs were glued to the concrete specimens using a strain rosette arrangement to facilitate strain readings, as shown in Figure 3-13. This arrangement allowed strain readings to be taken at

zero, forty-five, and ninety-degree angles to the 2 cm region of interest. A 50.8 mm (2 inches) Demec gauge was used with an accuracy of 2.5×10^{-5} . Using this approach, measuring the normal strains (not shear strains) on the material's surface was possible. From the three independent measurements of normal strain at a point (ϵ_0 , ϵ_{45} and ϵ_{90}), it was possible to determine the principal strains and their respective directions using Equation 3-5 to Equation 3-9.

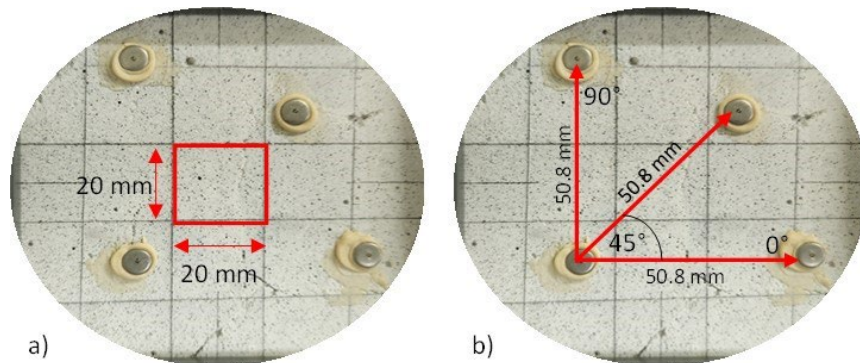


Figure 3-13:Region of interest (ROI) and strain rosette to validate DIC measurement.

$$\epsilon_x = \epsilon_0 \quad \text{Equation 3-5}$$

$$\epsilon_y = \epsilon_{90} \quad \text{Equation 3-6}$$

$$\gamma_{xy} = \epsilon_{45} - \frac{\epsilon_0 + \epsilon_{90}}{2} \quad \text{Equation 3-7}$$

$$\epsilon_{1,2} = \frac{\epsilon_x + \epsilon_y}{2} \pm \sqrt{\left(\frac{\epsilon_x - \epsilon_y}{2}\right)^2 + \gamma_{xy}^2} \quad \text{Equation 3-8}$$

$$\tan(2\theta_p) = \frac{2\gamma_{xy}}{\epsilon_x - \epsilon_y} \quad \text{Equation 3-9}$$

Once the specimen preparation was complete, specimens were placed in custom PVC specimen holders, as shown in Figure 3-14. The figure illustrates two stainless steel threaded rods chamfered at the ends to secure the specimens in position. This approach implies that the specimen is not entirely unrestrained and is subject to two-point restraints.

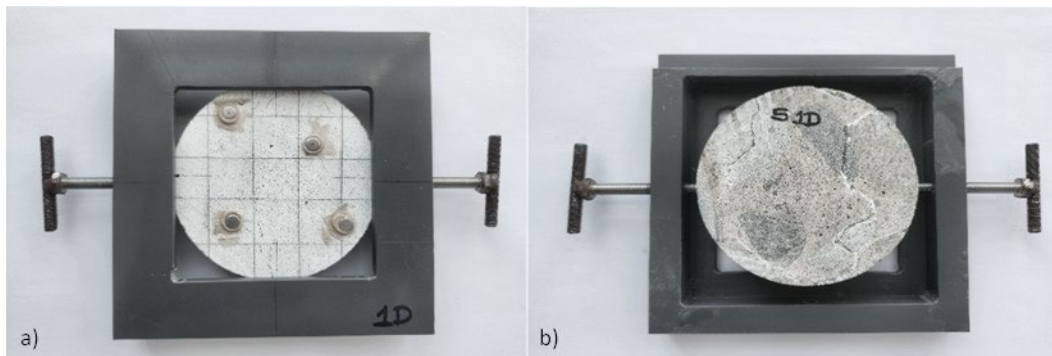


Figure 3-14: Aerial and underside view of concrete disc secured in specimen holder by steel threaded rods

3.10.3 Specimen conditioning

Reference images were captured at the start of the experiment using the set-up in Figure 3-15, followed by the systematic recording of manual strain readings.

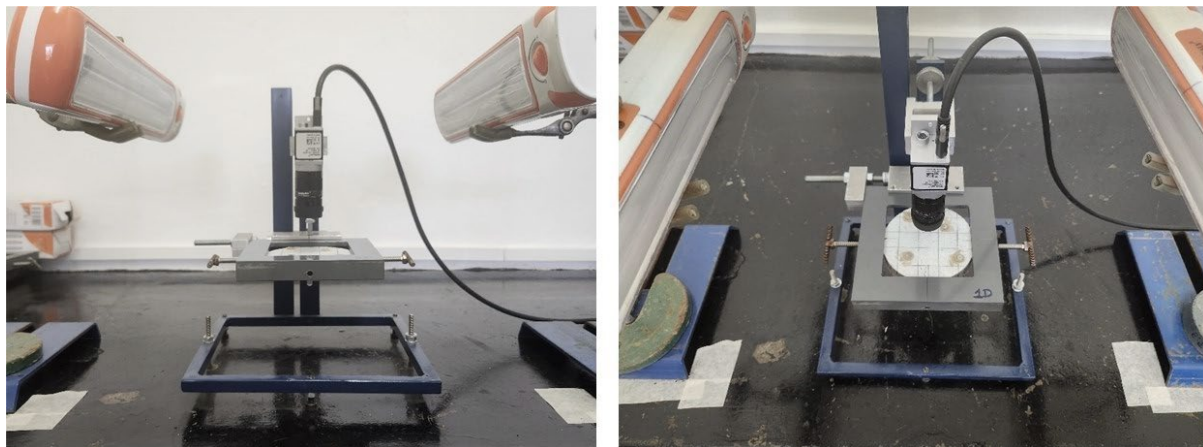


Figure 3-15: Front and elevation view of concrete disc secured in DIC rig when capturing image for DIC

Subsequently, the specimen was elevated within a plastic container containing a 1 cm layer of tap water, as shown in Figure 3-16. The container was then carefully positioned in an oven set to 50°C. Periodic strain measurements and images were captured at regular intervals of 28 days throughout the entire testing duration.

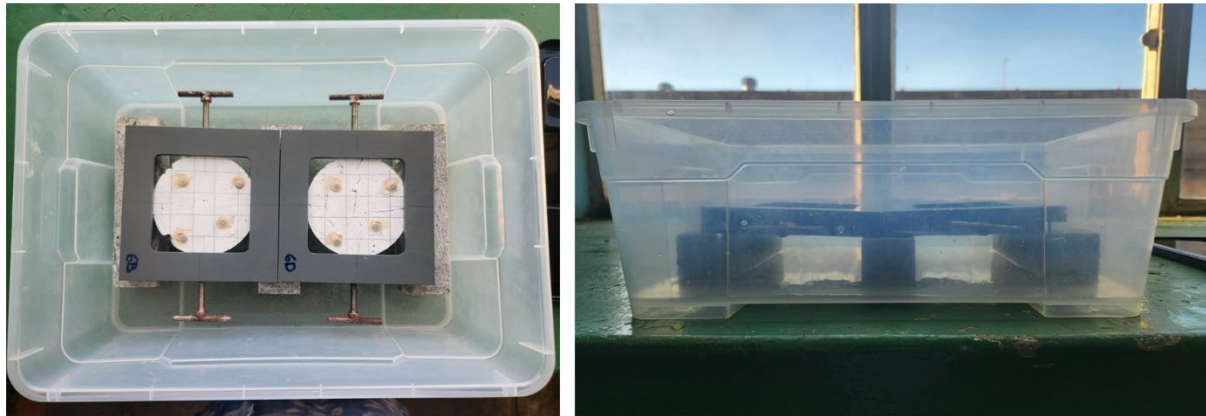


Figure 3-16: Specimen holders with concrete discs kept in plastic containers and placed in an oven set to 50 degrees

3.10.4 DIC software

Pylon Viewer software was used for image acquisition, and MATLAB based DIC and tracking, an open-sourced software, was used to run the DIC simulation. The software incorporates a series of steps that require decisions to be made to determine the strain of the concrete discs, as outlined below:

- Selection of the reference image to use in DIC analysis
- Input the first four letters that were used to label/identify the images
- Input the total number of images that were to be included in the DIC analysis
- Selection of the region of interest (ROI) in the reference image, as shown in Figure 3-17.

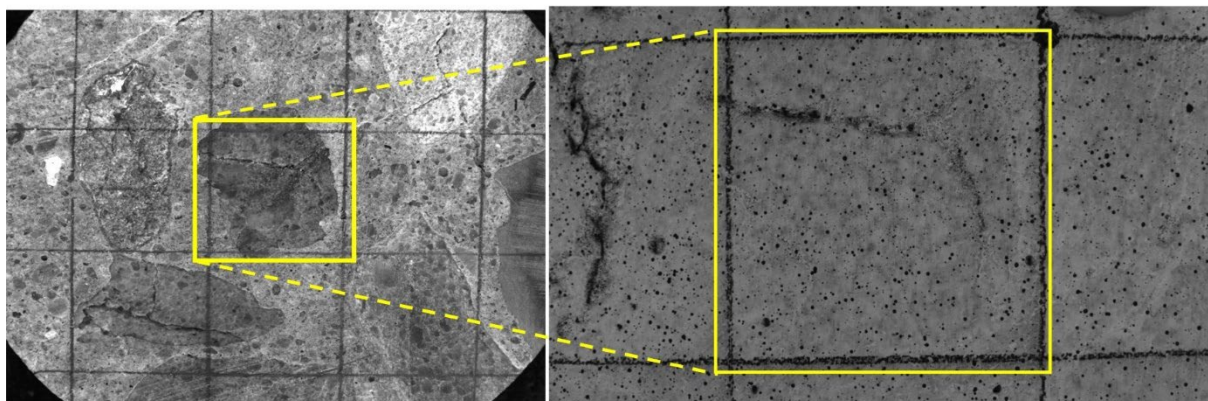
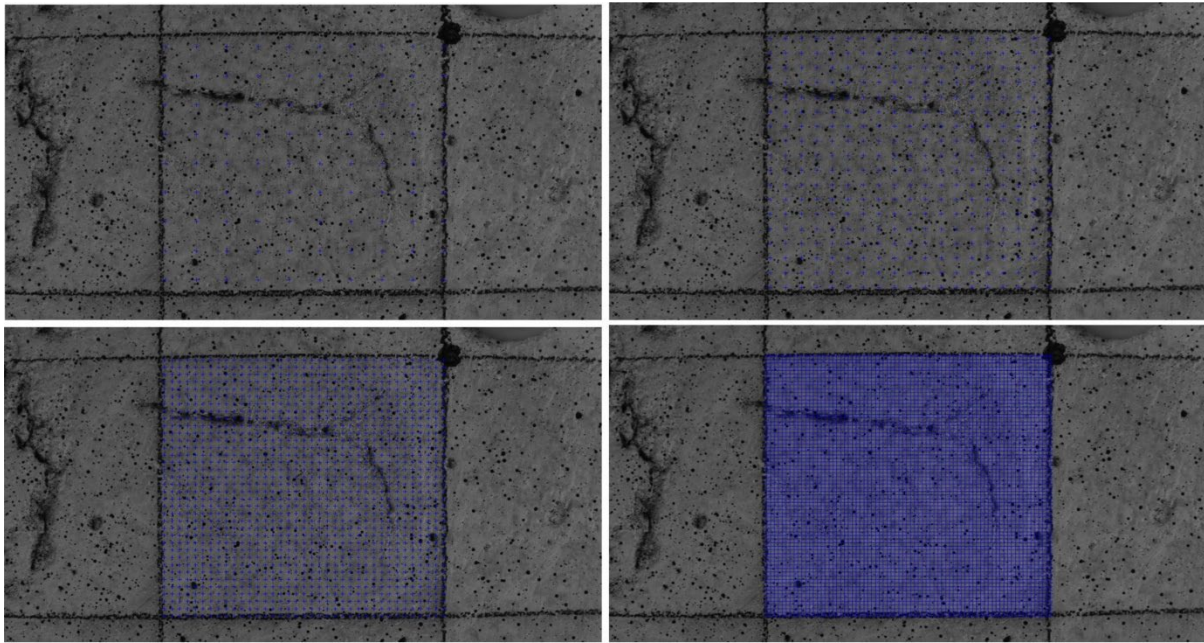


Figure 3-17: Original concrete disc and reference image showing the region of interest that was selected in the image for disc 12D

- Determine the size of the subset based on the number of pixels, as shown in Figure 3-18. This decision subsequently determines the number of raster points in the region of interest. As can be seen, the greater the subset size, the lower the number of raster points in the region of interest. These points are subsequently tracked when running the DIC computations over the series of images taken.



*Figure 3-18:*Effect of the number of subsets in the selected region of interest and the number of raster points that are subsequently tracked in the DIC computation

3.11 DIC simulation

The attempt to employ digital image correlation to quantify and assess the residual expansion strain on a local micro-scale proved unsuccessful. It can be seen that from Figure 3-19, the displacement results for disc 12D are zero pixels; thus, no strains were calculated using the DIC software. Despite the selection of small subsets to track the displacement, the software could not detect any deformation and subsequent strain readings.

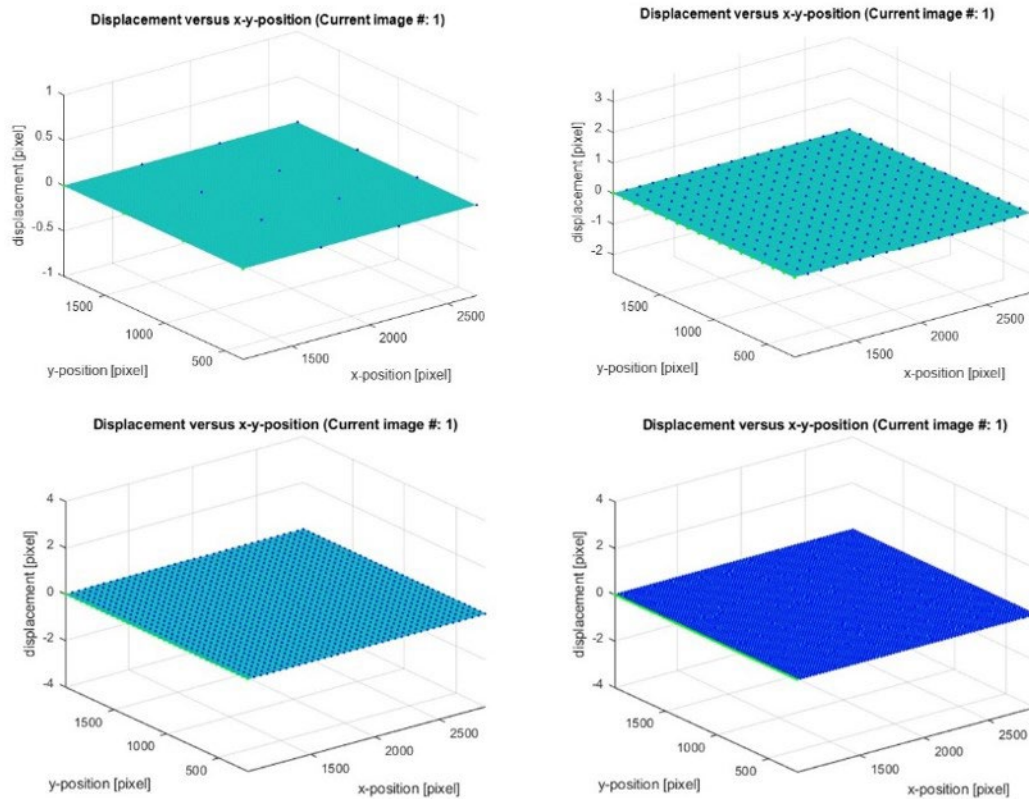


Figure 3-19: Displacement results for disc 12D when using subset size of 25, 50, 100 and 200 pixels

Despite the development of a customised rig and specimen holders designed to facilitate the placement of different specimens in the rig and capture images, two main concerns using this approach were noted.

Firstly, a 10 mm thick concrete disc, with a square region of interest of 20 mm, was used to quantify concrete's local potential residual strain subject to ASR deterioration. Based on the work conducted by Teramoto et al., (2018) in section 0, the use of DIC with a 10 mm thick specimen with a square 100 mm region of interest did not generate strains across individual phases and at the interfacial transition zone between cement paste and aggregate in concrete. Thus, the selected thickness of concrete discs, in addition to the magnified region of interest in the study, may have resulted in 2D DIC being unable to quantify expansion strain on a local scale.

Secondly, over time, it became increasingly difficult to return the specimen holder precisely to its original position for image capture. Due to the scale of the strain being measured across a 20 mm

region of interest, even a slight rotation or shift in the captured images created errors when tracking the image subsets (group of pixels) through DIC simulation. Consequently, the strain results were compromised, rendering them nonsensical.

3.12 Correlation of micro and macro potential residual strain

The study considered the anisotropic characteristics of residual expansion measurements, which are affected by the casting orientation, the stress state, and the method used to record potential residual readings (longitudinal versus diametral). The relationship between these variables proved essential in establishing a correlation between micro and macro strain, as depicted in the extracted concrete core shown in **Error! Reference source not found.** It can be seen that the macro (bulk) expansion strains were measured longitudinally along the length of the concrete cores. This was in contrast to the micro strains that were measured diametrically relative to the length of the concrete cores.

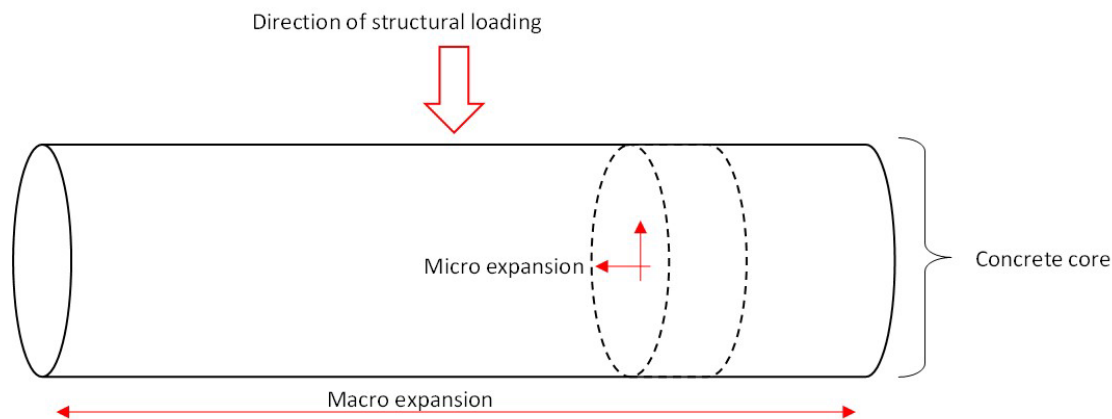


Figure 3-20: Direction of measured micro and macro potential residual expansion measured in concrete cores relative to the direction of structural loading

Research conducted by Merz and Leemann (2013) highlighted the difference in residual expansion readings observed between the longitudinal and diametral directions of cores, as depicted in Figure 3-21. In Phase 1 of the test, corresponding samples were conditioned at 20°C in containers with a thin layer of water at the bottom. Phase 2 involved storing specimens in a reactor at 38°C with RH > 95%. Lastly, Phase 3 involved removing the cores from their containers and drying them at 60–70% RH to their initial weight.

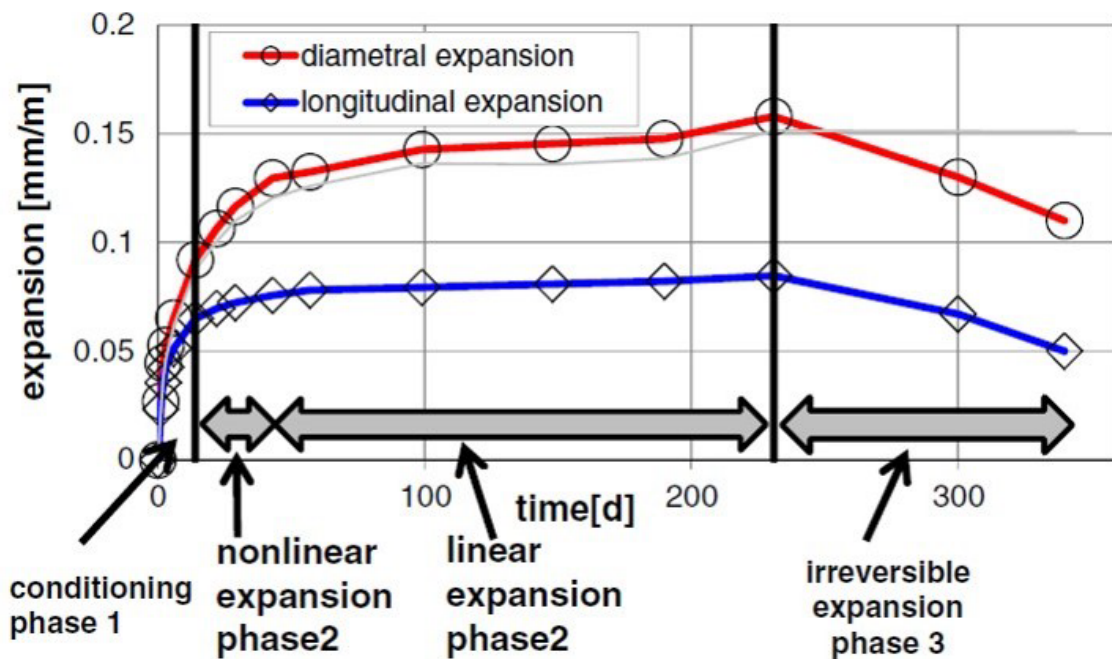


Figure 3-21: Different expansion phases during the residual expansion measurements in longitudinal and diametral directions of cores.

The maximum difference between longitudinal and diametral residual expansion in Phase 2, the linear expansion phase, was approximately 0.075 mm/m. This value was used to convert the maximum average longitudinal strains from the bulk residual potential expansion test to equivalent maximum diametral strain values. Furthermore, a direct comparison between local strain and bulk strain was deemed inaccurate due to the local strain occurring over a length of 50.8 mm, while bulk strain occurred over a length of 100 mm. Therefore, a conversion factor of 1.9685 was applied to all local strains, producing a local strain equivalent value that was then compared to bulk expansion strain values.

3.13 Summary

This chapter describes the field inspection and laboratory testing undertaken to gather data to determine if a correlation exists between the expansion exhibited (with respect to the swelling pressure and subsequent strain development) by concrete subjected to ASR deterioration on both micro and macro scales.

The field investigation and experimental methodology consisted of three main phases. Phase one involved identifying and visually assessing an in-service RC structure subjected to ASR deterioration. Phase two involved confirming the ASR diagnosis and characterising concrete taken from various locations of the in-service RC structure. Phase three assessed the potential residual expansion of concrete obtained from the in-service RC structure characterised in Phase 2 on a micro and macro scale.

Twelve cores were extracted from various locations of the RC railway bridge, showing varying degrees of distress. Cores were carefully selected for testing to ensure a representative sample from different areas of the RC structure—slightly damaged, moderately damaged, and severely damaged. Six of the twelve cores (1, 2, 6, 7, 9 and 12) were prepared similarly for petrography and to determine the free alkali metal concentration, bulk expansion, and local expansion. Three cores (3, 10, and 11) were prepared similarly for the stiffness damage index test.

The analyses and discussion of the results are presented in Chapter 4.

Chapter 4 Results

4.1 Introduction

The results and analyses of the field inspection and laboratory-based investigation into estimating the potential residual expansion due to ASR in in-service RC structure are presented in this chapter. The chapter is divided into three phases. The first phase presents the results, analyses and discussion of the data obtained from the visual assessment of the RC railway bridge subjected to ASR deterioration. The second phase presents the results, analyses and discussion of the data obtained from the characterisation tests performed on concrete specimens taken from various locations of the RC railway bridge subject to ASR deterioration. The third part presents the results, analyses, and discussion of the data obtained from potential residual expansion tests on micro- and macro-scale concrete specimens characterised in Phase 2.

Phase 1 – Results & Discussion

4.2 Selection of RC field structure

An RC railway bridge in central Johannesburg (circled in red in Figure 4-1), which displayed signs of distress in the form of severe map cracking, was selected as the field structure subject to ASR deterioration. The railway bridge runs over Treu Road and through the Crown Mines Spur and forms part of the railway line from Benrose to Soccer City. The Passenger Rail Agency of South Africa (PRASA) manages the RC railway bridge.



Figure 4-1: Geographical location of the RC railway bridge in Johannesburg (GPS co-ordinates 26°13'13.4"S 28°01'31.3"E)

4.3 Review of RC railway bridge structural drawings

Figure 4-2 provides an elevation view of the RC railway bridge obtained from existing construction drawings in Appendix A - Site plan and construction drawings of RC railway bridge located on Treu Road, in the Crown Mines Spur.

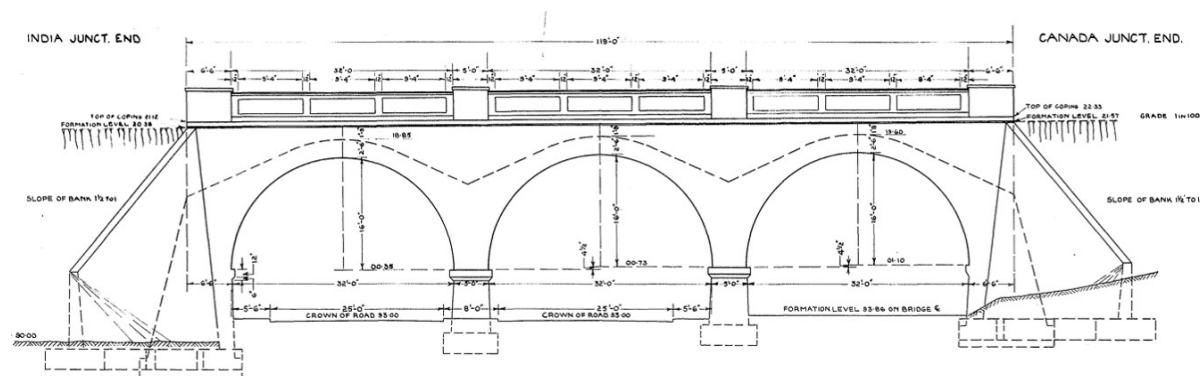


Figure 4-2: Elevation view of RC railway bridge with three 32 feet (9.7 metres) RC arches located on Treu Road and Crown Mines Spur, in Johannesburg (BE-4892_SHT_1, 1946)

The construction drawings were signed in 1946, which indicated the year when the RC railway bridge was likely constructed. Thus, the structure is approximately 78 years old. The construction drawing also provided details of the concrete mixture used, as seen in Table 4-1.

Table 4-1: Details of concrete mixture proportions used in the construction of RC railway bridge (BE-4892_SHT_1, 1946)

Location of concrete to be cast	Mix design
Concrete in Arch ring	1:2:4 (1.5 inch/38 mm stone)
Concrete in Haunches	1:2:4 (2.5 inch/63mm stone)
Concrete in Abutments and Piers	1:3:5 (2.5 inch/63mm stone)

Table 4-1 shows that Engineers specified a concrete mixture design using the ratio approach, which involved combining cement, sand, and stone in a volume ratio to construct the RC railway bridge. Water was typically added to the mixture until workability, judged by the eye, was satisfactory. The concrete mixture design proportions for the arch rings and haunches were driven primarily by the specified target strength of 20 MPa required of the concrete, as seen in Table 4-2. While the concrete mixture design proportions for the Abutments and Piers were driven by the specified target strength of 15 MPa

Table 4-2: Extract from British Standard Code of Practice CP 114 "The structural use of nominal reinforced concrete in buildings (British Standard Institute, 1969)

Concrete Strength (MPa)	Concrete mixture ratio (cement:sand:stone)
15	1:3:6
20	1:2:4
25	1:1.5:3
30	1:1:2

Furthermore, it was noted from the construction drawings that a general reinforcement cover of 1.5 inches (38 mm) was used for primary reinforcement. A total of 3 196 cubic yards (2 443 cubic meters) of concrete and 28 tonnes of steel was estimated to have been used to construct the RC railway bridge.

4.4 Visual inspection of the RC railway bridge

Figure 4-3 and Figure 4-4 provide a general overview of the RC railway bridge.



Figure 4-3: North-facing view of RC railway Bridge on Treu Road and Crown Mines Spur



Figure 4-4: South-facing view of RC railway Bridge on Treu Road and Crown Mines Spur

Three structural elements of the RC railway bridge were selected to be visually examined: the abutments, the piers, and the arches. This resulted in thirteen different elements being inspected

across the entire railway bridge. The inspection was limited to the concrete surface of the RC railway bridge, which was visually accessible.

The detailed report of the visual inspection of the RC railway bridge is located in Appendix B, this includes photographs and descriptions noted during the visual inspection.

An extract of the detailed visual report is presented, focusing specifically on the interior wall of Pier 1. Photographs of the element under consideration, accompanied by detailed notes addressing exposure, drainage, and the surface condition of the concrete, are noted.

Element: Pier 1 – Interior Wall



Figure 11: Pier 1 – Interior wall, facing eastwards



Figure 12: Pier 1 – Interior wall, facing westwards

Exposure: Sheltered, outer edges of wall exposed to rain

Drainage outlets: None

Surface condition of concrete:

- Extensive map cracking on the surface, primarily above the piers construction joint
- Random areas of dampness and rust straining present along the interior wall
- Continuous line of dampness running along the construction joint between the haunch and arch section
- Lower surface subject to graffiti, painted advertisements and posters

In summary, the results of the visual inspection indicated that globally, there was extensive map-cracking present on all thirteen railway bridge elements examined, revealing signs of distress in the bridge (Wan, Foley and Komp, 2010; Thomas, Fournier and Folliard, 2013; Olov Myklebust, Johansen and Hans Stemland, 2018). Elements “sheltered” from exposure to direct moisture (rain) also exhibited extensive map cracking, such as the underside of three arch surfaces, as seen in Figure 4-5 and Figure 4-6.



Figure 4-5: Evidence of map-cracking on the South facing exterior of RC railway bridge



Figure 4-6: Extensive map-cracking on the underside of “sheltered” arch surfaces

Furthermore, with regard to the surface condition, there was evidence of discolouration associated with dampness and rust staining that originated from various construction joints. In some instances, there was evidence of whitish gel exudations around cracks. It was also noted

that the entire exterior surface of the structure was coated with a grey coating, as seen in Figure 4-7. No reports were found that explained when and why the surface coating (type unknown) was applied to the entire structure surface. Lastly, concerning the overall apparent alignment of the structure, there were no signs of settlement, deflection or displacement of any structural elements.



Figure 4-7: A greyish coating is seen on the exterior concrete surface of the entire RC railway bridge.

Based on the observations noted in the visual assessment report, each of the thirteen elements was evaluated for the probability of being subject to ASR deterioration. Table 4-3 summarises all thirteen elements inspected. It incorporates six features typically associated with ASR deterioration, accompanied by descriptions of the severity of the feature as the deterioration mechanism progresses over time. Based on the visual assessment observations, the features most applicable to the RC railway bridge were noted in the appropriate column.

Four of the six features in the table fall under the medium and high probability category, indicating that the RC railway bridge is likely subject to ASR deterioration. The outcome of the visual inspection provided sufficient motivation for concrete cores to be extracted from the structure to confirm the diagnosis through laboratory testing.

Table 4-3:Global summary of RC railway bridge assessment used to determine the probability of the presence of ASR based on visual inspection results

<i>Feature</i>	<i>Probability of AAR</i>		
	<i>Low</i>	<i>Medium</i>	<i>High</i>
i Cracking & crack pattern	-	-	Extensive map-cracking or cracking aligned with primary reinforcement or stress; misalignment of facings (differential expansion on each side of the crack)
ii Deformation and displacement of elements	None	-	-
iii Surface discoloration	-	Slight surface discolouration associated with some cracks	-
iv Exudations	-	White exudations around cracks, possibility of colourless jelly-like substance	-
v Pop-outs with ASR gel deposit	None	-	-
vi Environment	-	-	Parts of components frequently exposed to moisture, e.g., rain and groundwater, water due to the natural function of the structure

4.5 Assessment of as-received cores

Twelve horizontal concrete cores were extracted from various locations of the railway bridge, which displayed varying degrees of visual distress, as shown in Figure 4-8, Figure 4-9, Figure 4-10 and Figure 4-11.



Figure 4-8:Location where cores C1, C2, C3 and C4 were extracted from Abutment 1- inner wall



Figure 4-9: Location where cores C5 and C6 were extracted from Pier 1 – North-facing



Figure 4-10: Location where cores C7, C8, C9 and C10 were extracted from Pier 1 – Interior wall, facing eastwards



Figure 4-11: Location where cores C11 and C12 were extracted from Pier 1 – South-facing

Table 4-4 summarises the extent of visual distress observed on the RC railway bridge and the respective concrete cores extracted from the various locations.

Table 4-4: Location of twelve concrete cores extracted from the RC railway bridge based on varying degrees of visual distress

Area characterisation based on visual assessment	Core ID	Location on Railway bridge
Slightly damaged	C7, C8, C9, C10	Pier 1 - Inner wall West-facing side
Moderately damaged	C1, C2, C3, C4	Abutment 1 - Inner wall
Severely damaged	C5, C6	Pier 1 - North-facing
Severely damaged	C11, C12	Pier 1 – South-facing

It can be seen that the slightly damaged area categorised in Table 4-4 exhibits signs of map cracking, indicating distress. This decision was based on the relative extent of map-cracking compared to the other areas and the concrete's surface condition and moisture exposure.

The detailed log of the as-received cores' visual assessment can be found in Appendix C (this includes descriptions captured during the visual inspection of the extracted concrete core). It was observed that the concrete mixture used consisted mainly of quartzite and shale aggregates (Figure 4-12), as expected from the underlying Johannesburg geology, with the largest aggregate size being approximately 50 mm. Regarding macro-features observed, cracking was more pronounced in the coarse aggregates than in the cement paste. There was also evidence of debonded aggregates in various concrete cores. There were no visible signs of gel exuding from the aggregate cracks, cement paste or surrounding the aggregate particles. This result may be attributed to the gel being washed out when the cores were extracted from the structure via the coring process, as water was used as a lubricant



Figure 4-12: Image of a piece of Core 6B illustrating quartzite and shale coarse aggregates used in the construction of RC railway bridge

4.6 2D images of the circumferential concrete surface & modified DRI

Figure 4-13 and Figure 4-14 show examples of the 2D image of the circumferential surface produced from the slightly damaged area (concrete core C10) and the severely damaged area (concrete core C6). The 2D images of the circumferential surfaces of the other ten concrete cores are presented in Appendix D - 2D global images of cores' cylindrical surface. The zero distance from the concrete exterior surface refers to the reference exterior concrete's surface as seen on the RC structures. The modified DRI approach was applied to the 2D images of the circumferential surface of each concrete core; examples of certain petrographic features noted in the analysis are shown in Figure 4-15.

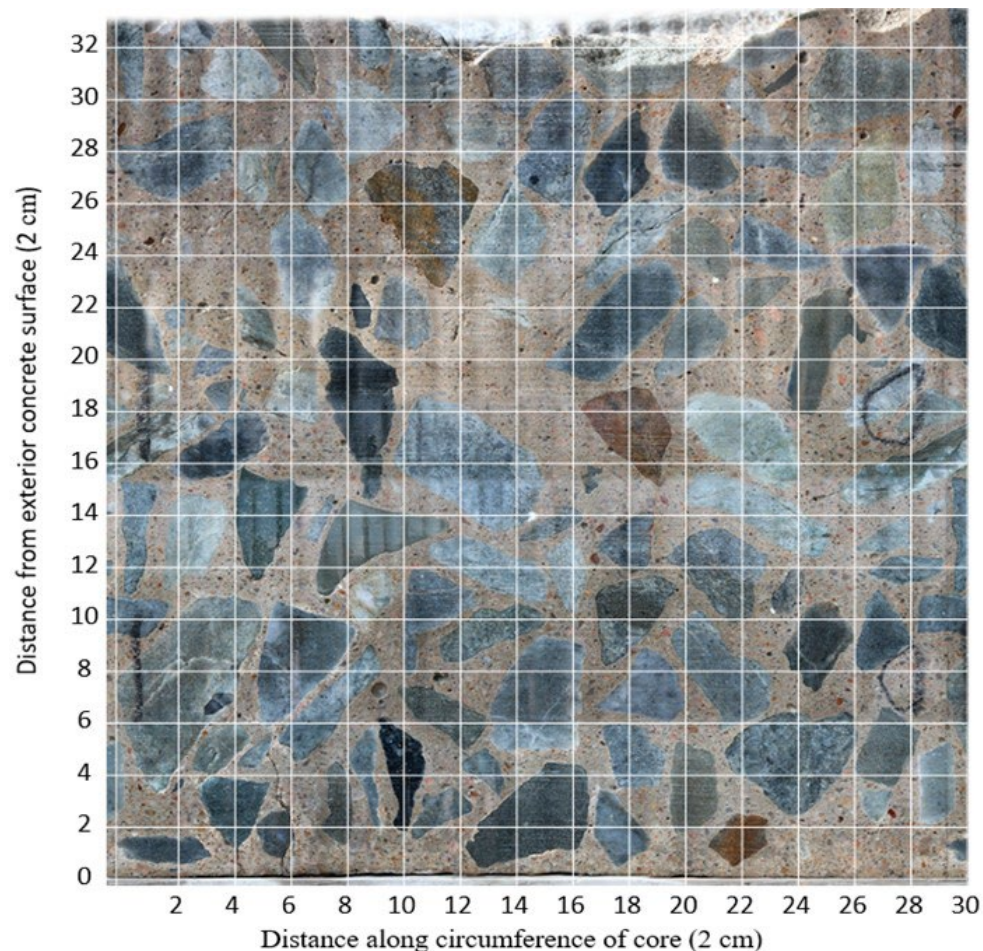


Figure 4-13: 2D global image of core C10 cylindrical surface, overlaid with 2 cm grid

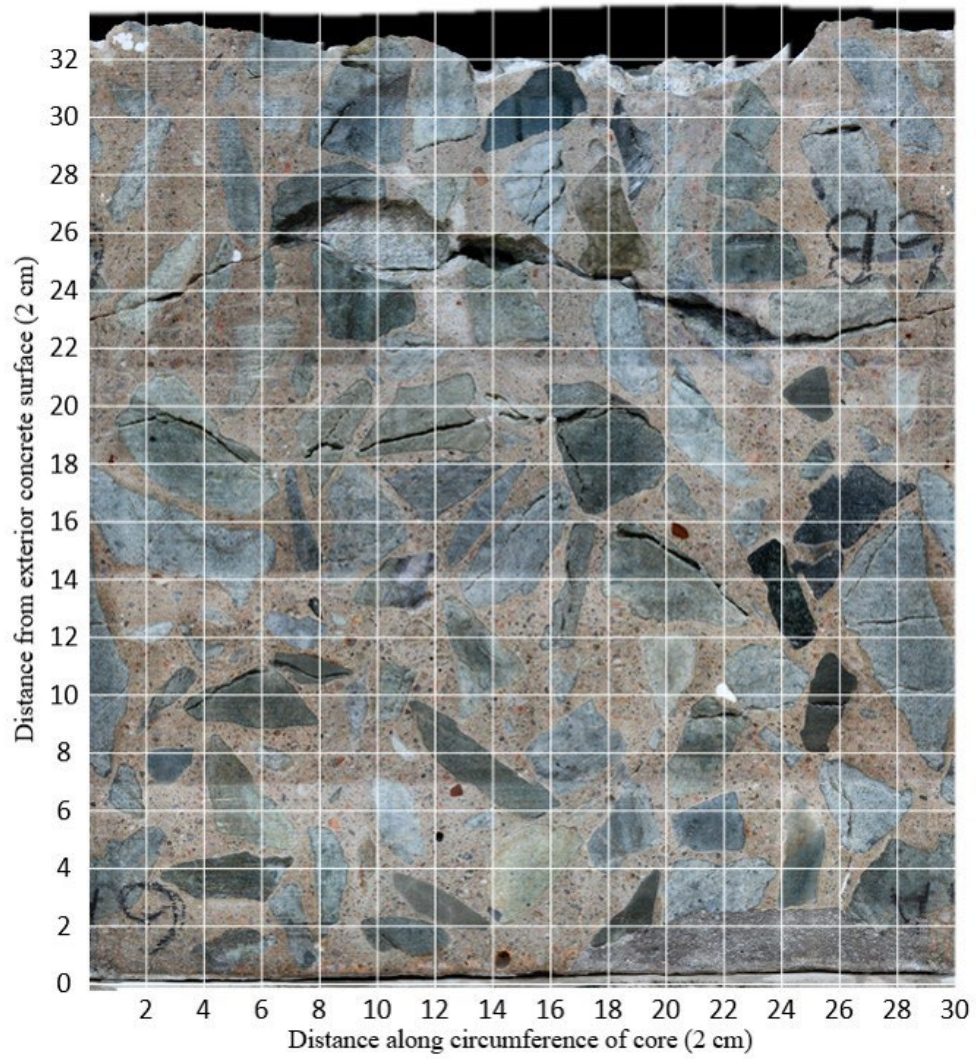


Figure 4-14: 2D global image of core C6 cylindrical surface, overlaid with 2 cm grid

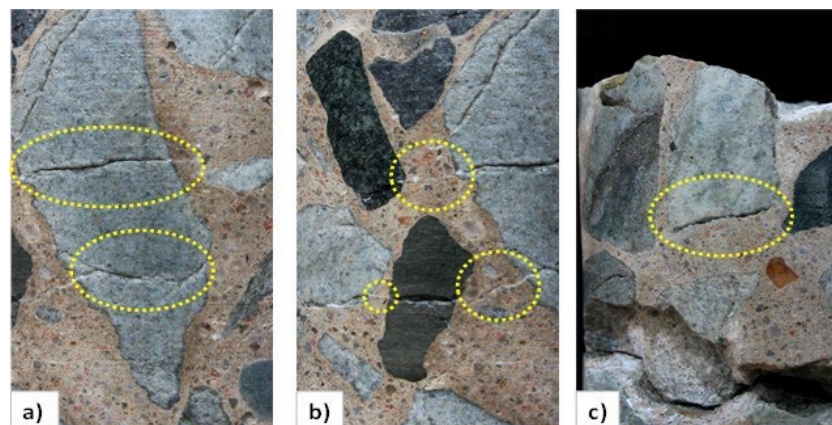


Figure 4-15: Images illustrating examples of concrete petrographic features: a) cracks present in the coarse aggregate, b) cracks present in the cement paste, c) de-bonded coarse aggregate

The petrographic values were then normalised to a surface area of 100 cm² (as the length of the concrete cores varied; thus, the total area examined varied for each concrete core) and plotted (no weighting factors were included at this stage), as seen in Figure 4-16.

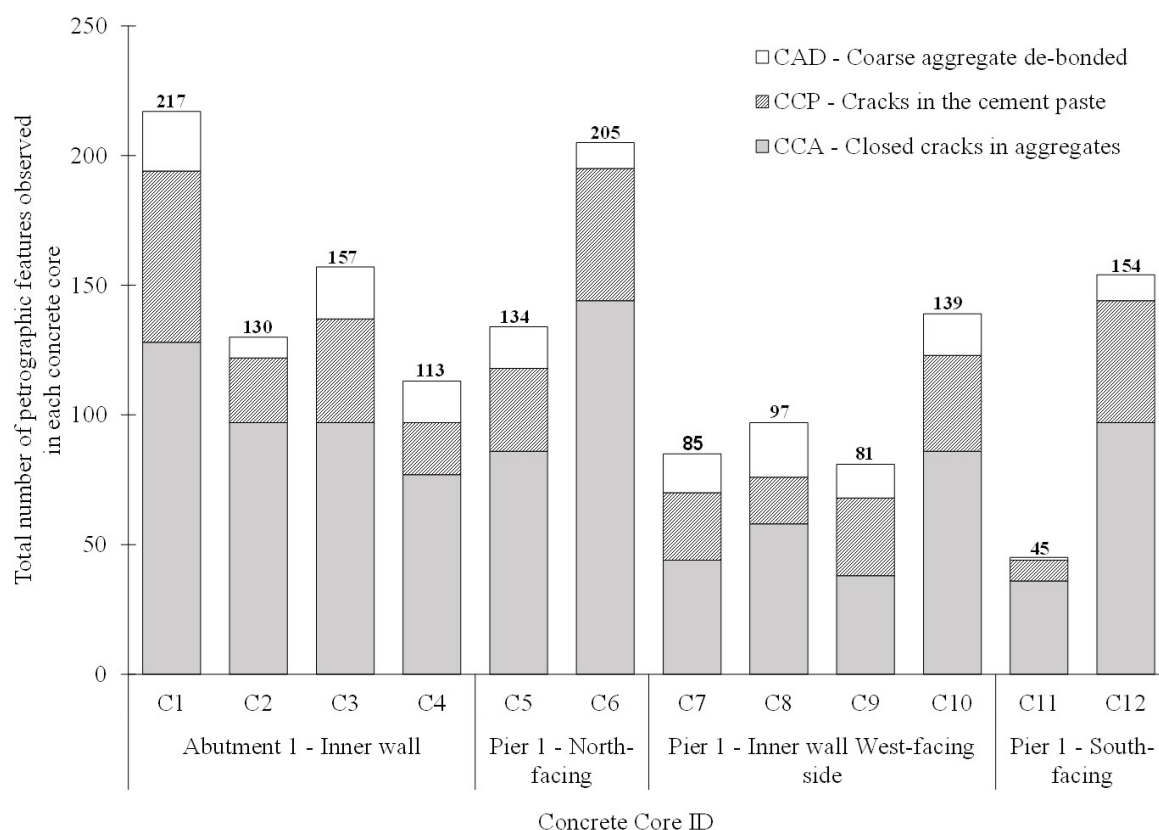


Figure 4-16: Number of petrographic features observed in 2D global images of each of the twelve concrete cylindrical surfaces after being normalised to 100 cm² area

In general, across all twelve concrete cores analyzed, the predominant petrographic features were closed cracks within the coarse aggregate (CCA), followed by cracks in the cement paste (CCP) and debonded coarse aggregates (CAD). The prevalence of these features is likely influenced by the limited sensitivity of the modified DRI approach, which relies on naked-eye visual assessment. This approach inherently favours the identification of more conspicuous, macroscopic features, while potentially underrepresenting finer-scale microstructural damage.

Additionally, the coring process itself may have contributed to the initiation or propagation of cracks, particularly due to the stiffness contrast between the coarse aggregate and the surrounding cement paste. Such mechanically induced cracking would be difficult to

distinguish from cracking caused by alkali–silica reaction (ASR), potentially confounding the interpretation of damage mechanisms.

It can be seen that there is an agreement that a higher number of petrographic features noted correspond with areas exhibiting higher damage (Pier 1, North and South), and low DRI values reflect areas with comparatively little damage (Abutment 1 inner wall and Pier 1 – inner wall). The extent of damage of core C11, located on Pier 1’s outer wall, was noted as an outlier as the core exhibited the lowest number of petrographic features with a value of 9 amongst all twelve cores analysed.

Core C1 had the highest number of petrographic features with a value of 217, followed by core C6 and C12 with a value of 205 and 154, respectively. The extent of damage apparent in Core C1 appears inconsistent in the set of results, as the value was the highest and not comparable to Core C2, which was extracted from the same wall but adjacent to the expansion joint, as seen in Figure 4-17. The possible cause of severe damage in core C1 may be attributed to moisture availability sweeping from the construction joint. Furthermore, the result can point to a variation in the composition of concrete used in casting that element of the railway bridge. Concrete extracted from various locations from the inner wall of Abutment 1, cores C2, C3 and C4 exhibited a comparable number of petrographic features noted with values of 130, 157 and 113, respectively.



Figure 4-17: Location of core C1 in relation to location of C2 from Abutment 1- inner wall

Core C6 and C12 were anticipated to have a high number of petrographic features with a value of 134 and 205, respectively, as the cores were extracted from Pier 1's outer walls, which are directly exposed to rain. The possible reason that the extent of damage observed in Core C12 was lower than that of Core C6 may be attributed to the steel pipe gantry running alongside the South-facing side of the railway bridge, which limits the concrete exposure to direct moisture.

Concrete extracted from various locations on Pier 1's inner wall, cores C7, C8, C9 and C10 had several features noted of 85, 97, 81 and 139, respectively. Core C10 appears inconsistent in the set of results, as the value was the highest and not comparable to Core C9, which was extracted from the same wall. The possible cause of increased damage in core C10 is moisture availability, as it was located closer to the exterior of Pier 1's inner wall.

Petrographic weighting factors presented in Table 2-10 were then applied to the results shown in Figure 4-16. Each core's damage rating index value was then determined and plotted, as seen in Figure 4-18.

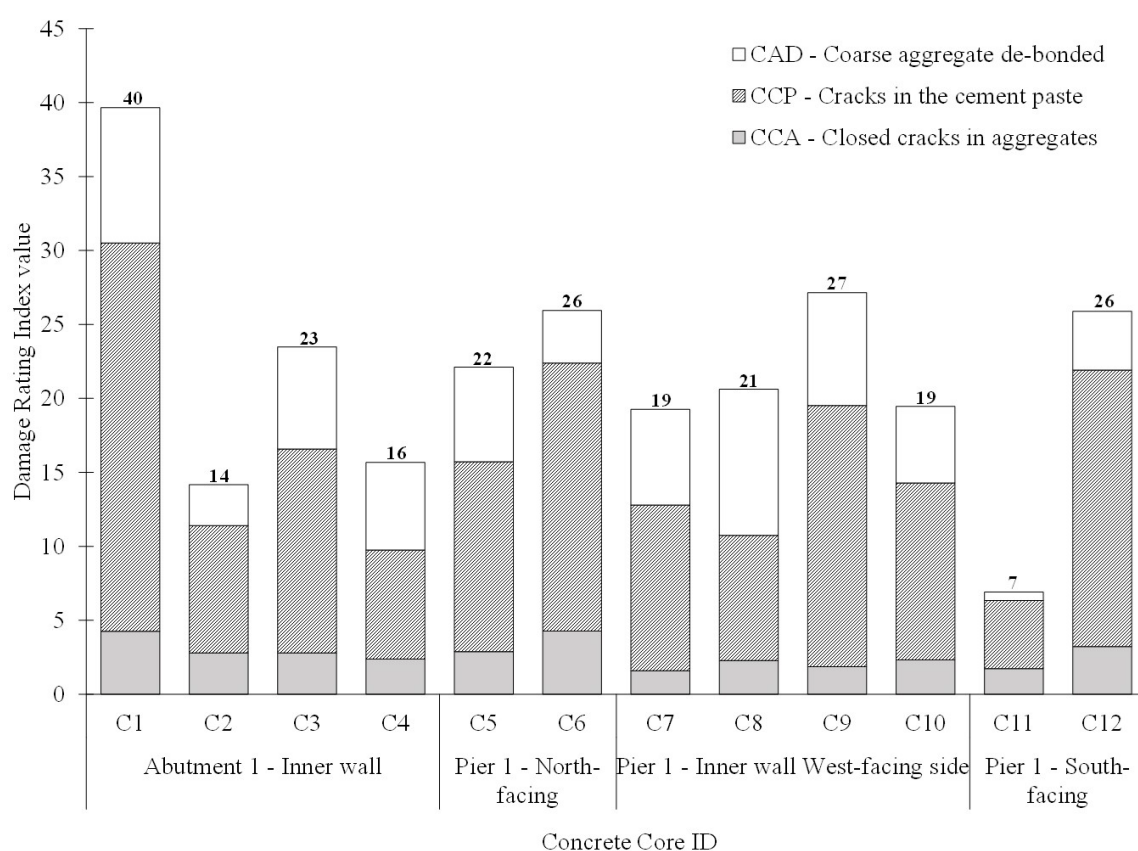


Figure 4-18: Damage rating index for all twelve cores examined

Figure 4-18concrete area closer to the concrete exterior surface was analysed for ASR petrographic features. In addition, most of the ASR petrographic features were noted further

into the bulk of the concrete, which could not be observed and quantified in the case of core C11. Notably, Core C1 presents an anomaly with the highest DRI of 40 among all cores. In regions identified as "slightly damaged concrete" (C7, C8, C9, and C10), DRI values are 19, 21, 27, and 19, respectively. Conversely, in areas categorised as "moderately damaged concrete" (C1, C2, C3, and C4), DRI values are 40, 14, 23, and 16, respectively. For the "severely damaged concrete" areas (C5, C6, and C12), DRI values are 22, 26, and 26, respectively.

Surprisingly, the analysis does not establish a consistent correlation between higher DRI values and more extensive damage in areas like Pier 1 (North and South). Conversely, lower DRI values do not consistently indicate areas with minimal damage, as observed in Abutment 1's inner and Pier 1's inner walls. These findings contradict the trends presented in the literature, as shown in Section 2.11.2 (Macdonald, Piersanti and Magnan, 2019; Sanchez, Drimalas and Fournier, 2020; Ahmed, Zahedi and Sanchez, 2022). This suggests that the modified DRI approach indicated a shared form of internal distress owing to ASR across concrete specimens from various locations, irrespective of the observed damage levels. This highlights the approach's limitations as it was ineffective at distinguishing concrete extracted between the visibly slightly damaged and severely damaged areas. This may be attributed to the increased magnification and grid size; thus, the approach is not sensitive enough to pick up all the respective petrographic features in the concrete.

The 2 cm grids of the 2D global images were then colour-coded to generate "heat maps" denoting the various petrographic features in each block, as seen in Figure 4-19. The horizontal axis is associated with the distance along the circumference of the concrete core, while the vertical axis is associated with the concrete core's length. The following colour legend was used: demonstrates that owing to the higher weighting factor assigned to cracks in the cement paste of 3, the CCP portion is the largest aspect contributing to the DRI values of all twelve concrete cores. Despite the high number of cracks observed in the cores' aggregates, the weighting factor is only 0.25, thus the lower contribution to the DRI value calculated. Analysis of concrete specimens from different damaged areas reveals a range of Damage Index (DRI) values ranging from 14 to 40. Core C11 was excluded from the analysis as only a short length of the core, 189 mm, was obtained from the structure. Subsequently, a significantly smaller

- Yellow – Cracks in aggregates
- Orange – Cracks in cement paste

- Pink – Coarse aggregate de-bonded
- Red – more than one petrographic feature present
- White – no petrographic features

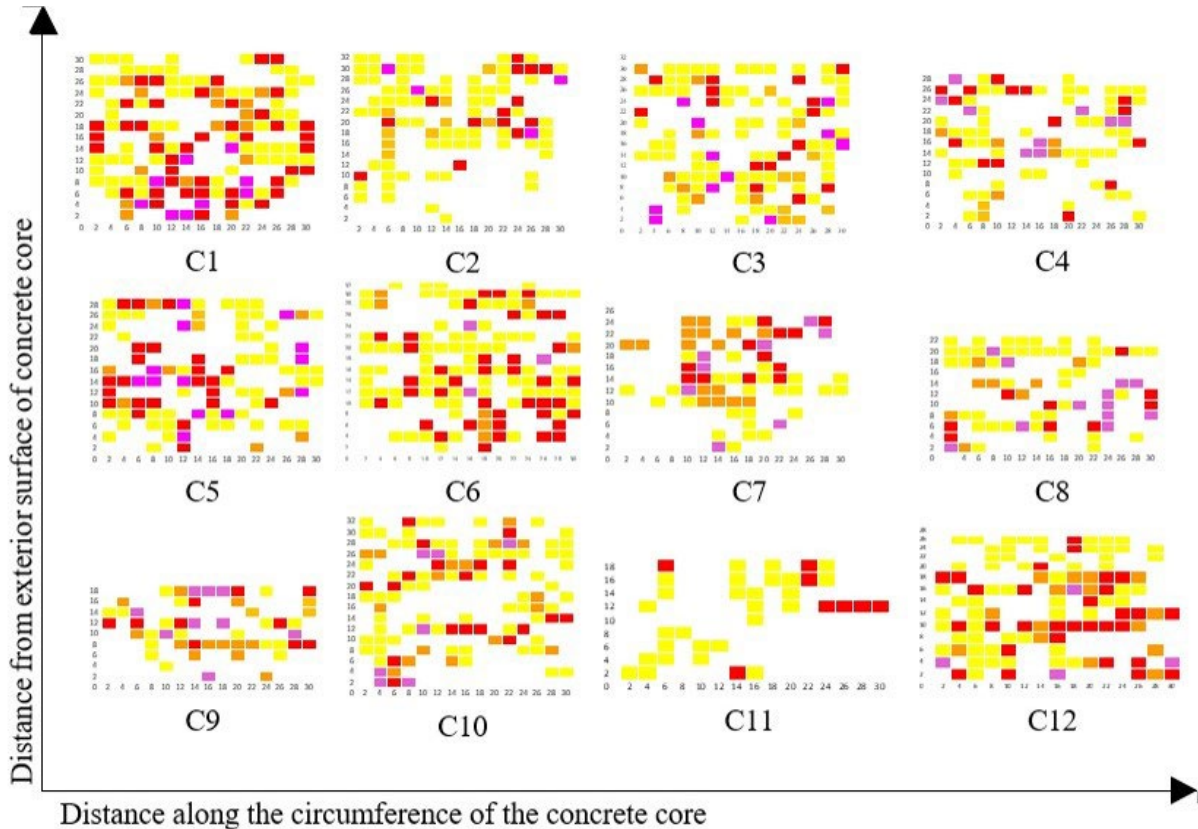


Figure 4-19: Heat maps' illustrating various petrographic features observed on all twelve cylindrical surfaces

In general, the nature of interconnectedness of damage in the concrete cores can be observed based on the scale of the 2 cm square grid. Concrete extracted from various locations on Pier 1's inner wall, cores C7, C8, C9 and C10, which were taken from a slightly damaged area, indicate that the extent of damage increases into the depth of the concrete. This result suggests that moisture may be trapped in the region of concrete furthest from the exterior surface or draining from the soil embankment above the arches, facilitating the damage owing to ASR deterioration.

The number of petrographic features along the length of each concrete core in 100 mm increments was analysed, as shown in Figure 4-20. A general trend that can be observed is that the number of petrographic features is not constant along the length of the concrete cores. Only one of the twelve cores (C12) exhibited the highest amount of petrographic features closest to the concrete exterior surface. This possible result may be attributed to the extent of damage

observed in Core C12's exterior concrete surface and subsequent moisture availability. While six of the twelve cores (C1, C5, C6, C8, C9 and C11) exhibited the highest petrographic features in the 100 mm - 200 mm depth. The remaining five of the twelve cores (C2, C3, C4, C7 and C10) exhibited the most petrographic features much further into the depth of concrete cores from the 200 mm to 300 mm band. This result may indirectly indicate that moisture availability varied along the core's depth, facilitating varying degrees of damage along the depth of the concrete cores. The potential for moisture being “trapped” in depths ranging from 100 mm to 300 mm, leading to increased degrees of damage warrants further investigation.

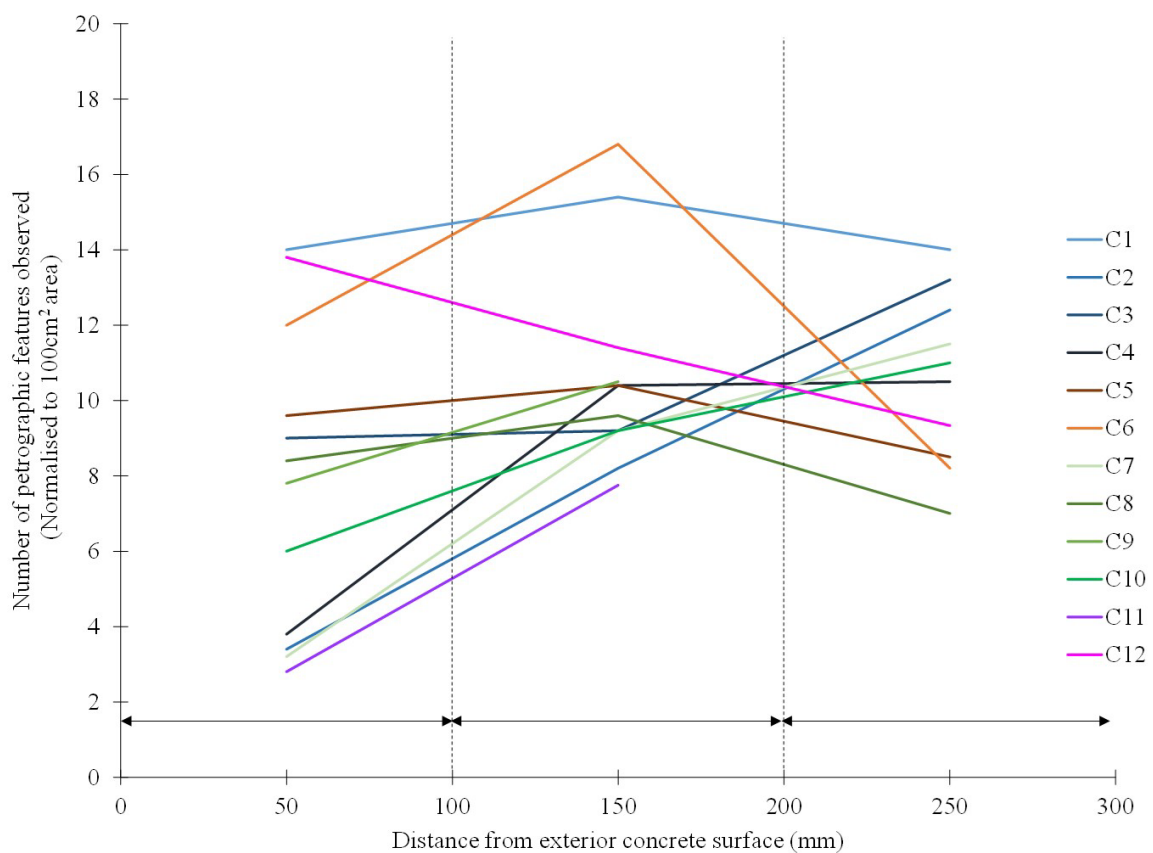


Figure 4-20: Average number of petrographic features along the depth of the core recorded in 100 mm increments

In addition, it is known that physical restraint through applied stress or reinforcement can significantly reduce expansion and subsequent damage due to ASR in the direction of restraint (Durand, 2000). Thus, the number of petrographic features on the outer surface of each concrete core was analysed to determine if there was any correlation between the orientation of the concrete core as extracted from the RC structure and the number of petrographic features observed. Figure 4-21 illustrates the notation that was used to relate the orientation of the core

from the structure when extracted to the circumference of the concrete core. Note that zero degrees is at the vertical top of the core, and angles are measured positively clockwise.

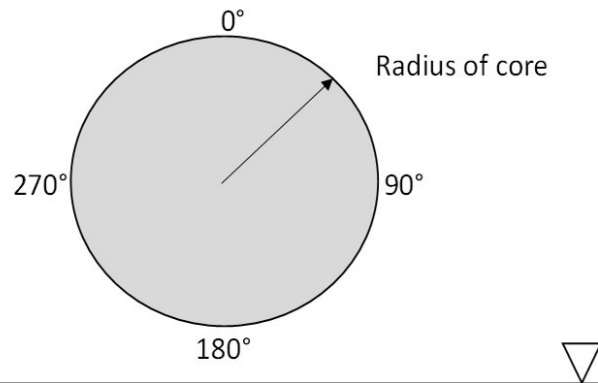


Figure 4-21: The relationship between the orientation of the core as extracted from the structure and the circumference of the concrete core

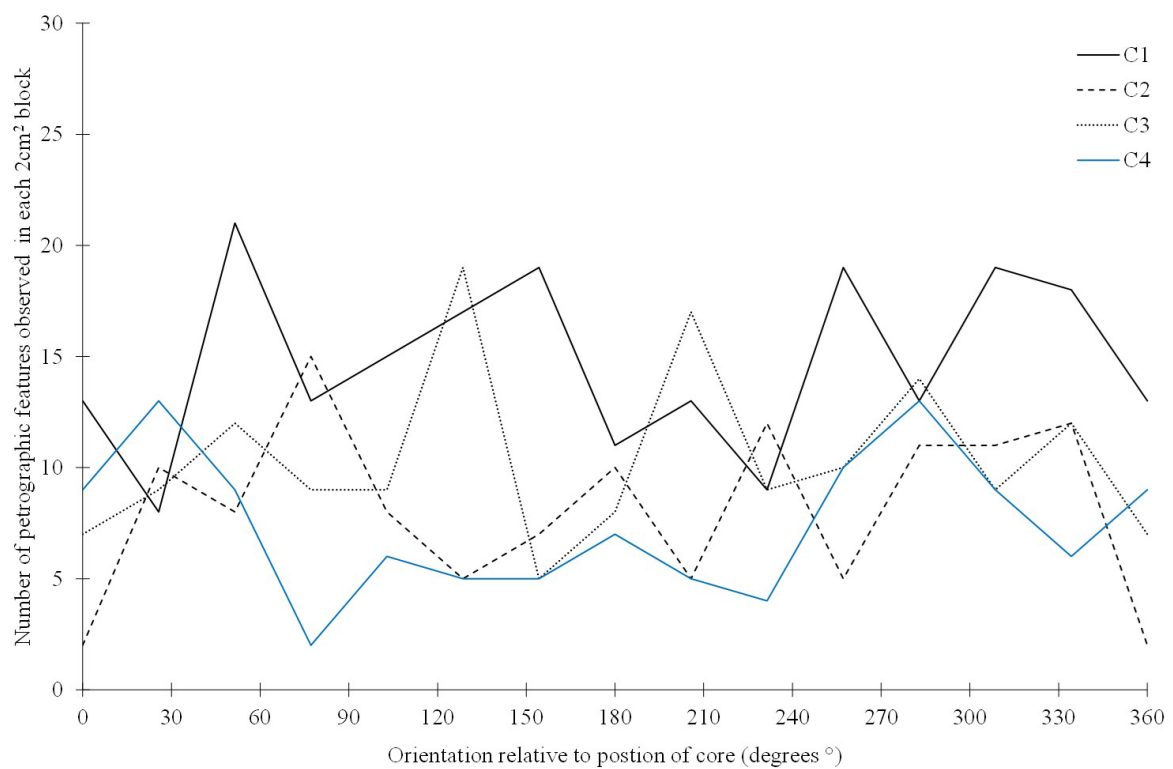


Figure 4-22: Abutment 1 - Inner wall, number of petrographic features along the circumference of each concrete core

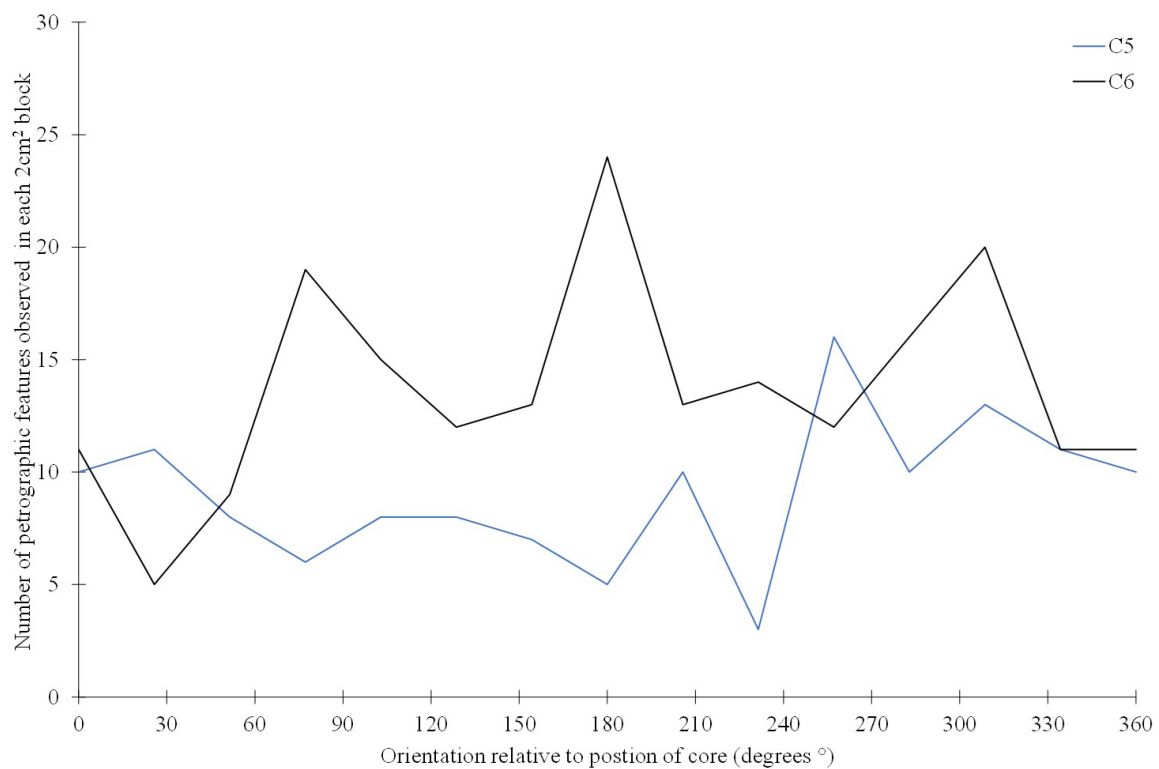


Figure 4-23: North-facing view, number of petrographic features along the circumference of each concrete core

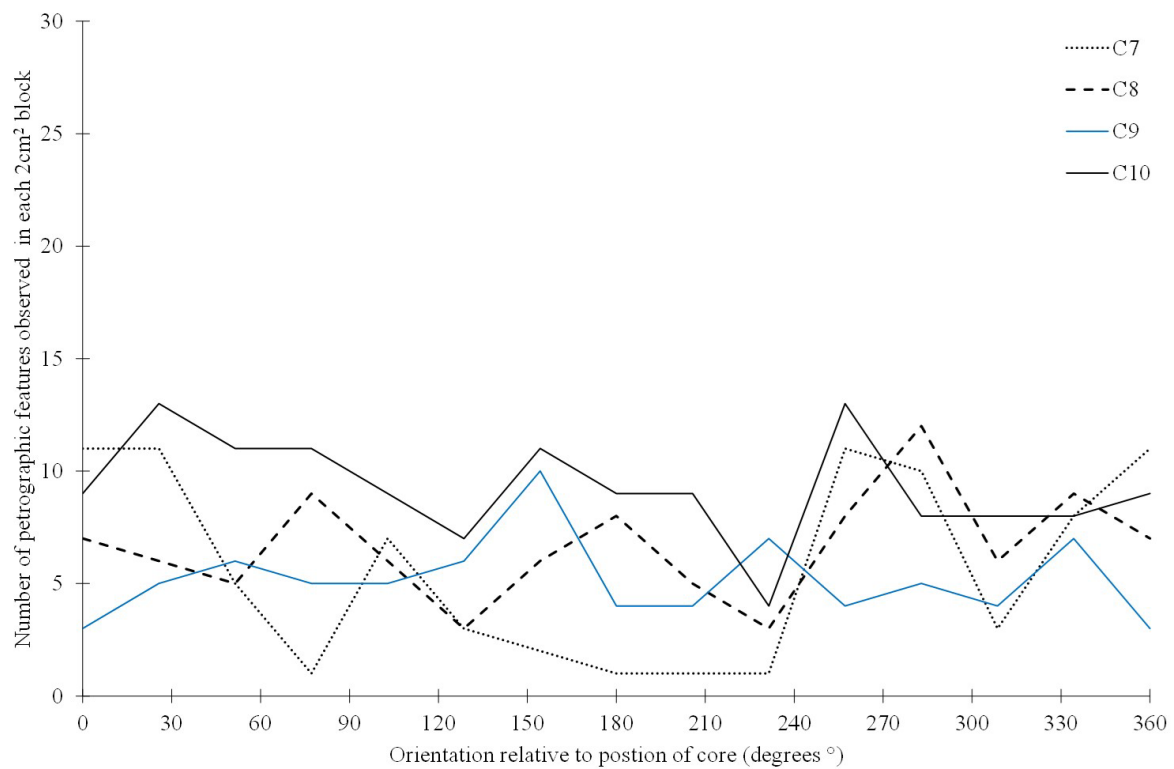


Figure 4-24: Pier 1 - Inner wall West-facing side, number of petrographic features along the circumference of each concrete core

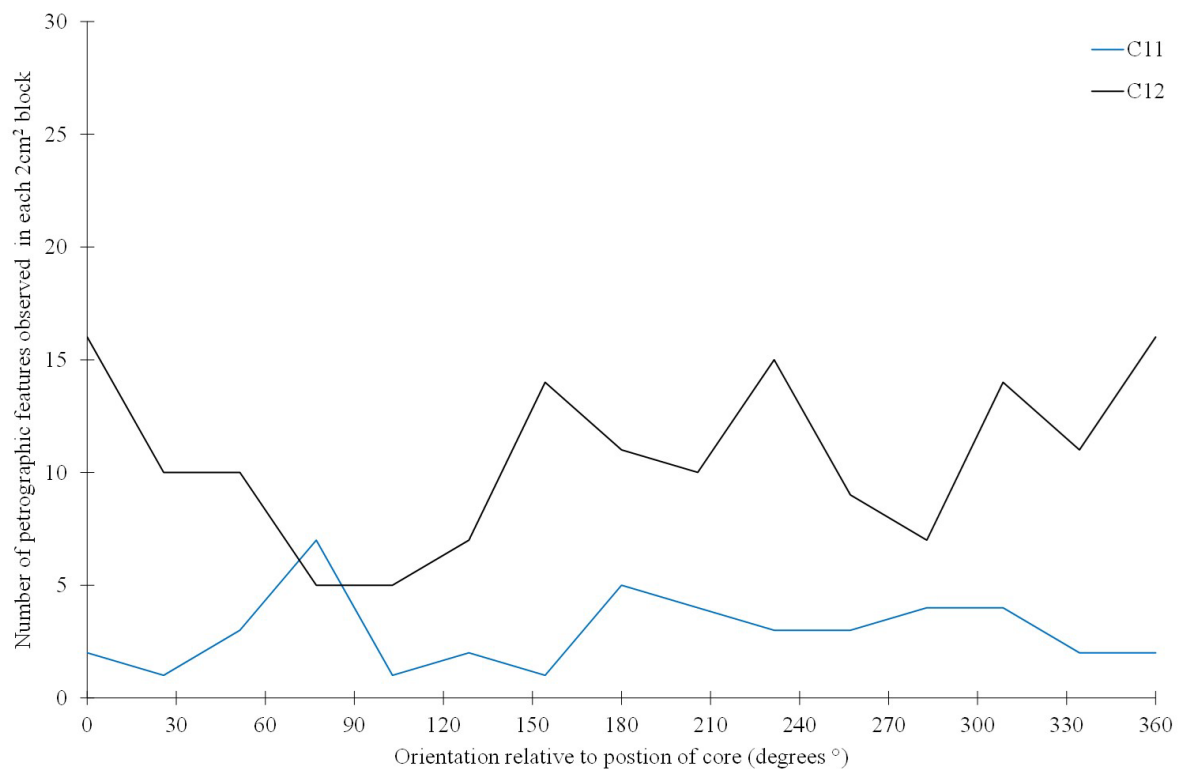


Figure 4-25: Pier 1 - South-facing view, number of petrographic features along the circumference of each concrete core

In Figure 4-24, concrete cores extracted from a slightly damaged area (C7, C8, C9 and C10) exhibit relatively constant damage relative to the orientation of the core removed from the RC railway bridge. However, Figure 4-22, Figure 4-23 and Figure 4-25 show that the extent of damage relative to the orientation of the core was cyclic. In Figure 4-22 concrete from cores C1, C2, C3 and C4, all extracted from Abutment 1's – inner wall, displayed varying degrees of damage. However, a general trend can be observed with respect to the number of petrographic features along the circumference of each concrete core. The number of damaged features at the top (0°) and bottom (180°) of the core was lower in comparison to the positions in-between, e.g. 30° - 150° and 210° - 330° .

The trend's nature may be attributed to the stress the material is experiencing in the structure due to loading (self-weight and imposed). RC arch bridges are designed to transfer load acting on the bridge to the foundation primarily through compression. Thus, the material aligned with the direction of the compression load is less likely to expand and crack due to the restraint imposed by the compressive loads. The same material is expected to expand and crack in the direction perpendicular to the compression load as the material is subject to less restraint, as

shown in Figure 4-26 by red arrows. Furthermore, the effect of de-stressing may have occurred during the process of coring the concrete from the structure. Thus allowing any potentially closed cracks in the concrete in the direction of loading to potentially open.

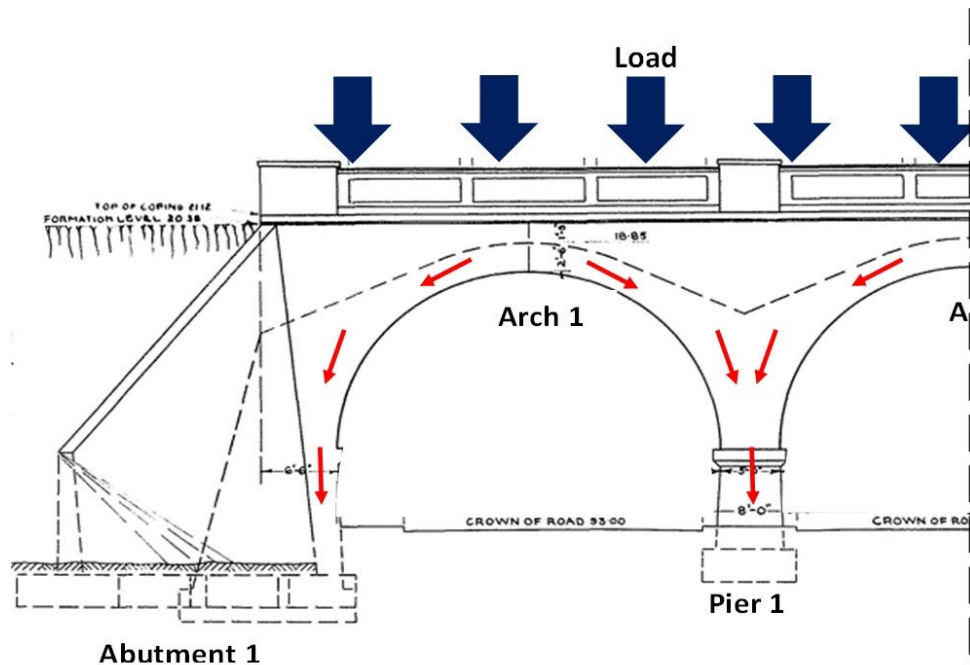


Figure 4-26: Forces acting on the RC railway bridge situated on Treu Road and Crown Mines Spur

This phenomenon was further confirmed when it was noted that the orientation of the cracks in the concrete cores was predominantly parallel to the exterior surface of the concrete, as seen in Figure 4-27 and subsequently, parallel to the direction of compression loading in the RC structure.

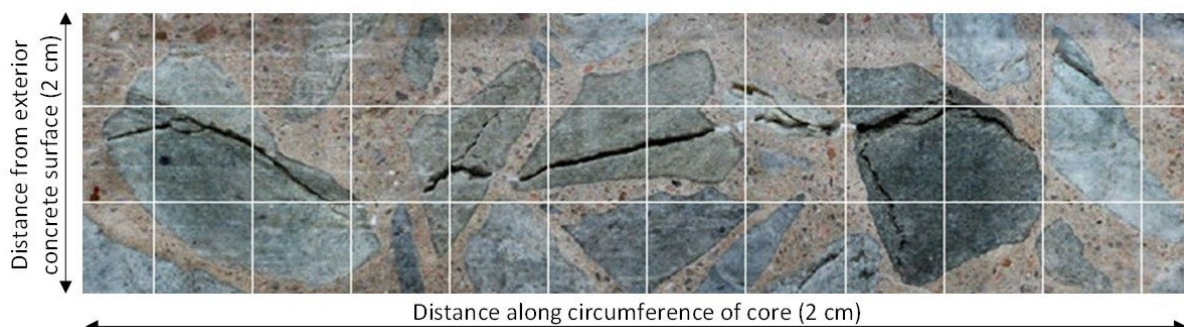


Figure 4-27: Snapshot of Core C6 2D global cylindrical surface illustration of the predominant orientation of the cracks

The orientation of the cracking can be attributed to the lower restraint of the concrete material parallel to the exterior surface of the concrete, thus allowing the material to expand and develop cracks predominantly in this direction. The observation noted agrees with work carried out by Zahedi et al., (2021), who stated that most ASR-induced cracks are perpendicular to the main

restrained direction (i.e. reinforcement direction) in cores extracted perpendicular to the main restraint direction.

4.7 Summary of Phase 1 - Results

In this section, a summary of the findings of Phase 1 is presented, and a number of trends and conclusions are generalised to draw useful information.

- i. A 78-year-old railway bridge in the Johannesburg region that displayed visual distress (in the form of extensive map cracking on the surface) was identified as the field structure subject to ASR deterioration.
- ii. A visual inspection of thirteen elements of the railway bridge showed extensive map cracking, white exudations around cracks, and dampness in certain areas. No signs of structural settlement or displacement were noted.
- iii. Based on the visual inspection observations, the railway bridge presented a medium to high probability of the presence of ASR.
- iv. The visual assessment findings provided a basis for classifying concrete regions of the RC railway bridge as slightly, moderately damaged, and severely damaged.
- v. Twelve concrete cores were extracted and categorised based on the three locations displaying different levels of visual distress.
- vi. For all twelve concrete cores analysed, the majority of the petrographic features observed were closed cracks present in the coarse aggregate (CCA), followed by cracks in the cement paste (CCP) and debonded coarse aggregates (CAD), using 2D global images of the cylindrical surface of the concrete core.
- vii. The modified DRI method used in this study has shown that it is possible to assess the extent of damage to concrete taken from various locations of an RC railway bridge using 2D global images of the cylindrical surface of the concrete core. There was, however, no direct correlation between the concrete cores taken from areas that exhibited various degrees of visual distress and the calculated DRI values. DRI results indicate a shared form of internal distress across concrete specimens from multiple locations, irrespective of the observed damage levels.

- viii. Eleven of the concrete cores exhibited the greatest damage in terms of petrographic features noted along the length of each core, between 100 mm and 300 mm, from the exterior surface of the concrete cores. This result is likely attributed to moisture being 'trapped' in the concrete.
- ix. The extent of damage, with respect to the number of petrographic features along the circumference of each concrete core relative to the orientation of the core, was cyclic. The number of damaged features at the top (0°) and bottom (180°) of the core was lower in comparison to the positions in-between, e.g. 30° - 150° and 210° - 330° .
- x. Cracks observed on the concrete matrix were predominantly orientated parallel to the exterior surface and parallel to the direction of compression loading in the RC railway bridge.

The following section deals with the steps taken to confirm the structure's ASR diagnosis and the results and analysis of the characterisation of concrete taken from the various locations of the railway bridge.

Phase 2 - Results & Discussion

4.8 Petrography

Figure 4-28 combines three images taken under plain polarised light (PPL) and cross-polarised light (XPL) from different coarse aggregate particles contained within the concrete matrix sourced from various locations of the RC railway bridge.

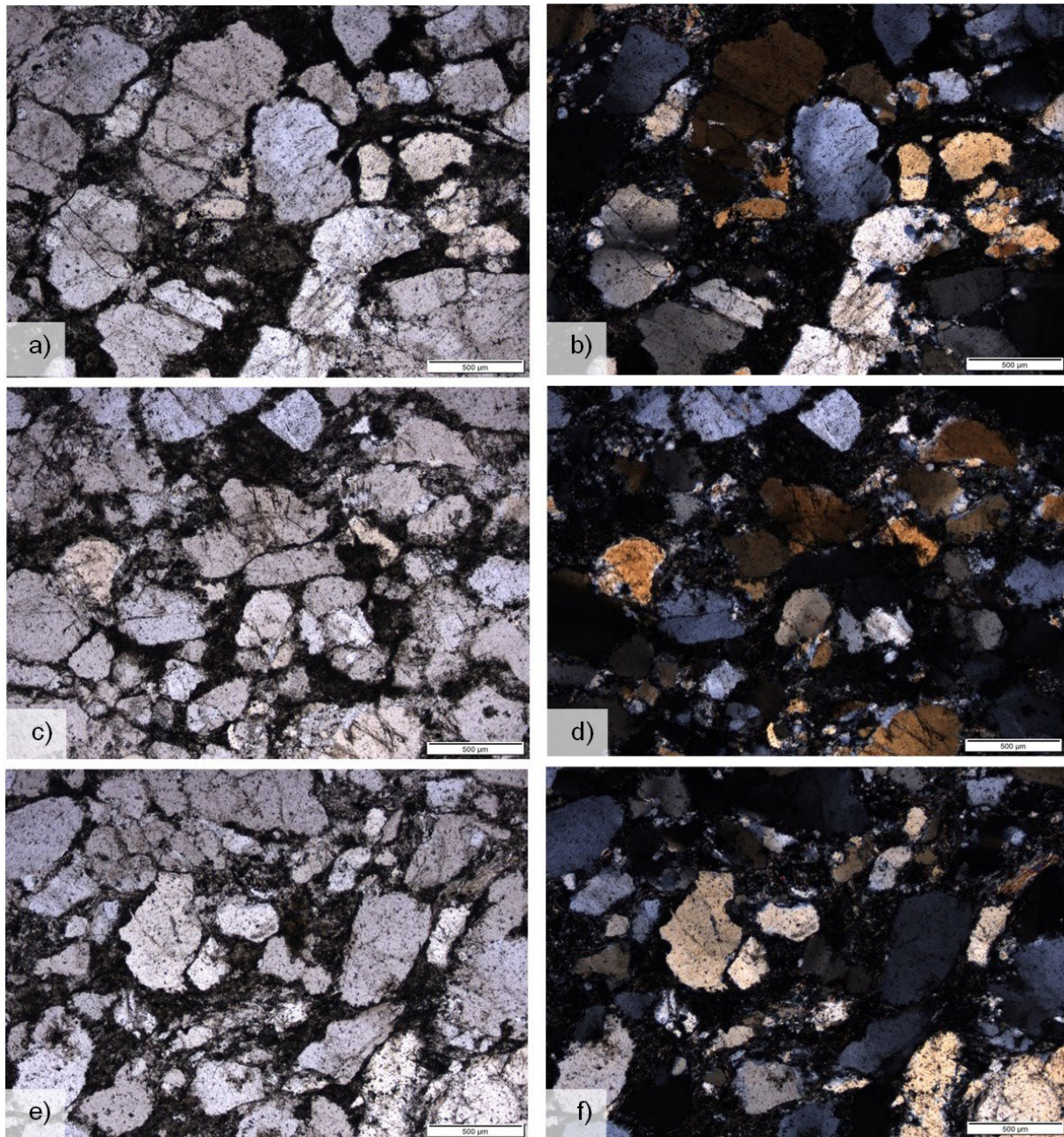


Figure 4-28: Quartzite aggregate particles in concrete matrix extracted from a RC railway bridge situated in Johannesburg, South Africa, under PPL and XPL (Scale bar 500 µm)

Quartz grains were identified by first-order white-grey interference colours and very weak first-order yellow colour as seen under XPL (Figure 4-28b, d and f). The aggregates contain impure

quartzite composed of medium to coarse-grain quartz particles. The coarse grains are angular to sub-rounded and comprise of monocrystalline and polycrystalline particles of quartz. Surrounding the grain boundaries, a fine-grained matrix of clay minerals is evident. In comparison to the reactive quartzite presented in Table 2-9 (Fernandes and Sims, 2016), the quartzite from the RC railway bridge displayed fractures within the quartz grains, denoted by distinctive "scratched" features. In addition, the size of the quartzite grains are smaller. The optical properties of the mineral grains found in the aggregate particles confirm the use of Witwatersrand Quartzite aggregates in the production of concrete for the RC railway bridge under investigation.

The distinction between the aggregate grains being innocuous and potentially reactive depended on the types of silica present (e.g., quartz, opal) and its geological history, causing various textures. Figure 4-29a illustrates a piece of concrete under PPL; there is a clear distinction between the aggregate particle and the cement matrix, as indicated by the inter-transitional zone.

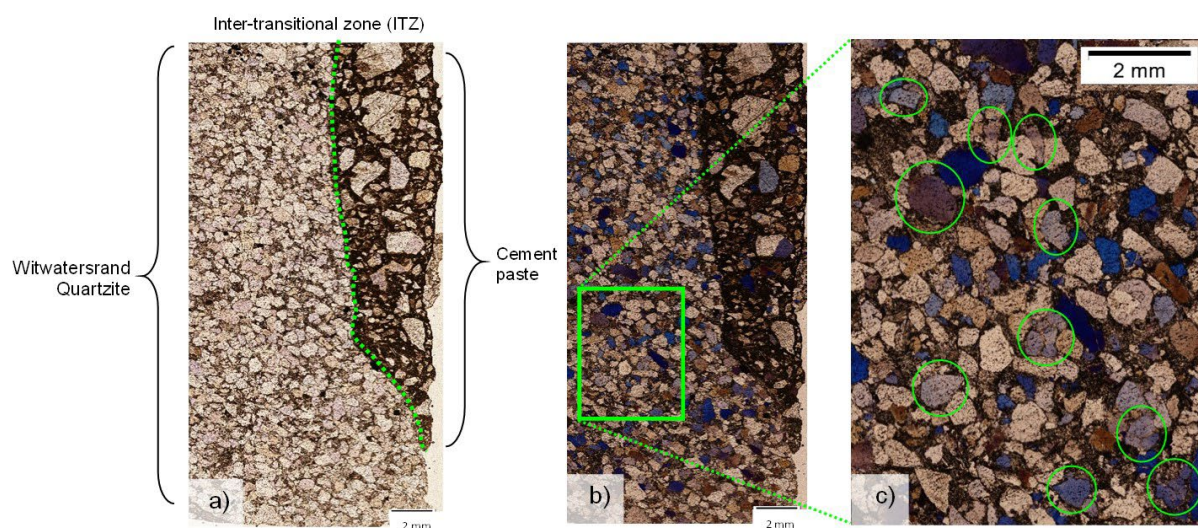


Figure 4-29: a) Concrete thin section examined under PPL (Scale bar 2 mm) b) Concrete thin section examined under XPL (Scale bar 2 mm) c) Quartz grains in the coarse aggregate particle exhibiting undulatory extinction (Scale bar 2 mm)

When examining the thin section under XPL (Figure 4-29b), various quartz grains in the coarse aggregate particle exhibited undulatory extinction, circled in light green in Figure 4-29c. This characteristic results from the strain imposed on the crystal grains during formation. Moreover, as Fernandes and Sims (2016) as well as (Davis Richard Deschenes and Micah Hale, 2018) stated, this property is known to serve as a diagnostic feature for aggregates susceptible to ASR. The exact degree of straining of the Quartz grains remains unknown.

Figure 4-30a highlights various types of cracks present in the concrete matrix under PPL. Firstly, a prominent crack through the coarse aggregate A3 is noted. Furthermore, the development of cracks in the cement paste bridging the cracks formed along the ITZ of A3 and A2. Under XPL, the Quarts grain in Figure 3b exhibits similar undulatory extinction features circled in light green in Figure 4-30c.

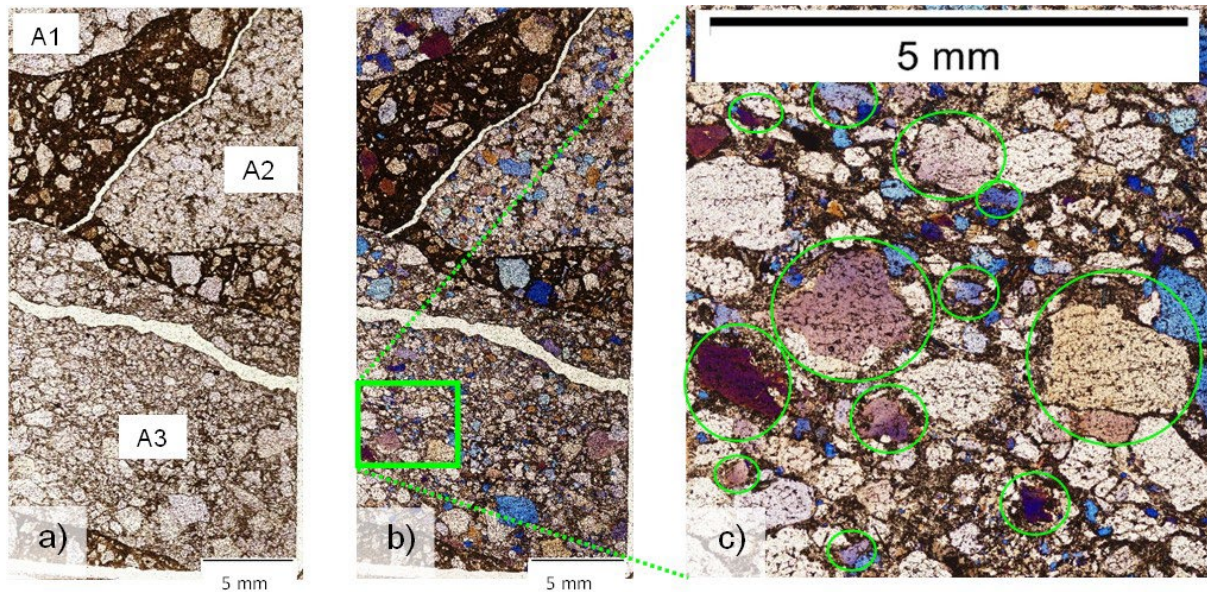


Figure 4-30: a) Thin section examined under PPL displays a concrete matrix containing three coarse aggregates (A1, A2 and A3) (Scale bar 2 mm), b & c) Thin section examined under XPL quartz grains exhibiting undulatory extinction

The same thin section examined previously in Figure 4-30 was used to diagnose the presence of ASR gel in the concrete. Four sub-areas were selected from the thin section image in Figure 4-31a, which shows a selection of crack veins.

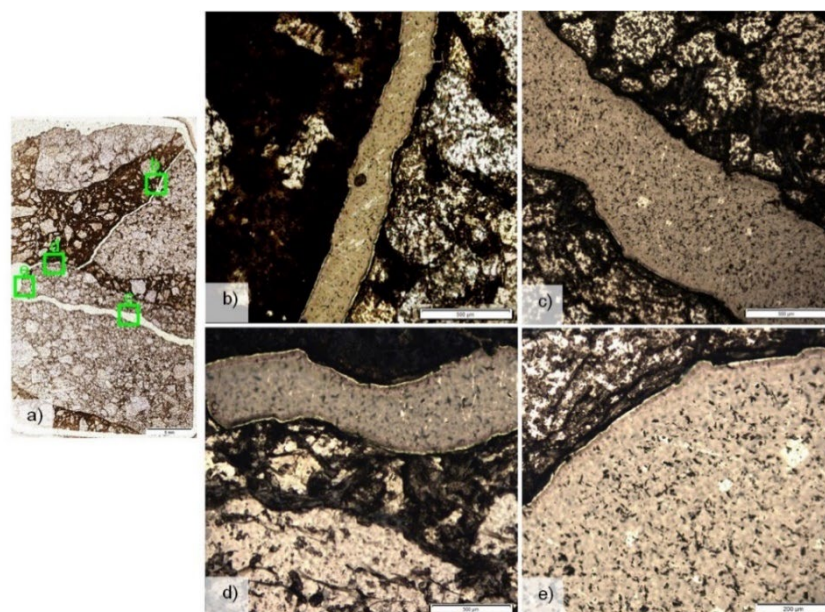


Figure 4-31: a) Thin section examined under PPL displays a concrete matrix containing various types of cracks (Scale bar 2 mm), 28b, 28c, 28d & 28e) Selected crack veins in the concrete matrix (Scale bar 500 um)

In Figure 4-31b, there is evidence of a thin gel lining the walls of the crack vein. However, epoxy, used in thin-section preparation, can be seen to fill a significant area of the crack veins. In Figure 4-31c, the crack vein is located within the coarse aggregate; ASR gel also lines the crack wall but is less prominent. Figure 4-31d is another example of a crack vein at the interface between the cement paste and coarse aggregate, showing the presence of ASR gel. Lastly, Figure 4-31e shows the presence of ASR gel at the aggregate interface at higher resolution (200 μm). The limited presence of ASR gel in the concrete could be attributed to water used as a lubricant during the core extraction from the RC railway bridge. Furthermore, the potential shrinkage of ASR gel upon dehydration after extraction could explain the limited gel presence in the concrete.

Figure 4-32a focused on the interface between a Witwatersrand quartzite coarse aggregate and the cement paste under PPL, with a sub-area under XPL revealing microcrack veins in the coarse aggregate. Several microcracks branch off the predominant crack in Figure 4-32b. These cracks indicate an intricate crack network likely initiated by ASR-induced swelling pressure within the aggregate, which reduces its stiffness and permeability. The reduced permeability of the aggregate can facilitate the diffusion of alkalis further into the aggregate, promoting further development of ASR in the concrete.

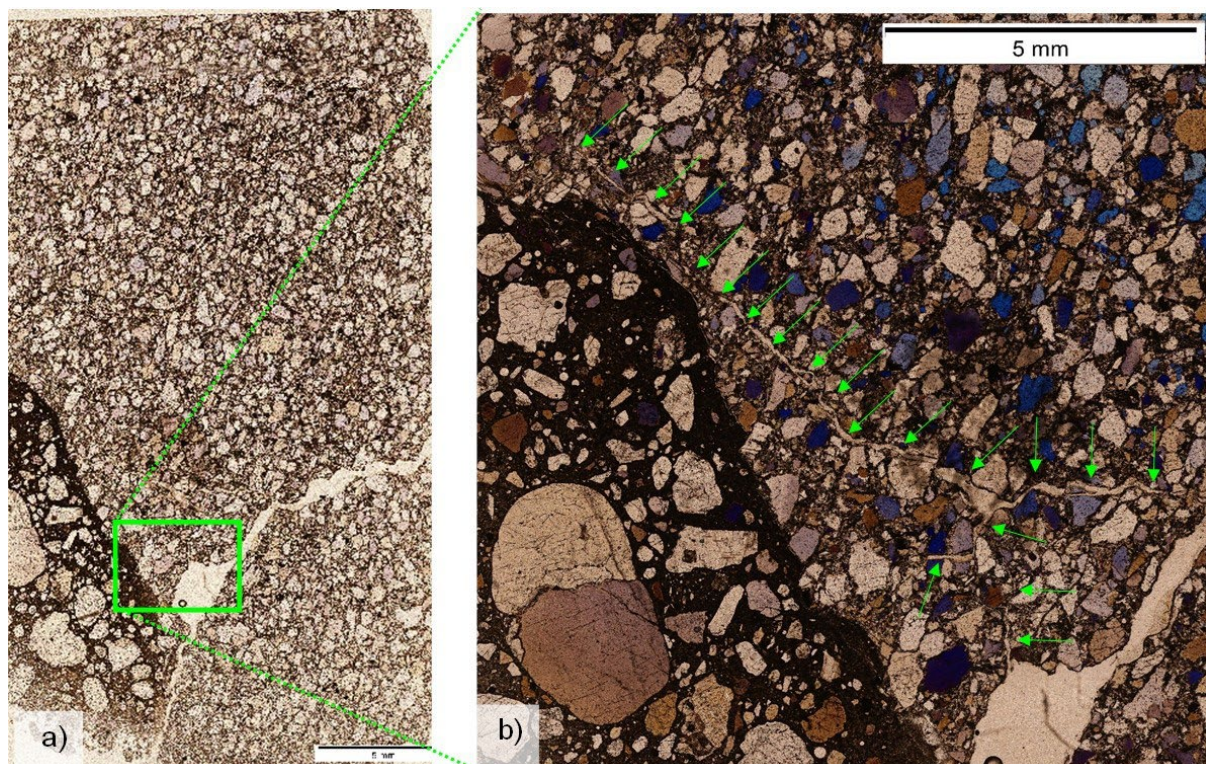


Figure 4-32: a) Concrete thin section examined under PPL (Scale bar 2 mm) b) Concrete thin section examined under XPL displaying crack network (Scale bar 5 mm)

Strain Quartzite grains were present in fine and coarse aggregate particles, as seen in Figure 4-33. This likely implies that the same parent rock type, Witwatersrand Quartzite, was crushed and used as fine aggregates to produce concrete.

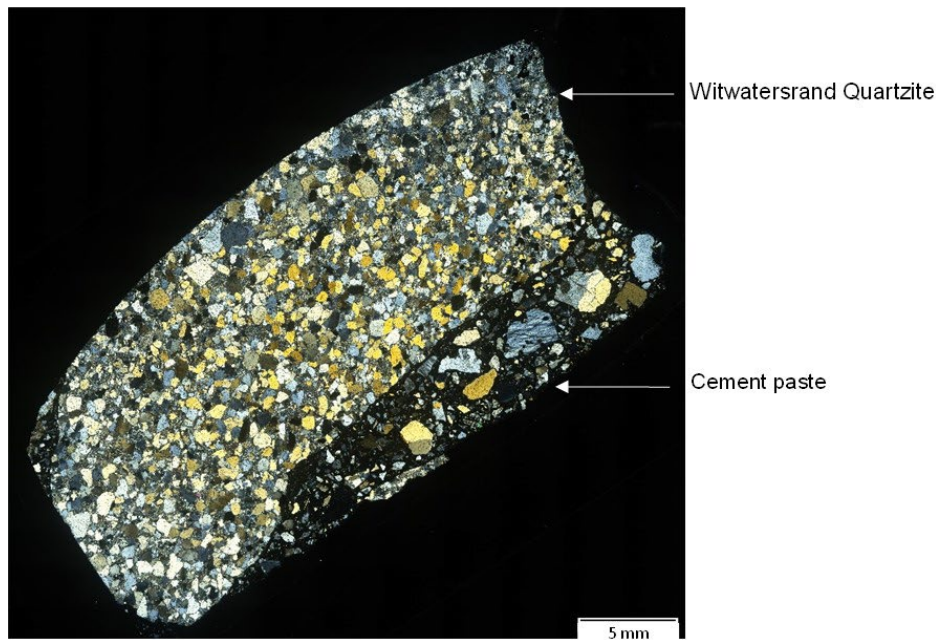


Figure 4-33: Image of concrete taken from slightly damaged area under XPL (scale bar: 5mm)

From first principles, the expectation is that ASR gel is more likely to initiate around fine aggregates due to the increased surface area of reactive grains and the close availability of alkalis in the surrounding cement matrix. Yet, from observation, evidence of cracking is more prominent in the coarse aggregates, where the strained grains are packed closer, reducing surface area, and there is limited availability of alkalis.

The following hypothesis can explain the observation: It may be that ASR does initiate around the fine aggregates (Kim *et al.*, 2019; Malone *et al.*, 2021a; Akhnoukh, 2023; Zapała-Sławeta, 2023). However, due to the size of the fine aggregate particles, the effects are not as noticeable, as seen in Figure 4-34a and Figure 4-34b.

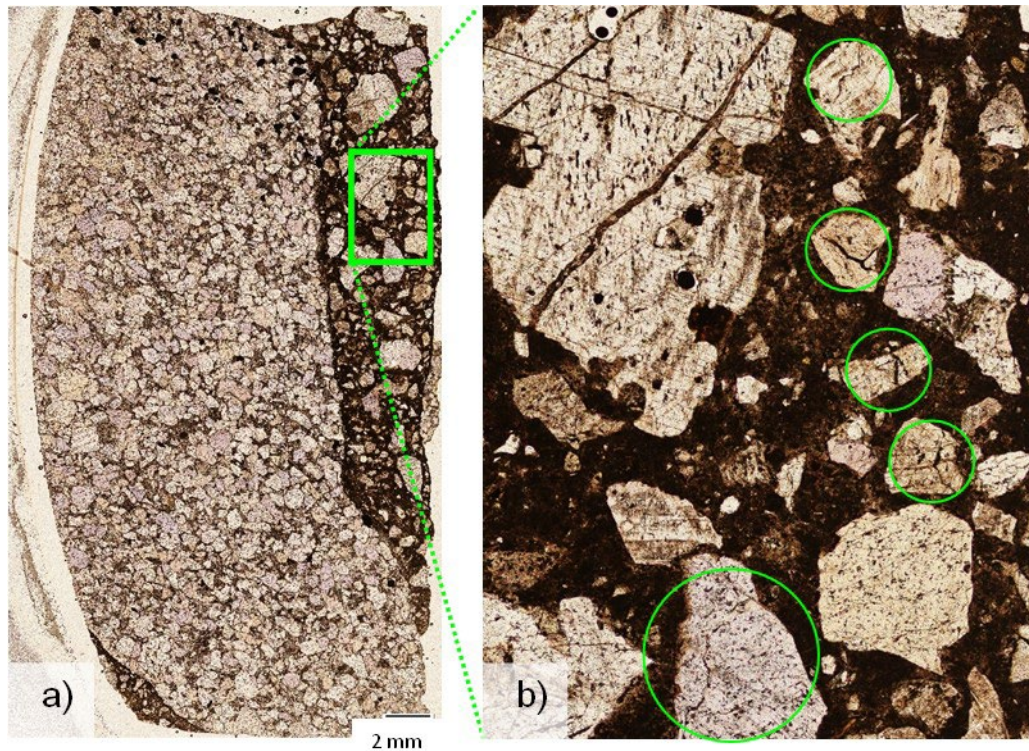


Figure 4-34: a) Concrete thin section examined under PPL (Scale bar 2 mm) b) Concrete thin section examined under PPL displaying cracked fine aggregates

Furthermore, while ASR may initiate around fine aggregates, the gel's ability to move into the more porous cement paste may reduce stresses from swelling pressure, leading to less prominent damage. In comparison to that of coarse aggregate, where the material is significantly stiffer than that of cement paste, thus any development of ASR in the aggregate would lead to the development of significant swelling pressures. Due to there being no voids for the gel to move into, the cracks propagate through the aggregate. Further studies are required to validate this hypothesis.

In conclusion, the petrography analysis of various thin sections confirmed ASR as the underlying deterioration mechanism. This was based on multiple diagnostic features of concrete subject to ASR deterioration, such as reactive aggregates, ASR gel, various crack types and crack networks.

4.9 u-XRF (micro X-ray fluorescence)

This section presents the results of using a micro X-ray fluorescence spectrometer to examine concrete discs; each disc was 94 mm in diameter and 10 mm thick. In Figure 4-35, the reference image of a concrete disc cut from one of the extracted cores is displayed, along with an elemental map highlighting the presence of calcium.

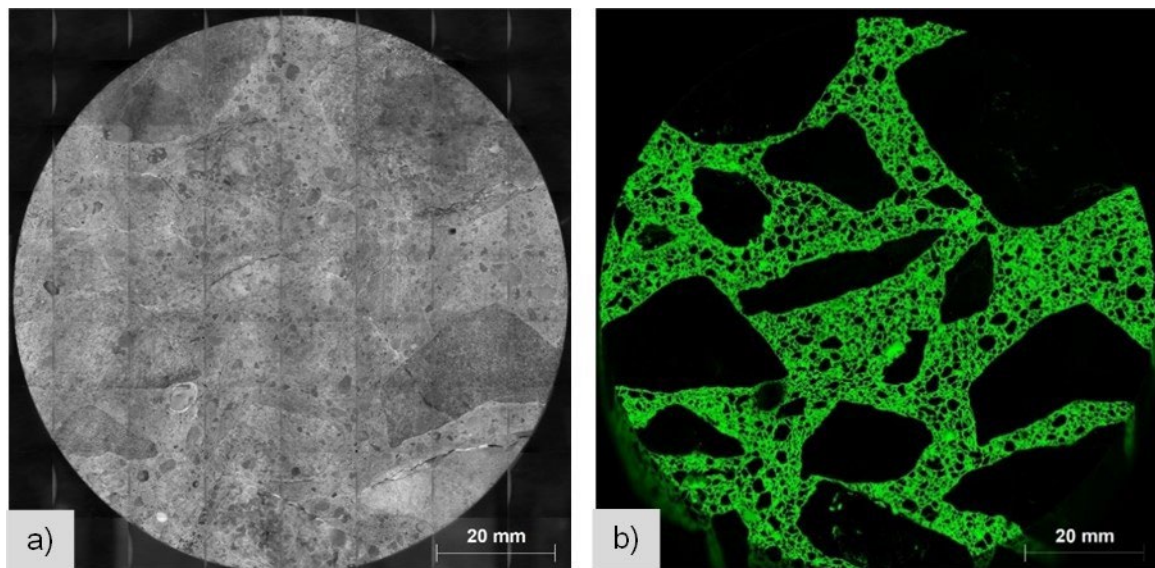


Figure 4-35: a) Reference concrete disc. b) Elemental map showing the presence of calcium on the surface of the concrete disc.

Both coarse and fine aggregates in concrete are distinguishable from the cement paste, depicted in green. The presence of calcium in concrete was attributed to the hydration of cement clinker with water, which leads to the reaction of anhydrous components resulting in the formation of hydrated products, mainly calcium silicate hydrate (C-S-H) accounting for approximately 70% (by mass) and well-crystallized calcium hydroxide (Portlandite) accounting for around 20% (by mass) (Diamond 1976).

The presence of alkalis in concrete, particularly sodium and potassium, indicated the potential for ASR development. In Figure 4-36, the reference image is accompanied by an elemental map highlighting the presence of silicon (in blue) and potassium (in purple). This step was crucial in identifying the presence of ASR gel in the concrete. However, the detection of sodium in the concrete was challenging using this technique due to its low molar mass.

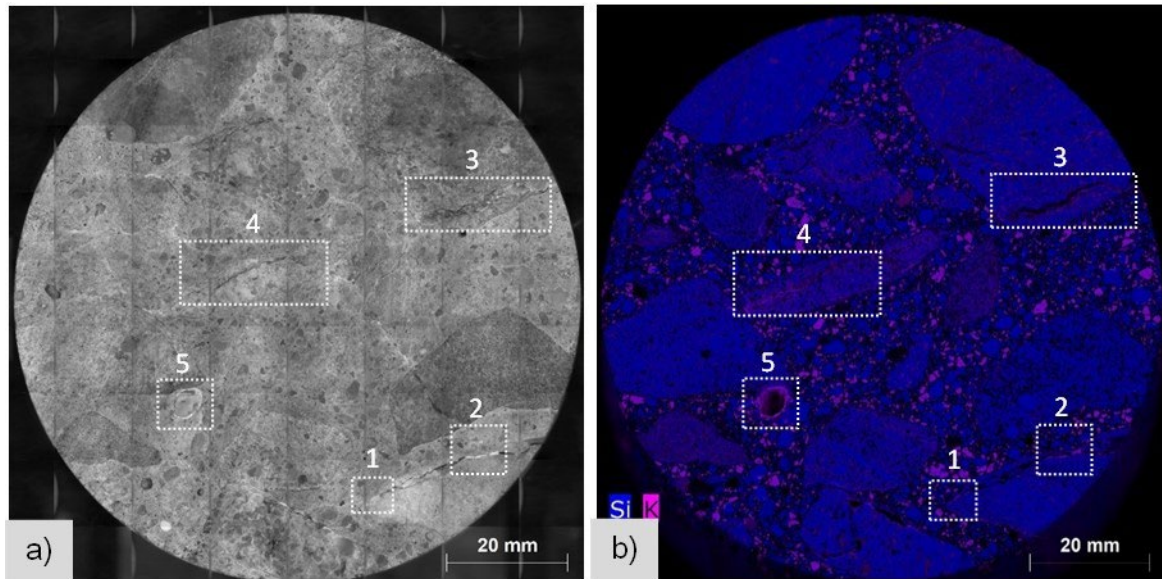


Figure 4-36: a) Reference concrete disc. b) Elemental map showing the presence of silica and potassium on the surface of the concrete disc.

Certain crack veins (areas 1 and 2) were observed to contain a whitish gel with potassium as part of its composition. Meanwhile, potassium was also detected in the surrounding regions of other veins without visible white gel, such as area 3. In area 4, even in the absence of visible white gel, a substantial amount of potassium was present within the crack veins. Lastly, another common feature in diagnosing the presence of ASR in concrete is the presence of "white-ish" aggregate rims, as seen in area 5, where the white-ish material containing potassium is also observed to contain potassium. This result was in agreement with findings by Kalina et al., (2018) and Malone et al. (2021b).

In addition to identifying the presence of ASR gel, the technique allowed spot tests to determine the chemical composition and confirm the various features noted in the concrete matrix. Figure 4-37a presents a primary phase map of a sub-area of the image from Figure 4-36, accompanied by an elemental map highlighting the presence of key elements: calcium (in green), silicon (in dark yellow), and potassium (in purple).

It can be seen that there are several locations where potassium was present in the concrete matrix. Firstly, potassium was located along the crack vein despite no signs of whitish gel in the reference image. Furthermore, traces of potassium can be seen, lining the boundary between the aggregate of the ITZ. Secondly, the aggregate at the bottom right of the image seems to contain potassium as part of its composition. Lastly, it can be seen that several of the fine

aggregate particles are rich in potassium, which could provide an additional source of alkalis to facilitate ASR (Kawamura, Koike and Nakano, 1989).

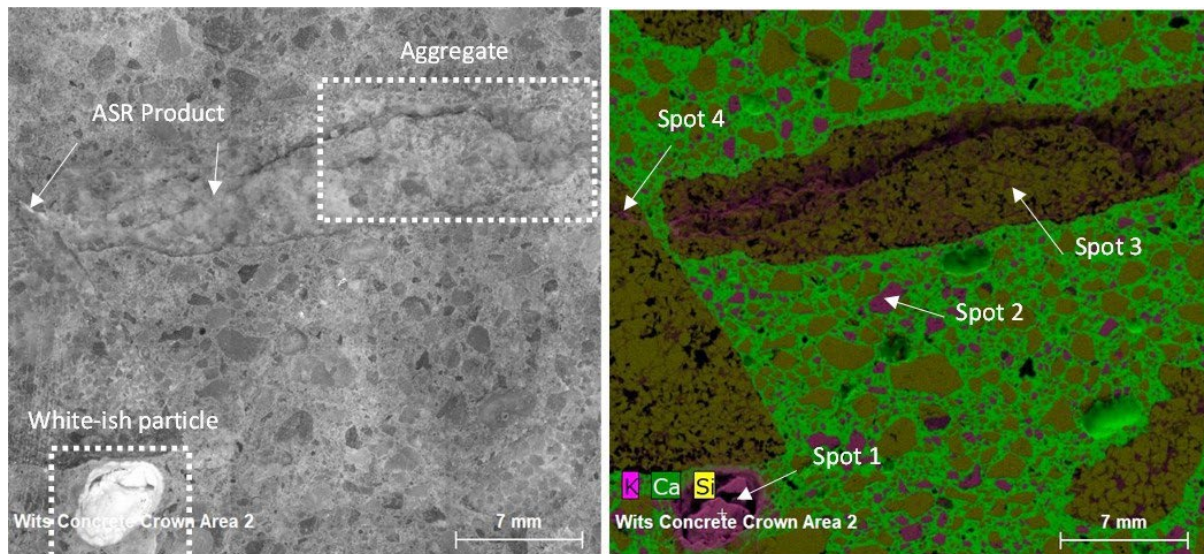


Figure 4-37:a) Reference concrete disc outlining ASR product, aggregates and whitish-particle b) Elemental map showing the presence of potassium, calcium and silica on four spot areas on the surface of the concrete disc.

Figure 4-38 to Figure 4-41 illustrate the XRF spectrums obtained for spots 1, 2, 3 and 4, respectively, as highlighted in Figure 4-37.

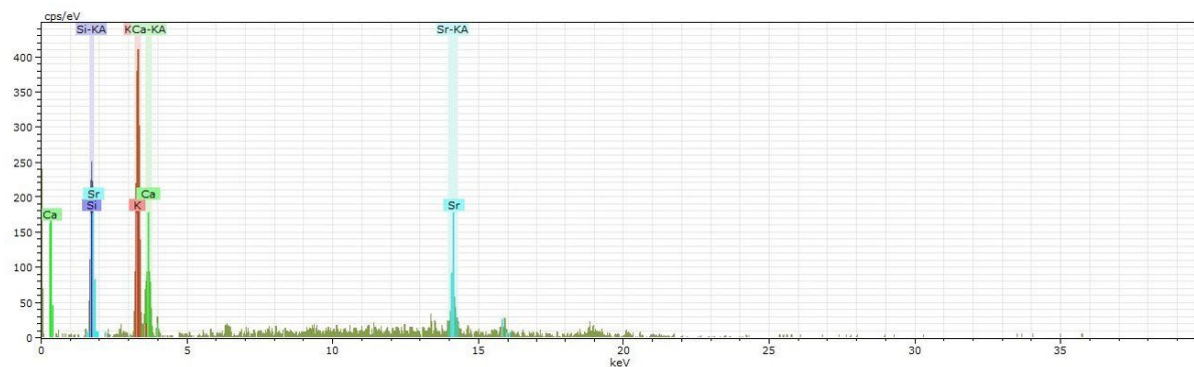


Figure 4-38:Spectrum of phases identified in Spot 1 (Cavity Material)



Figure 4-39: Spectrum of phases identified in Spot 2 (K-Rich Aggregate)



Figure 4-40: Spectrum of phases identified in Spot 3 (Aggregate)

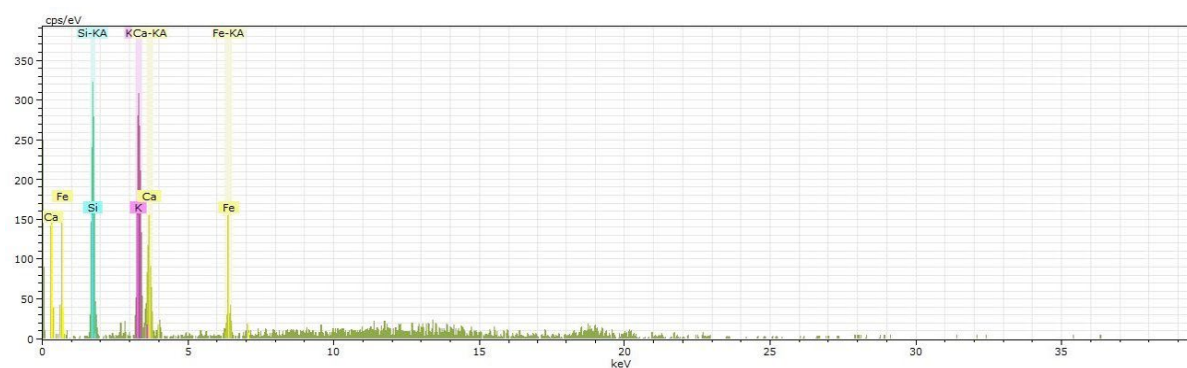


Figure 4-41: Spectrum of phases identified in Spot 4 (ASR Product)

Table 4-5 summarises the quantification of different phases within the respective scanned spots 1-4.

Table 4-5: Summary of distribution and proportion of different phases for spot areas 1-4

	Ca	K	Si	Al	Fe	O	Sr	Total	Phase
Spot 1 (Cavity Material) (wt. %)	4.5	23.0	30.5	-	0.1	41.3	0.5	99.9	K-Shlykovite
Spot 2 (K-Rich Aggregate) (wt. %)	6.3	7.1	27.2	12.2	0.8	46.1	-	99.8	Orthoclase
Spot 3 (Aggregate) (wt. %)	0.4	5.6	40.0	3.7	-	50.2	-	99.9	Quartz
Spot 4 (ASR Product) (wt. %)	3.5	17.3	34.5	-	-	44.3	-	99.7	K-Shlykovite

Studies have shown that sodium/potassium silica gels are variable in chemical composition (Stanton, 1940; Idorn, 1967; Poole A.B, 1975; Berube and Fournier, 1986; Davis and Oberholster, 1986; Hanson *et al.*, 2003; Mladenovic *et al.*, 2009). Thus, there is no standard/reference composition to which one could compare the composition of the ASR gel obtained.

The “white materials located in the crack vein, identified as K-Shlykovite, is a known form of ASR product as identified by Shi et al. (2020). The type of ASR gel formed is governed by the types of alkali sources available from the original cement; in most cases, Portland cement contains more potassium than sodium. In this instance, the additional availability of potassium-rich fine aggregates may have also contributed to the formation of K-shlykovite instead of Na-shlykovite. A similar ASR product is also identified in Spot 4 based on its composition. Lastly, it can be seen that the cracked coarse aggregate is primarily composed of Quarts.

Micro X-ray fluorescence spectrometry provided information about elemental distribution in the concrete matrix. This technique highlighted the location and characterisation of ASR gel over a large scan area, which was not possible with the naked eye or petrography. The results also affirm that the primary cause of the deterioration of the RC railway bridge was an alkali-silica reaction.

4.10 Free alkali metal content profiles

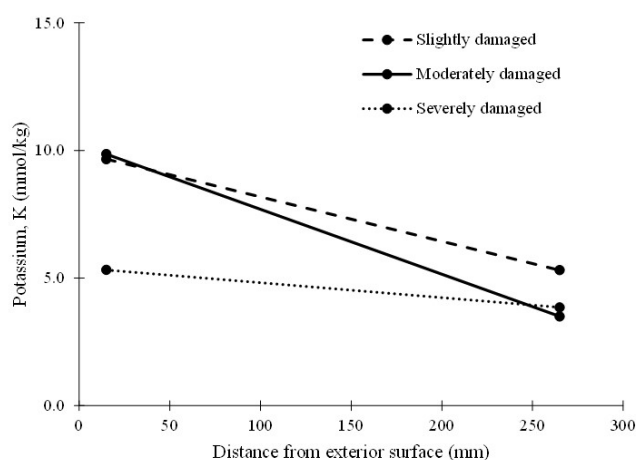
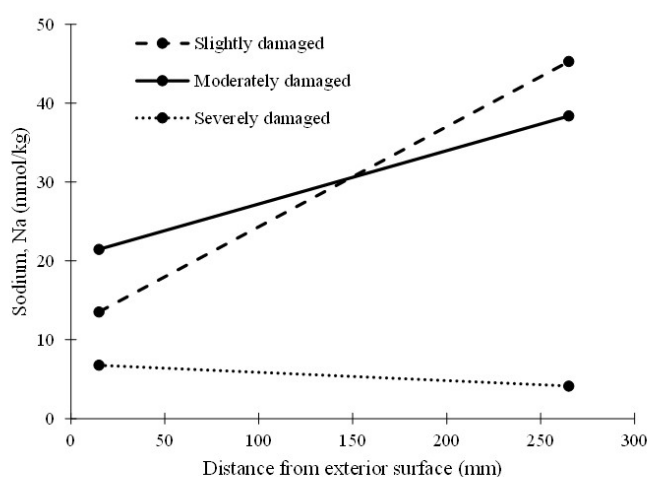
Determining the variation in alkali metal content served as a critical parameter in determining the extent of ASR in the concrete. This statement was based on the assumption that a lower percentage of alkalis present in the concrete indicated that a greater extent of ASR development has already occurred due to the consumption of alkalis to facilitate ASR.

Firstly, the alkali contribution from the Witwatersrand Quartzite used in constructing the RC railway bridge was determined. Kawabata and Yamada, (2017) has shown that certain aggregate types can contribute a significant percentage of alkalis to the pore solution through leaching. Table 4-6 summarises the amount of alkali metals released from the coarse aggregates using the CWE method. The aggregates tested were extracted from the concrete sample taken from a slightly damaged area.

Table 4-6:Release of alkali metals from the Witwatersrand Quartzite coarse aggregates during CWE (average of three measurements)

Element	Average	Unit
Na	4	mmol/kg
K	69	mmol/kg

Figure 4-42 shows the free alkali metal profiles (Na^+ , K^+ , and Ca^{2+}) obtained for concrete from slightly damaged, moderately damaged, and severely damaged regions using the CWE method. The data points were obtained at two locations along the length of the cores tested: 15 mm and 265 mm from the exterior surface of the core. Each point on the graph represents an average of three readings taken. The results have not been corrected to consider the aggregates' alkali contribution. The raw data can be located in Appendix E.



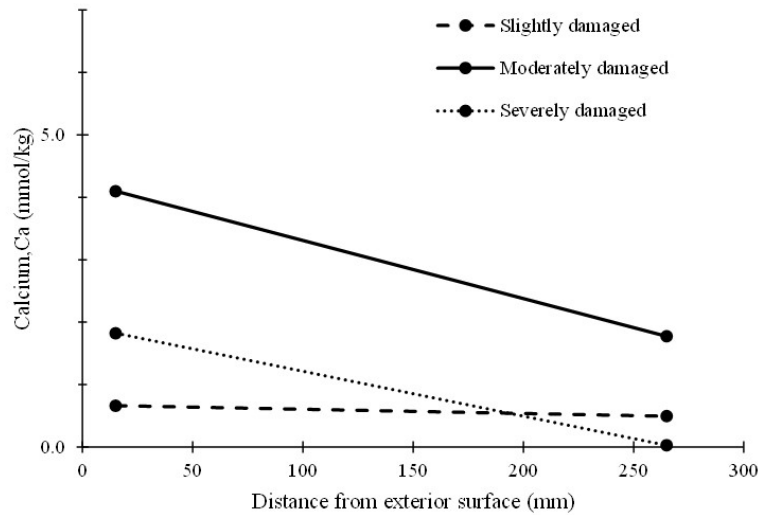


Figure 4-42: Na^+ , K^+ , and Ca^{2+} profiles (mmol/kg of concrete) as a function of distance from the reference surface from concrete from slightly damaged, moderately damaged, and severely damaged regions. This raw data was not corrected for potential alkali metal release from the aggregates.

As seen in Figure 4-42, the sodium molarity was generally lower close to the exterior surface and increased with increasing depth into the concrete. The sodium consumption close to the reference surface may likely be attributed to leaching. The assumption is supported by work done by Kagimoto, Sato, and Kawamura (2000), which showed that alkali metal leaching typically affects concrete cores up to about 100 mm depth from the exterior surfaces. The sodium molarity close to the exterior surface was 21 mmol/kg for concrete from moderately damaged regions. In contrast, sodium molarity for the slightly damaged and severely damaged regions was 36% and 66% lower, respectively. In contrast, the sodium molarity at the interior of the concrete was 45 mmol/kg for concrete taken from the slightly damaged region, while sodium molarity from the moderately damaged and severely damaged region was 15% and 90% lower, respectively.

In contrast, the potassium molarity was greater close to the exterior surface and decreased with increasing depth into the concrete for all three regions. This result appears inconsistent as the sodium and potassium profile trends follow each other (Plusquellec *et al.*, 2018), as demonstrated in Section 2.11.3, Figure 2-24. “Wick action” can also result in alkali’s being drawn to the exterior surface. However, the same trend would have been observed in the sodium profiles, which was not the case. A possible explanation for this result may be potassium leaching from the aggregates over time. Table 4-6 shows that the potassium molarity

of the aggregate is 17 times greater than that of sodium. The high quantity of potassium in the aggregate was also noted in the u-XRF elemental maps analysis in Section 4.9. This hypothesis requires further testing and investigation.

The potassium molarity close to the exterior surface was 10 mmol/kg for concrete taken from the slightly damaged and moderately damaged regions, while the potassium molarity for the severely damaged region was 50% lower. The potassium molarity at the interior of the concrete was 5 mmol/kg for concrete taken from the slightly damaged region, while the potassium molarity for the moderately damaged and severely damaged regions was 34% and 27% lower, respectively.

The calcium profiles are also plotted in Figure 4-42. These profiles were also obtained using CWE, but methanol was used for the extraction instead of deionised water. The calcium molarity, like potassium, was greater close to the exterior surface and decreased with increasing depth into the concrete for all three regions. Concrete from the moderately damaged region exhibited the highest calcium molarity of 4.1 mmol/kg at the exterior compared to 1.8 mmol/kg and 0.7 mmol/kg for the severely damaged and slightly damaged concrete, respectively. The interior calcium molarity was 1.8 mmol/kg for the moderately damaged region, compared to 0.5 mmol/kg and 0 mmol/kg for the slightly damaged and severely damaged region, respectively.

No information was available on the original cement type and subsequent alkali content from when the bridge was constructed. Thus, it was not possible to determine the percentage of alkalis consumed over the 78-year duration. Therefore, the highest sodium and potassium values were selected as the reference “original” alkali content, and the subsequent percentage of free alkali metals consumed was determined for concrete cores extracted from the various locations of the RC railway bridge.

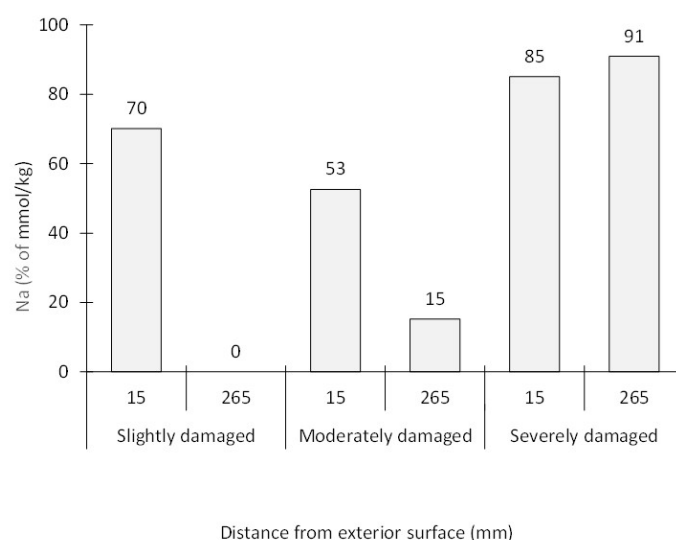


Figure 4-43:Percentage of sodium consumed from concrete from slightly, moderately and severely damaged areas

Firstly, Figure 4-43 showed that the reference (highest) sodium molarity was measured from concrete taken from a slightly damaged region, further into the depth of the concrete. As expected, the severely damaged concrete (close to the surface and further into the depth of the concrete) exhibited the greatest sodium consumption of 85% and 91% when compared to slightly damaged and moderately damaged concrete.

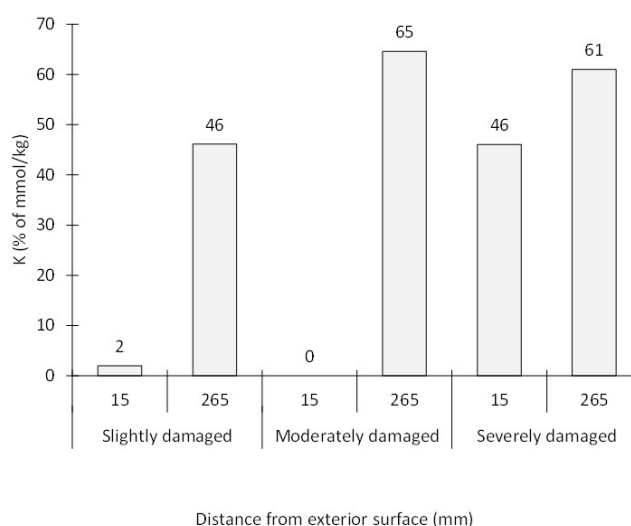


Figure 4-44: Percentage of potassium consumed from concrete from slightly, moderately and severely damaged regions.

Figure 4-44 showed that the reference (highest) potassium molarity was measured from concrete taken from slightly damaged and moderately damaged regions closer to the surface of the concrete. In this instance, the inner concrete from the moderately damaged areas exhibited the greatest consumption of potassium, 65%. However, it can be seen that the severely damaged

concrete close to the surface and further into the depth of the concrete exhibited the subsequent highest consumption of sodium, 46% and 61%, respectively.

The results from the alkali metal inventory were then used as inputs to determine the equivalent sodium oxide content of the concrete, using Equation 3-4. Figure 4-45 shows the sodium oxide equivalent content for the three regions and the corrected sodium oxide equivalent content, taking into account the contribution from aggregates (assuming an aggregate proportion of 70% of concrete mixture design), using Equation 3-2 and Equation 3-3.

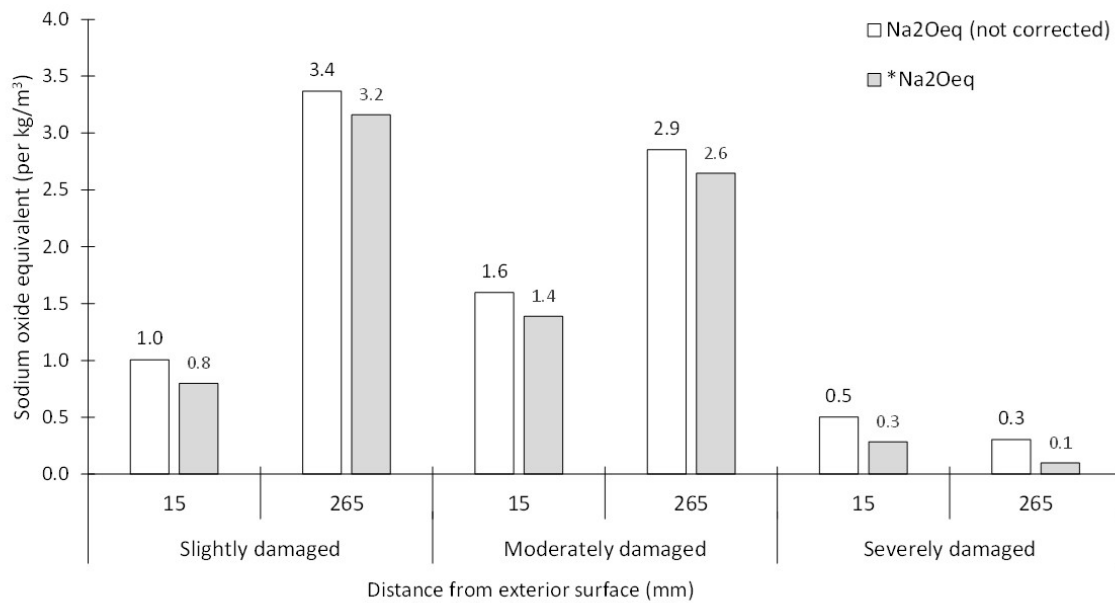


Figure 4-45: Comparison of the free alkali metal profiles obtained for concrete from slightly, moderately and severely damaged regions. The data are expressed in kg/m³ Na₂O_{eq}, and the results shown have been corrected for the contribution of alkali from the aggregates during CWE.

The general trend observed in Figure 4-45 is that the sodium oxide equivalent content decreased as the extent of observed concrete damage increased, as expected. These results support the hypothesis that the reduction of the alkali hydroxides in the concrete is attributed to the initiation of ASR, which results in the consumption of alkalis in the pore solution (Bérubé, Ollivier and Ballivy, 2007; Liu *et al.*, 2022). This reaction facilitates the production of ASR gel, which is visually noted through signs of distress on the exterior surface of the RC railway bridge. The difference between uncorrected and corrected sodium oxide equivalent content was 0.21 kg/m³, owing to the alkali contribution from Witwatersrand quartzite. Concrete from the interior slightly damaged region exhibited the highest corrected sodium oxide equivalent content of 3.2 kg/m³, compared to 2.6 kg/m³ and 0.1 kg/m³ for the concrete from the interior moderately and severely damaged region, respectively. Concrete from the exterior moderately damaged region exhibited the

highest corrected sodium oxide equivalent content of 1.4 kg/m^3 , compared to 0.8 kg/m^3 and 0.3 kg/m^3 for the concrete from the interior slightly damaged and severely damaged region, respectively. The maximum sodium oxide equivalent content corresponding to concrete taken from the slightly damaged region was then used to determine the theoretical pore solution composition necessary for the residual expansion tests, as illustrated in Figure 4-46.

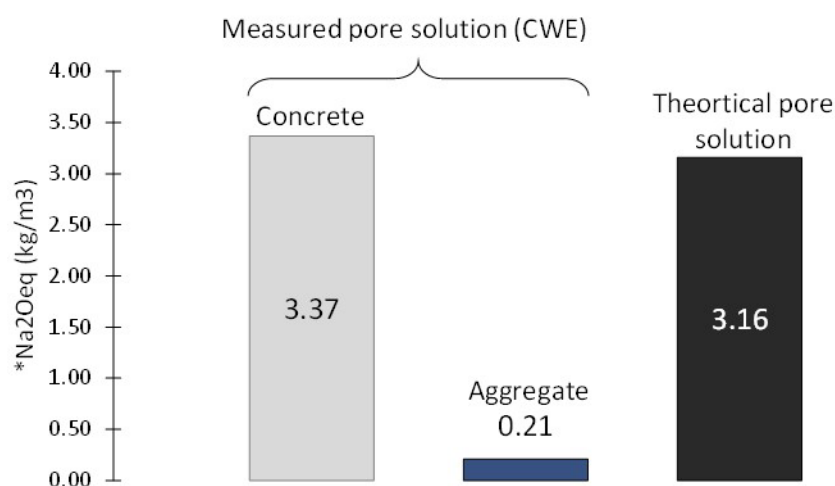


Figure 4-46: Determination of the theoretical pore solution composition based on the CWE measurements

It can be seen from Figure 4-46 that the theoretical pore solution, based on sodium oxide equivalent content, amounts to 3.16 kg/m^3 . When comparing this amount to the stipulated alkali limits for controlling expansion from reactive aggregate used locally in South Africa in Table 2-7, the value exceeds the 2.0 kg/m^3 limit for Witwatersrand aggregates (Alexander and Blight, 2017) by 58%. This result supports the premise that the concrete produced to construct the RC railway bridge had a sufficiently high concentration of alkalis in the cement used for construction. Thus, the increased alkalinity in the concrete pore solution was sufficient to facilitate the development of ASR (Shayan, Xu and Andrews-Phaedonos, 2015; Borchers, 2021a; Behravan *et al.*, 2025).

In conclusion, the CWE method provided information about the alkali inventory in the concrete extracted from various locations of the RC railway bridge. This knowledge was useful in determining the percentage of alkalis consumed in the concrete relative to the extent of visual ASR damage observed on the structure. Lastly, the corrected sodium oxide equivalent content was determined based on the alkali inventory and the alkali contribution from aggregates. This was a crucial step in determining the theoretical pore solution composition required to replicate the alkalis available (in the original concrete pore solution of the RC railway bridge) when conducting the residual expansion test in the last phase of the study.

4.11 Stiffness Damage Test

The compressive strength was necessary to establish the maximum load to apply to the specimen for the stiffness damage test. 30% of the ultimate compressive strength for concrete from a slightly damaged region corresponded to a 9.5 MPa stress. Figure 4-47 depicts a typical result of the strain–stress relationship obtained for concrete.

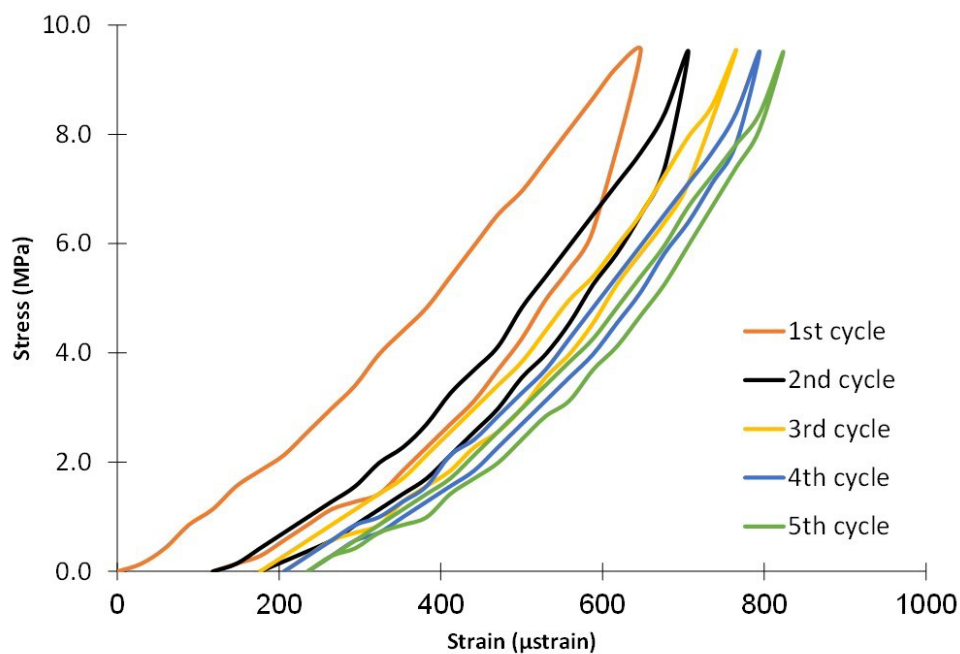


Figure 4-47: SDT stress-strain curve for concrete taken from a slightly damaged area

Figure 4-47 shows that the shape of the stress-strain curve obtained over the five-cycle test duration is convex as opposed to concave. This observation was confirmed by an NLI value of 0.85, lower than one, validating the stiffening effect of the material over the test duration. Similar results have been reported by Kongshaug et al. (2020) and Zahedi et al. (2022), attributing the result to the crack network being parallel to the test loading direction. This result contradicts the findings obtained from the examination of 2D images of the circumferential surface of each concrete core, where most cracks were found to be parallel to the exterior surface but perpendicular to the direction of the applied load in the test.

Comparing stress-strain curves for saturated concrete taken from slightly, moderately, and severely damaged regions in Figure 4-48, an apparent increase in hysteresis area, plastic deformation, and reduction in modulus of elasticity, is evident over the five cycles, in agreement with the increasing extent of damage due to ASR.

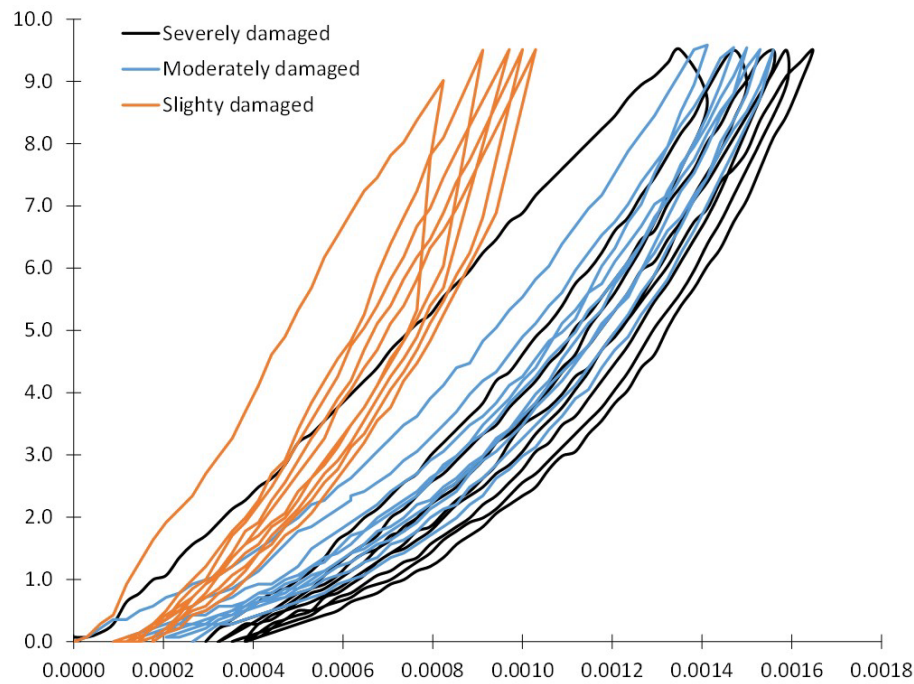


Figure 4-48:SDT stress-strain curves for saturated concrete taken from slightly, moderately, and severely damaged areas

The subsequent calculation of modulus of elasticity (E-Mod), hysteresis area (HA), and SDT indices (SDI, PDI, NLI) is presented in Table 4-7 and highlights the decrease in material stiffness due to increasing internal damage owing to ASR deterioration.

Table 4-7: Comparison of SDT output parameters for concrete taken from slightly, moderately and severely damaged regions

	E-Mod (GPa)	HA (J/m³)	SDI	PDI	NLI
Slightly damaged region	18.0	1510	0.53	0.14	0.85
Moderately damaged region	14.0	2635	0.88	0.29	0.86
Severely damaged region	12.4	2842	1.08	0.31	0.79

The modulus of elasticity decreased with the increased extent of damage, which agrees with the work cited in the literature. Compared to concrete from slightly damaged areas, there is a 22.2% and 31.1% reduction in elastic modulus for the concrete taken from the moderately and severely damaged regions, respectively. In the absence of the design (original) elastic modulus value, the results were compared to values estimated using BS 8110-2:1985 (1985) and incorporating various aggregate types from South Africa (Alexander, 1991; Alexander and Davis, 1991, 1992; Mark Alexander, 1993), as shown in Equation 4-1.

$$E_{c,six\ months\ or\ older} = K_0 + \alpha F_{cu,six\ months\ or\ older} \quad \text{Equation 4-1}$$

where :

E_c = modulus of elasticity at a stipulated age

K_0 = aggregate stiffness factor related to the elastic modulus of the aggregate and its volume concentration

α = coefficient

F_{cu} = compressive strength at a stipulated age.

The general aggregate stiffness for Gauteng reef Quartzite ranges from 33 to 35 GPa, while the associate coefficient for concrete older than six months is 0.1. Using the compressive strength of slightly damaged concrete of 31.1 MPa led to the estimation of the design elastic modulus. Considering the lower elastic modulus estimate of 33 GPa, this equates to a 45%, 58% and 62 % reduction in elastic modulus compared to the slightly, moderately and severely damaged concrete, respectively. In addition, the upper elastic modulus estimate of 35 GPa equates to 48%, 50%, and 65% reductions in elastic modulus compared to the slightly, moderately, and severely damaged concrete. The upper range of 60% reduction in elastic modulus is in line with values stated by the Institution of Structural Engineers (1992)

Concrete from the severely damaged area produced the highest hysteresis area, SDI and PDI values, followed by moderately damaged and slightly damaged concrete. This trend is in agreement with the work cited in the literature, whereby concrete with the greatest extent of internal damage attributed to ASR will generate higher hysteresis area and subsequent SDI and PDI values.

When comparing the SDT outputs, it can be seen that the hysteresis area, SDI and PDI from moderately damaged regions are relatively closer to damaged concrete than slightly damaged concrete. The result may be attributed to the use of Core C10 as a representative specimen from a slightly damaged region. It can be seen that in

Figure 4-10, the location of the core C10 is closer to the edge of the interior wall. Thus, it was subject to increased moisture exposure, possibly attributing to the greater extent of internal damage reflected in the stiffness damaged test. The NLI values for all three concrete types were lower than one, confirmed by the convex stress-strain curves shown in Figure 4-48.

When considering the implications of the SDT results, with regard to the reduction in concrete properties in the actual RC railway bridge, the results cannot simply be adopted blindly. The direction of loading in the test and the direction of loading in the actual structure must be considered in conjunction with the predominant orientation of cracks in the extracted concrete (as seen in Figure 4-49). In this instance, the direction of loading in the test is perpendicular to the loading direction in the actual structure. Based on the orientation of the cracks relative to the direction of loading, it can be seen that the STD results can differ, as shown in Figure 2-28. The modulus of elasticity of the concrete in the structure is expected to be higher than the SDT results obtained; further testing is required to estimate the elastic modulus when the loading conditions are aligned to that of the actual structure, in addition to the effects of restraint present in difference directions in the actual RC structure.

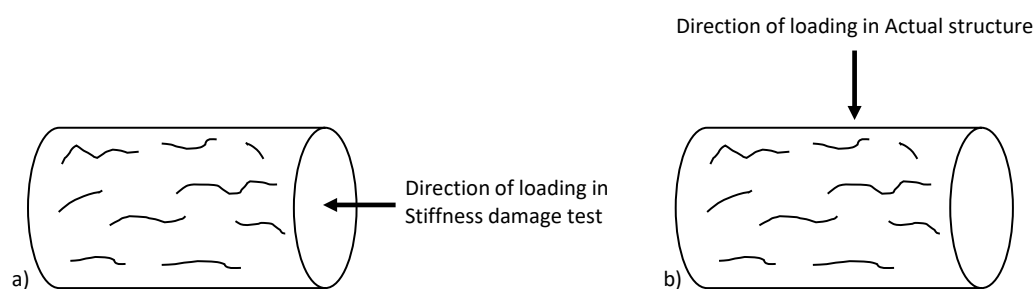


Figure 4-49: Schematic of loading of concrete in SDT compared to loading of concrete in actual loading, relative to the concrete core predominant crack orientation

Furthermore, the influence of wetting and drying cycles on stiffness damage parameters of concrete subject to ASR deterioration was investigated. As stated in Section 3.8, specimens were initially immersed in water at $23^{\circ}\text{C} \pm 2^{\circ}\text{C}$, followed by the performance of a SDT. Subsequently, the specimens were subject to a one-month drying period at room temperature (23°C), followed by the performance of a SDT. This cyclic exposure of being submerged in water and subsequent drying was repeated three times. Figure 4-50 depicts a typical result of wetting and drying cycles on the strain–stress relationship obtained for concrete from slightly damaged concrete. These results were displayed on two separate graphs, as it proved challenging to identify any trends when combining all six STD graphs on a single axis.

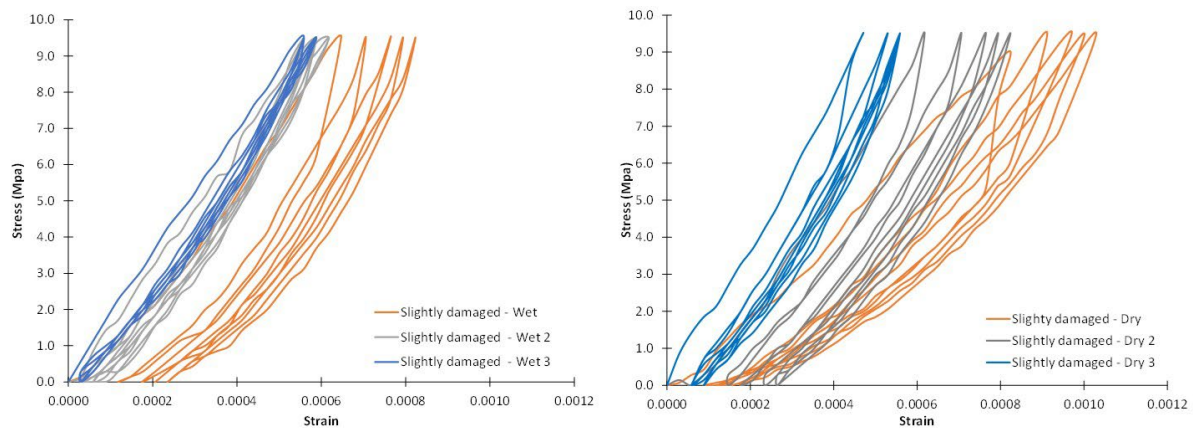


Figure 4-50: Effect of moisture conditioning SDT stress-strain curve for concrete taken from a slightly damaged area

Table 4-8 summarises all the SDT output parameters for concrete tested from slightly, moderately, and severely damaged regions subject to wetting and drying.

Table 4-8: Comparison of SDT output parameters for concrete taken from slightly, moderately and severely damaged regions subject to wetting and drying

		E-Mod (GPa)	HA (J/m ³)	SDI	PDI	NLI
Slightly damaged region	Wet - 1	17.0	2635	0.880	0.29	0.86
	Dry - 1	11.6	4691	0.896	0.37	0.95
	Wet - 2	17.0	388	0.054	0.10	0.94
	Dry - 2	18.0	2092	0.497	0.32	0.81
	Wet - 3	17.3	848	0.175	0.05	1.04
	Dry - 3	19.9	1223	0.324	0.16	0.90
Moderately damaged region	Wet - 1	13.8	1510	0.533	0.14	0.85
	Dry - 1	7.2	5162	0.662	0.17	0.74
	Wet - 2	16.4	1436	0.321	0.05	0.97
	Dry - 2	17.3	1854	0.390	0.41	0.83
	Wet - 3	16.2	933	0.196	0.09	0.94
	Dry - 3	20.9	1297	0.344	0.11	1.05
Severely damaged region	Wet - 1	12.3	2842	1.088	0.31	0.79
	Dry - 1	7.9	7504	0.811	0.23	0.95
	Wet - 2	10.6	4126	0.625	0.21	0.86
	Dry - 2	12.0	2150	0.356	0.16	0.82
	Wet - 3	11.1	1952	0.315	0.13	0.84
	Dry - 3	14.5	4032	1.147	0.15	0.77

A general trend observed for all three concrete types (slightly, moderately and severely damaged) was that the elastic modulus of concrete was generally higher under wet conditions than in dry conditions. The result was expected (Shayan, Xu and Andrews-Phaedonos, no date; Zhang *et al.*, 2020; Borchers, 2021b; Wang *et al.*, 2022), as under moist conditions, the concrete has additional pore pressure and existing ASR gel pressure that can likely be attributed

to increased stiffness and provide subsequent resistance to load. It was not possible to separate the contribution from either source using these results. However, this result warrants further investigation. Interestingly, the elastic modulus increased throughout the entire test duration of three wetting and three dry cycles, indicating that the material became stiffer over time. The only exception was concrete from severely damaged regions under wet - 2 conditions.

Furthermore, it can be seen that, in general, the amount of energy dissipated for a given concrete type decreased over the complete test duration. When comparing the results from a drying cycle to a previous drying cycle and similarly for the wetting cycles, results from slightly damaged—Wet 3 and severely damaged dry—3 cycles did not follow this trend.

Moreover, the behaviour of the cracks and surface texture in the concrete over the duration of the SDT may contribute to increased stiffness. Initially, when the load is applied to the specimen, in the 1st cycle, a large amount of energy is dissipated. This results from stress being applied to the specimen, leading to existing cracks opening and propagating to a certain degree. However, when such cracks close due to the unloading of the specimen, the rough surface creates an interlocking effect that causes cracks to slide and close, resulting in the stiffening of the material. This can be observed in the subsequent SDT cycle, 2 to 5, where the material dissipated much lower energy. This result was also supported by corresponding SDI and PDI values, which were reduced when comparing the SDI and PDI results over the course of the testing duration.

Lastly, the results from the SDT over various wetting and drying cycles were used to investigate the tangent modulus versus the secant modulus to determine the modulus of elasticity, as shown in Figure 4-51. In addition, the use of the loading versus unloading curve from the SDT results to determine the modulus of elasticity was also considered part of the investigation.

The first method involves measuring the loading curve's secant modulus (**SEC E₁**) for the 2nd, 3rd, 4th and 5th SDT cycles and determining the average value. The second method involves that same process, except the unloading curve is used to determine the secant modulus (**SEC E₂**) – this is the recommended approach. The third method involves measuring the tangent modulus (**TAN E₁**) at the stress corresponding to 20% of the ultimate compressive strength for the loading curve for the 2nd, 3rd, 4th and 5th SDT cycle and determining the average value. The last method involves that same process, except the unloading curve was used to determine the

tangent modulus (**TAN E₂**). The results of using the various approaches are summarised in the Table 4-9.

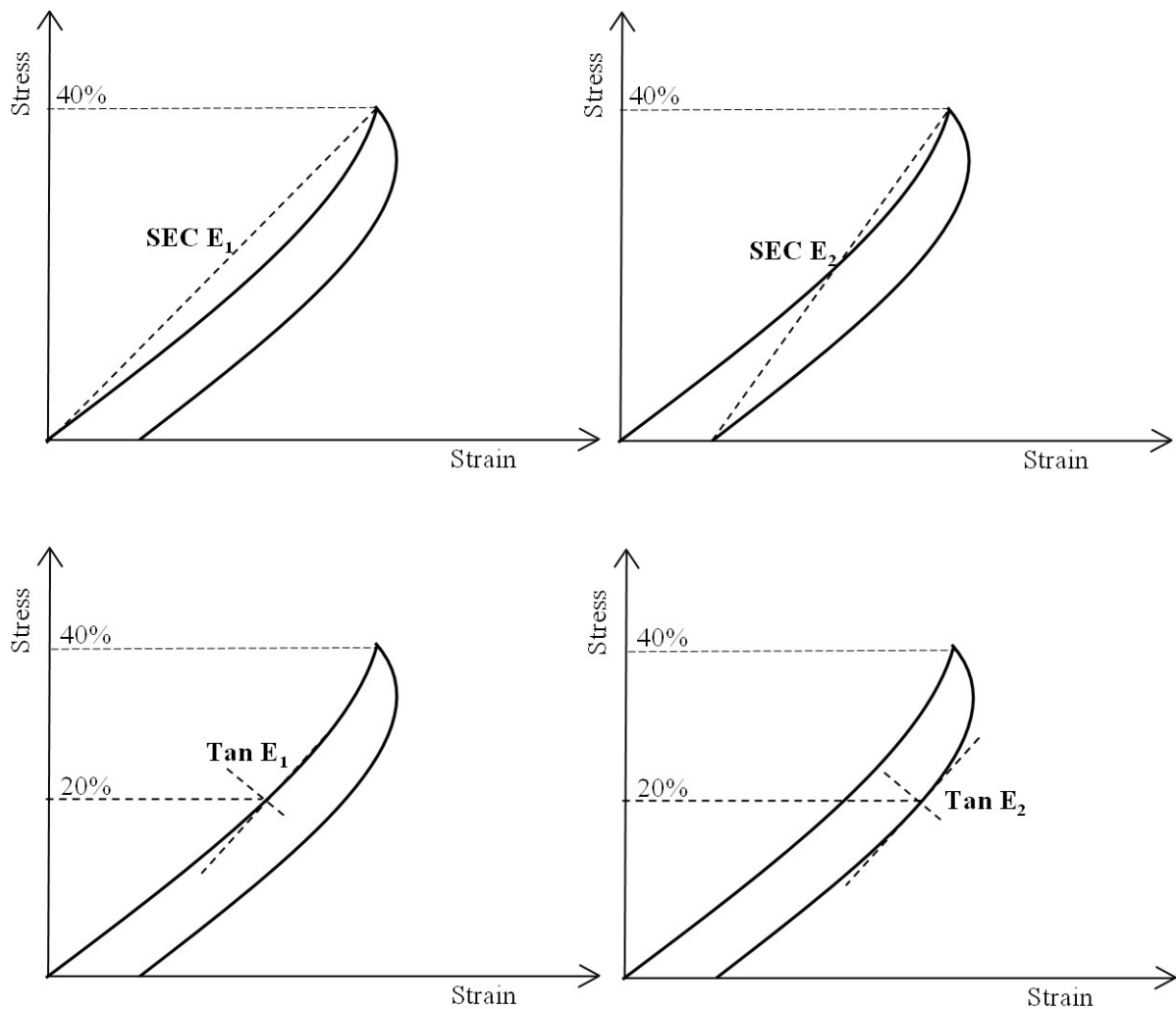


Figure 4-51: Comparison of elastic modulus using various forms for concrete taken from undamaged, moderately and severely damaged regions subject to wetting and drying

Table 4-9: Comparison of elastic modulus using various forms for concrete taken from undamaged, moderately and severely damaged regions subject to wetting and drying

		Elastic modulus (GPa)						
		E ₁	E ₂	Tan E ₁	Tan E ₂	Mean	Std. Dev.	Var.
Slightly	Wet - 1	16.2	17.0	14.5	21.7	17.4	3.1	9.51
	Dry - 1	11.3	11.6	12.7	11.5	11.8	0.6	0.39
	Wet - 2	16.8	17.0	16.9	18.7	17.3	0.9	0.82
	Dry - 2	17.0	18.0	16.9	16.3	17.0	0.7	0.48
	Wet - 3	17.3	17.3	16.9	16.3	16.9	0.5	0.20
	Dry - 3	19.6	19.9	19.3	19.3	19.5	0.3	0.09
Moderately	Wet - 1	13.6	13.8	13.3	12.7	13.3	0.5	0.21
	Dry - 1	7.0	7.2	7.5	9.4	7.8	1.1	1.17
	Wet - 2	16.4	16.4	17.5	15.7	16.5	0.7	0.56
	Dry - 2	16.0	17.3	16.9	17.1	16.8	0.6	0.32
	Wet - 3	16.2	16.2	16.9	15.1	16.1	0.7	0.55
	Dry - 3	20.6	20.9	19.9	19.3	20.2	0.7	0.50
Severely	Wet - 1	11.9	12.3	19.3	16.9	15.1	3.6	13.07
	Dry - 1	7.7	7.9	8.1	10.3	8.5	1.2	1.45
	Wet - 2	10.5	10.6	11.5	13.3	11.5	1.3	1.70
	Dry - 2	11.7	12.0	10.9	14.5	12.3	1.6	2.43
	Wet - 3	11.1	11.1	12.1	11.5	11.5	0.4	0.20
	Dry - 3	14.4	14.5	14.5	14.5	14.5	0.1	0.00

For a given degree of concrete distress (slightly, moderately and severely), 50% of the values obtained using the E₁ approach resulted in the lowest elastic modulus value compared to E₂, Tan₁ and Tan₂. This result was expected as any cracks in the material were likely to propagate under the application of increasing load, thus resulting in reduced material stiffness. This approach would lead to the worst-case scenario when considering the reduction in elastic modulus and subsequent structural implications in an actual structure. In contrast, 30% of the values obtained using E₂ and Tan₂ lead to the most conservative elastic modulus values. This result was expected as any cracks in the material were likely to close under the release of load, resulting in stiffening of the material.

Figure 4-52, Figure 4-53, Figure 4-54 and Figure 4-55 display plots of the various elastic modulus approaches investigated. The various elastic modulus approaches were compared using a line with a slope equal to 1 (equality line) to determine the correlation. Additionally, a Bland-Altman statistical analysis was conducted to assess the agreement between the four elastic modulus approaches investigated. The selected statistical approach compared two measurement techniques and assessed whether there was an agreement between two data sets.

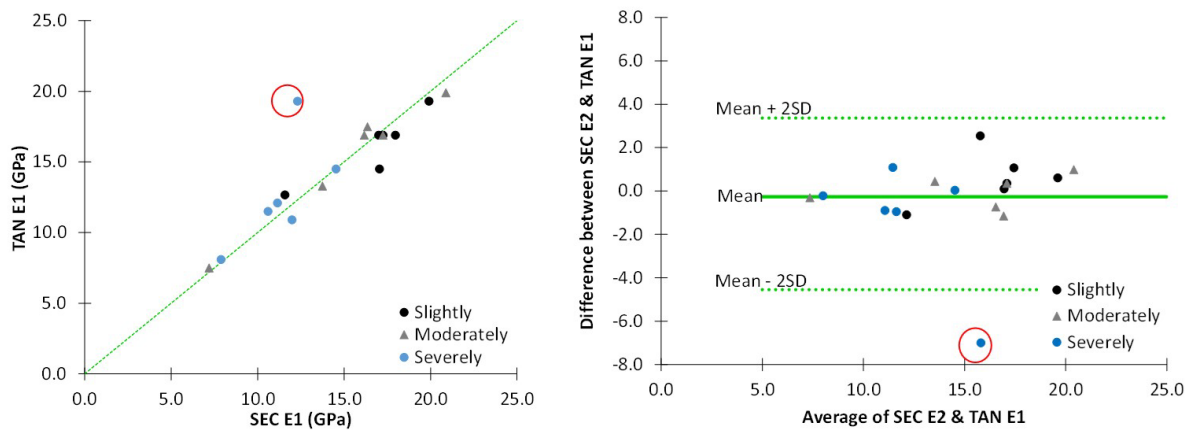


Figure 4-52: Comparison of elastic modulus using secant modulus (E_1) and tangent modulus (E_1) of loading using 2nd, 3rd, 4th and 5th SDT cycle and a Bland-Altman plot

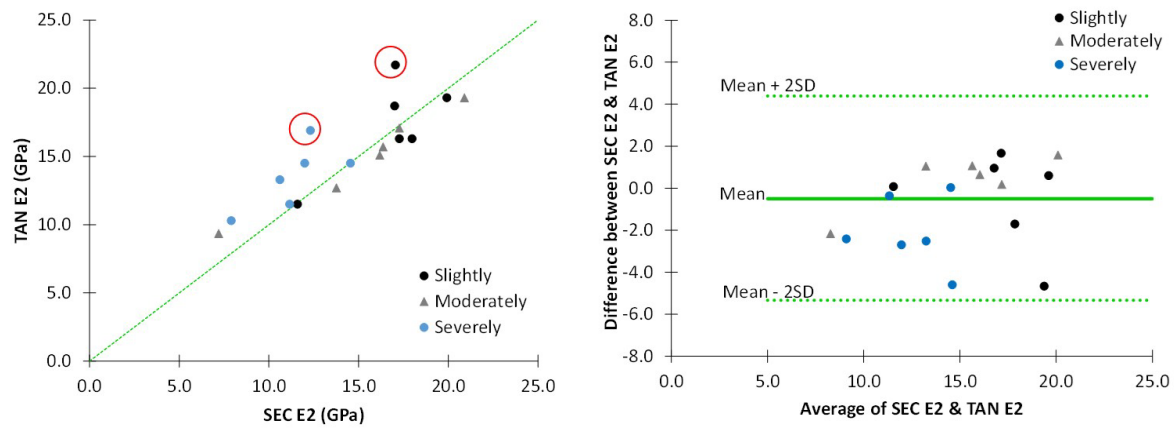


Figure 4-53: Comparison of elastic modulus using secant modulus (E_2) and tangent modulus (E_2) of unloading using 2nd, 3rd, 4th and 5th SDT cycle and a Bland-Altman plot

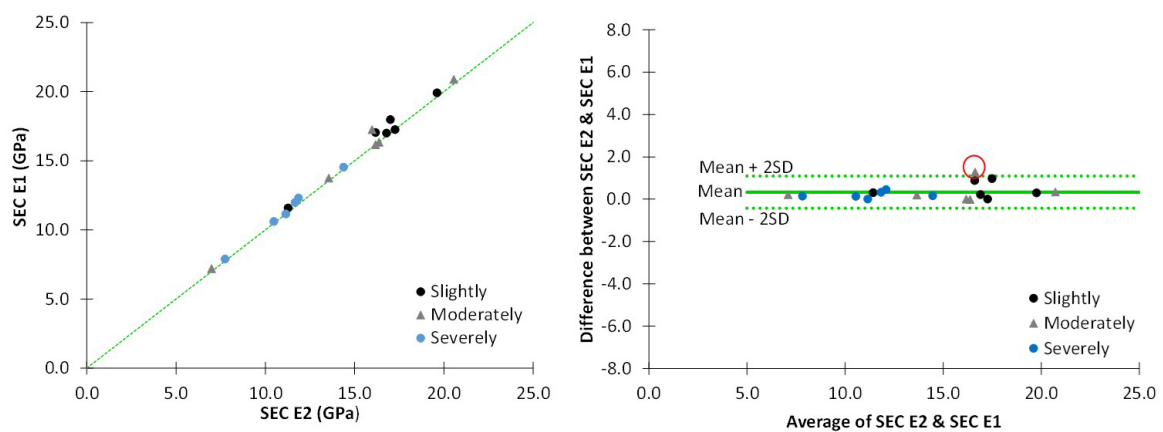


Figure 4-54: Comparison of elastic modulus using secant module of loading (E_1) and unloading (E_2) using 2nd, 3rd, 4th and 5th SDT cycle and a Bland-Altman plot

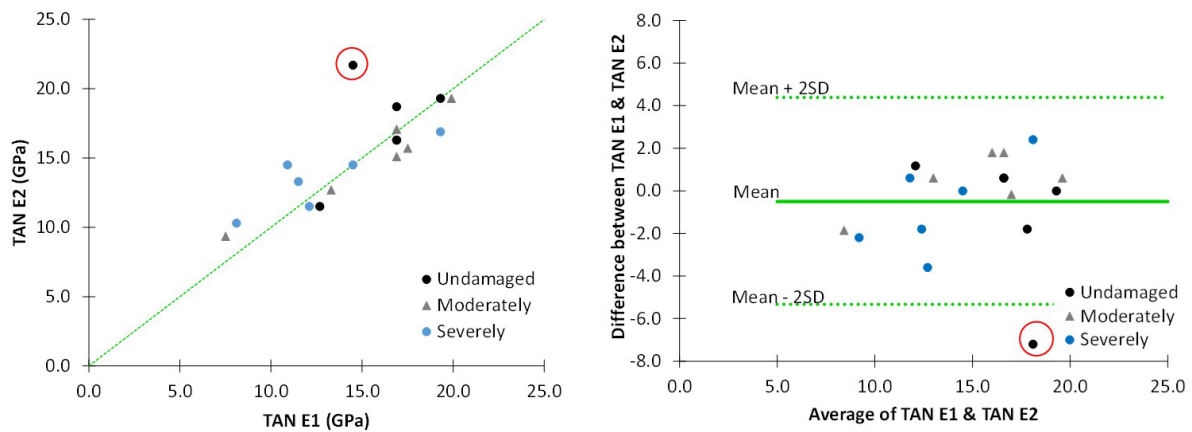


Figure 4-55: Comparison of elastic modulus using tangent modulus of loading (E_1) and unloading (E_2) using 2nd, 3rd, 4th and 5th SDT cycle and a Bland-Altman plot

Figure 4-52 shows the plot of elastic modulus determined using the tangent modulus versus the secant modulus, using the loading SDT curves. The linear correlation between the TAN E_1 and SEC E_1 for concrete taken from undamaged, moderately damaged, and severely damaged regions was obtained with values of 0.88, 0.97, and 0.39. One data point from severely damaged data was identified as an outlier (circled in red), and this outcome was confirmed using the Bland-Altman plot as it lay outside the limits of agreement (indicated by the two green lines showing the mean ± 2 *standard deviation). Twelve of the data points (66.6%) lay on and above the mean value (-0.3), indicating that the secant modulus (E_1) resulted in higher values when compared to the tangent modulus (E_1) of loading SDT curves. Lastly, it can be seen that there is a good agreement between the tangent modulus and the secant modulus, using the loading SDT curves, as all seventeen points lie between the limits of agreement.

In comparison to Figure 4-53, which shows the plot of elastic modulus determined by the tangent modulus versus the secant modulus displayed using the unloading curves, the linear regression values of 0.55, 0.96, and 0.52 were obtained when comparing concrete taken from undamaged, moderately, and severely damaged regions, respectively. Two data points from severely and undamaged areas were identified as outliers (circled in red). However, this outcome was confirmed to be false using the Bland-Altman plot, as both points lay within the limits of agreement. Eleven of the data points (61.1%) lay on and above the mean value (-0.7), indicating that the secant modulus (E_1) resulted in higher values when compared to the tangent modulus (E_1) of unloading SDT curves.

Figure 4-54 shows a strong correlation between the secant modulus E_1 and E_2 for concrete taken from undamaged, moderately and severely damaged regions, which were 0.98, 0.98, and

0.99, respectively. It can be seen that there is a relatively small variability in the results obtained as most of the data points are positioned closely to the mean in the Bland-Altman plot. One data point from moderately damaged data was identified as an outlier (circled in red), and this outcome was confirmed using the Bland-Altman plot as it lay outside the limits of the agreement. Fourteen of the data points (77.7%) lay on and below the mean value (0.3), indicating that the secant modulus (E_2) resulted in higher values than the secant modulus (E_1) of SDT curves. The result can be attributed to the material's stiffening under loading conditions.

Furthermore, when assessing the elastic tangent modulus E_1 and E_2 in Figure 4-55 for concrete taken from undamaged, moderately and severely damaged regions, linear regression values of 0.21, 0.93 and 0.73 were respectively obtained. One data point from severely damaged data was identified as an outlier (circled in red), and this outcome was confirmed using the Bland-Altman plot as it lay outside the limits of agreement. Twelve of the data points (66.6%) lay on and above the mean value (-0.5), indicating that the tangent modulus (E_2) resulted in higher values than the secant tangent (E_1) of SDT curves. Similarly to the second modulus, the result can be attributed to the material's stiffening under loading conditions.

In summary, when the tangent modulus approach was compared to the secant modulus approach for both the loading and unloading scenarios, both scenarios resulted in lower elastic modulus values compared to the secant modulus approach. The tangent approach should be considered, as it results in the worst-case scenario when considering the structural implications and subsequent distribution of stresses in the RC structure. However, the Bland-Altman plots have shown a good agreement between the two approaches (in Figure 4-52, Figure 4-53, Figure 4-54 and Figure 4-55). Thus, the secant modulus approach can be used to determine the elastic modulus, as there is no significant difference between the two approaches.

Furthermore, the loading approach results in lower elastic modulus values when using the loading versus unloading curve approach for both the tangent and secant modulus. Similarly, this approach should be considered, as it results in the worst-case scenario when considering the structural implications and subsequent distribution of stresses in the RC structure. However, the Bland-Altman plots show a good agreement between the two approaches. Thus, the unloading curve can be used to determine the elastic modulus, as there is no significant difference between the two approaches.

Lastly, after the series of SDT tests were completed, the cylindrical test specimens were cut in half and polished to verify the crack orientation, as seen in Figure 4-56.

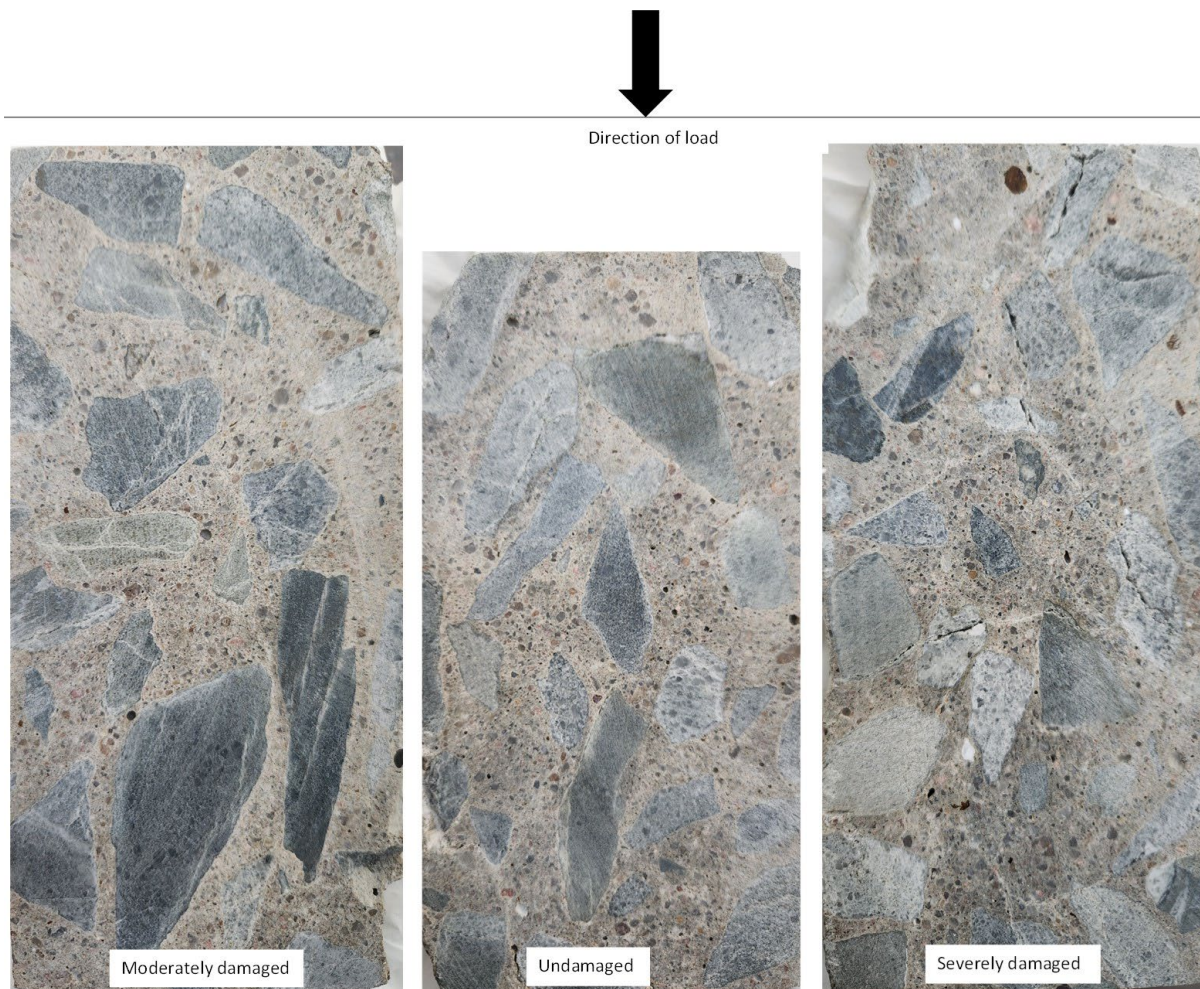


Figure 4-56: Polished SDT specimens showing crack orientation

In Figure 4-56, the orientation of cracking can be seen to be a combination of parallel and perpendicular relative to the direction of the applied load. Although the SDT is considered a non-destructive test, the application of load may have caused existing cracks to propagate and become more visible (with respect to their location and orientation). Thus, it can be seen that several prominent cracks are parallel to the direction of the load, in contrast to what was observed using the 2D images of the circumferential surface of each concrete core (this may have been attributed to the coring process).

In the case of the concrete from a severely damaged region, the parallel crack can be seen running through the coarse aggregates. Based on this observation, the results of obtaining a

convex SDT curve in conjunction with obtaining NLI values in the range of 0.9 and 1 correlate to the observed predominate crack orientation.

4.12 Summary of Phase 2 - Results

In this section, a summary of the findings of Phase 2 is presented, and a number of trends and conclusions are generalised to draw useful information.

- i. Witwatersrand Quartzite was used as aggregate (both coarse and fine) in the production of concrete for the RC railway bridge, a known aggregate type susceptible to ASR
- ii. Petrographic analyses of the aggregates particle and concrete confirmed various diagnostic features of ASR, such as reactive grain particles, limited ASR gel presence, various crack types, and intricate crack networks.
- iii. Micro X-ray fluorescence spectrometry results also affirmed that the primary cause of the deterioration of the RC railway bridge was an alkali-silica reaction. The technique has shown that it is possible to locate and characterise ASR gel, which is not possible with the naked eye based on the elemental distribution (silica, potassium, and calcium) in the concrete matrix.
- iv. The cold water extraction method was used to determine the free alkali metal content inventory with respect to Na^+ , K^+ and Ca^{2+} for concrete extracted from various damaged regions. These results were used to determine the equivalent sodium oxide content for concrete, which showed that it is possible to assess and compare the extent of ASR deterioration damage of concrete based on the consumption of various alkali metals. Concrete from the interior undamaged region exhibited the highest corrected sodium oxide equivalent content of $3.2 \text{ kg/m}^3 \text{ Na}_2\text{O}_{\text{eq}}$; in contrast, concrete from the interior damaged region exhibited the corrected sodium oxide equivalent content of $0.1 \text{ kg/m}^3 \text{ Na}_2\text{O}_{\text{eq}}$.
- v. Furthermore, the results from the cold water extraction method were useful in determining the composition of the pore solution used to manufacture the synthetic pore solution required to carry out the residual bulk expansion test in phase 3. The theoretical pore solution for the concrete, based on sodium oxide equivalent content (accounting for alkali release from aggregates), amounted to $3.16 \text{ kg/m}^3 \text{ Na}_2\text{O}_{\text{eq}}$.
- vi. The stiffness damage test used in this study has shown that it is possible to assess the stiffness and compare the extent of damage of concrete taken from various locations of a RC railway bridge using various test outputs: modulus of elasticity (E-Mod),

hysteresis area (HA), stiffness damage index (SDI), plastic damage index (PDI), and non-linearity index (NLI). Compared to concrete from the undamaged area of 18.0 GPa, there was a 22.2% and 31.1% reduction in elastic modulus for the concrete taken from the moderately and severely damaged regions.

- vii. The stiffness damage test also compared the extent of damage to concrete when subjected to wetting and drying cycles. A general trend observed for all three concrete types (undamaged, moderately and severely damaged) was that more energy was dissipated per volume of concrete when comparing the hysteresis area measured for dry conditions compared to previous moist conditions. The elastic modulus of concrete was generally higher under wet conditions than in dry conditions due to the influence of pore pressure and existing ASR gel, resulting in additional stiffness. Interestingly, the elastic modulus increased throughout the test duration of three saturated and three dry cycles, indicating that the material became stiffer over time. The result was attributed to the orientation of the cracks relative to the loading, which was confirmed as predominately parallel.
- viii. Lastly, the two numerical approaches, the tangent modulus and the secant modulus approach, were used to determine the elastic modulus utilising the output of SDTs. The Bland-Altman plots showed a good agreement between the methods used to determine the modulus of elasticity.

The next section deals with the results and analysis of the residual potential strain, on a micro and macro scale, of concrete taken from the various locations of the RC railway bridge.

Phase 3 - Results & Discussion

4.13 Macro residual expansion of field concrete

Figure 4-57Figure 4-57 illustrates the average longitudinal expansion measurements on core specimens C1, C2, C6, C7, C9 and C12 over 112 days, as well as the associated mass gain of the specimens in Figure 4-58. Each average longitudinal expansion point on the graph is an average of four readings.

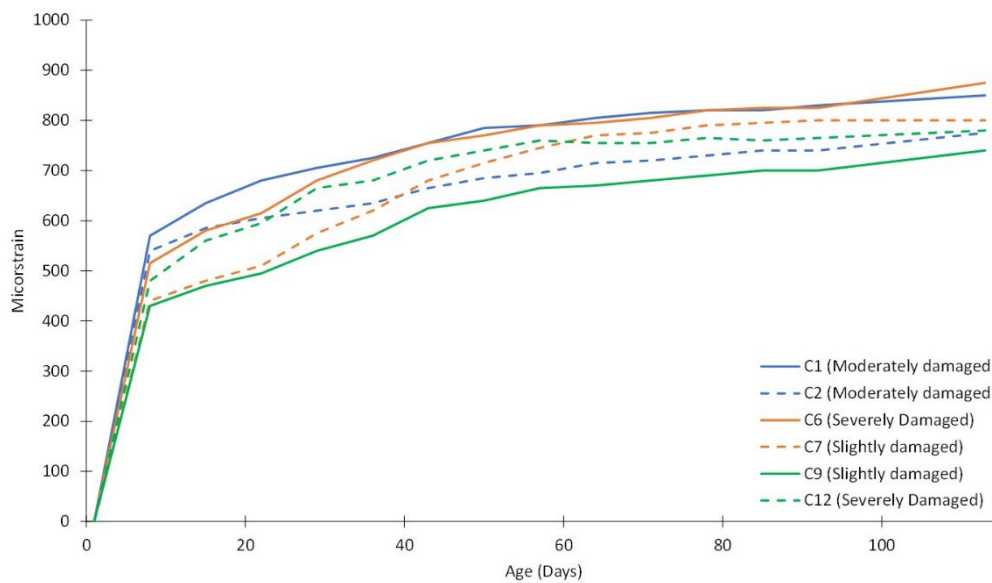


Figure 4-57: Average longitudinal expansion measurements from bulk residual expansion tests, with specimens saturated in fabricated pore solution, kept at 38°C

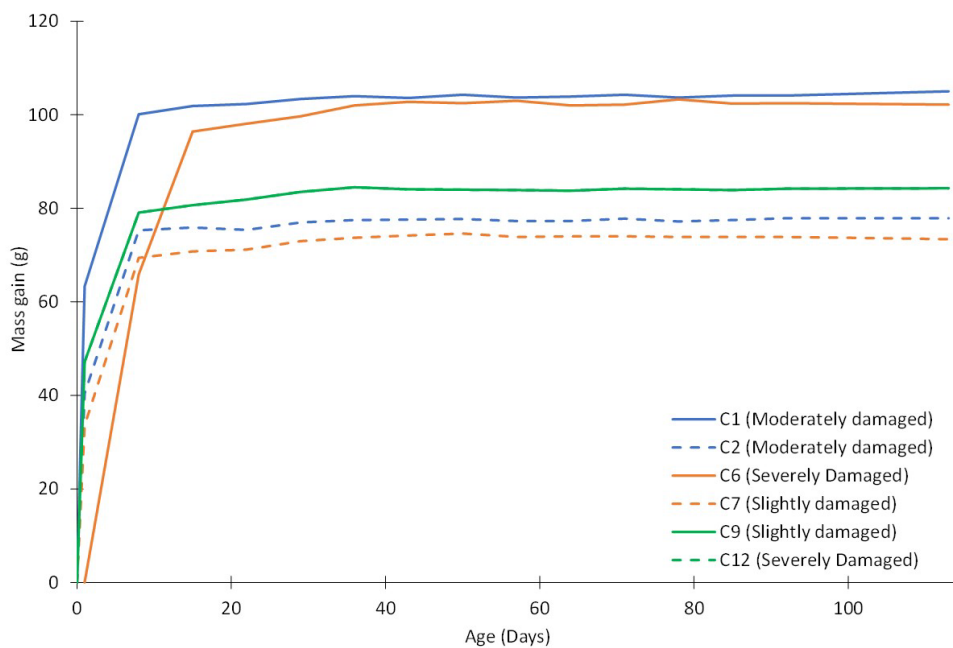


Figure 4-58: Specimen weight gain from bulk expansion tests, with specimens saturated in fabricated pore solution, kept at 38°C

Unlike the characteristic expansion S-curve trend, which demonstrated low initial expansion and accelerated expansion, the specimens in Figure 4-57, which are already subject to ASR deterioration, all show a rapid initial expansion (over the first seven days), followed by a decreased expansion reaction rate. The rapid expansion was attributed to cores being taken from a relatively dry location and the subsequent wetting of the pre-existing gel.

The strain measurement in the residual expansion test represents the maximum potential residual strain for the concrete, as the temperature and relative humidity during the test are more favourable for ASR development than the conditions generally present in the field structure. It can be seen that the maximum longitudinal strain recorded over the test duration ranged from $830\mu\epsilon$ to $700\mu\epsilon$. Initially, a rapid expansion was observed over the first seven days of 68%, 72%, 62%, 55%, 61% and 63% of the total strain measured for cores C1, C2, C6, C7, C9 and C12, respectively. This rapid expansion period also coincides with significant water uptake for each specimen, as shown in Figure 4-58. Subsequently, the expansion rate slowed down, as noted by the slope of the expansion curves to 3.1%, 2.4%, 3.7%, 4.3%, 3.2% and 3.4% for cores C1, C2, C6, C7, C9 and C12 respectively.

It can be seen that cores C6 (damaged region) and C1 (moderately damaged region) exhibited the greatest potential residual strains of 875 and $850\mu\epsilon$, respectively, over 112 days. Furthermore, both cores also showed the greatest mass gain over the test duration; this can be attributed to the extent of damage of both cores, with regard to the presence of several cracks allowing the subsequent water uptake, facilitating ASR expansion. In contrast, Core C9, extracted from an undamaged area, exhibited the lowest potential residual strain value over the test duration.

Surprisingly, the analysis correlated higher potential residual strain values and more extensive damage in areas like Pier 1 (North and South) as observed in studies conducted by Roy, Unobe and Sorensen, (2022) and Zhou et al., (2023a, 2023b). Conversely, lower potential residual strain values in areas of minimal damage, as observed in Abutment 1's inner and Pier 1's. These results do not follow trends in literature whereby higher expansion levels are associated with concrete taken from undamaged areas. There are two possible reasons for the greater residual expansion exhibited in concretes from extensive damaged areas may be attributed to the increased permeability of the concrete, owing to the damage induced by ASR. The increased moisture to reactive silica sites and free alkalis in the simulated pore solution facilitated increased ASR expansion. Secondly, the alkali solution used to soak the specimens was not

aggressive enough to cause the formation of new, extensive ASR gel. The solution mimicking the pore solution will cause ASR gel formation at a slow rate, similar to in-situ conditions; this will possibly require one year of soaking. The solution made moisture readily available for the expansion of existing ASR gel; hence, ‘slightly damaged’ samples showed the least expansion from the lower amounts of the existing ASR gel. Further investigation is required to validate this hypothesis

Subsequently, it was not possible to contextualise and frame potential residual expansion with respect to the damaged concrete with respect to undamaged concrete from the in-service RC railway bridge, as anticipated for two reasons. Firstly, the concrete from damaged regions exhibited greater bulk expansion strains than concrete from undamaged regions. Secondly, when considering the difference in bulk expansion strain measured from a specimen that generated the highest and compared to the lowest, the difference of $135\mu\epsilon$ is relatively insignificant over the 112 days.

Lastly, the measured bulk potential residual expansion readings were used as input to determine the structural severity of the RC railway bridge structure subject to ASR deterioration as outlined in Section 2.12. It must be noted that a structural severity rating is typically determined for each structural member. However, in this instance, the approach was considered a guideline to estimate the overall current severity of ASR deterioration in the elements investigated in the RC railway bridge.

Johannesburg’s relative humidity ranges from approximately 73% in summer to 47% in winter. Thus, the RC railway bridge site condition was classified as dry, as the relative humidity was less than 75%. The RC railway bridge is well-anchored, so it is classified under Reinforcement Detailing Class 1. It can be seen that, irrespective of the Expansion Index, the structure is assigned a negligible severity rating, as seen in Table 4-10 (highlighted in red).

Table 4-10: Structural severity of RC railway bridge subject to ASR deterioration

Site Environ ment	Reinforc ement detailing class	Expansion Index									
		I $\varepsilon ASR\% < 0.06\%$		II $0.06\% \leq \varepsilon ASR\% < 0.10\%$		III $0.10\% \leq \varepsilon ASR\% < 0.15\%$		IV $0.15\% \leq \varepsilon ASR\% < 0.25\%$		V $\varepsilon ASR\% \geq 0.25\%$	
		Consequence of failure: Slight or Significant									
		Slight	Sign.	Slight	Sign.	Slight	Sign.	Slight	Sign.	Slight	Sign.
Dry	1	N	N	N	N	N	N	N	N	N	N
	2	N	N	N	N	N	N	N	N	N	Mi
	3	N	N	N	N	N	N	N	Mi	Mi	Mo
Interme diate	1	N	N	N	Mi	Mi	Mo	Mi	Mo	Mi	Mo
	2	N	N	Mi	Mo	Mi	Mo	Mo	Mo	Mo	S
	3	N	Mi	Mi	S	Mo	S	S	VS	S	VS
Wet	1	Mi	Mi	Mi	Mo	Mi	Mo	Mo	S	Mo	S
	2	Mi	Mi	Mo	S	Mo	S	S	S	S	VS
	3	Mi	Mo	Mo	VS	S	VS	VS	VS	VS	VS

*Structural severity ratings: N = Negligible; Mi = Mild; Mo = Moderate S = Severe; VS = Very Severe;

From the Table 4-10, it appears that the railway bridge does not require immediate attention regarding structural repairs or rehabilitation. However, this is an oversimplified conclusion; additional testing, such as loading and analysis of the behaviour of the structures, is required (Ye *et al.*, 2021). However, routine structural inspections should still be conducted at least once every two years, which is the generally accepted frequency for railway bridges. Limited moisture access is critical to limiting ASR deterioration in existing RC structures. As seen in the bulk expansion results, even in areas that displayed severe ASR distress, the concrete may still exhibit substantial residual expansion in the future.

4.14 Micro residual expansion of field concrete

4.14.1 DIC approach

As noted in Section 0, the attempt to employ DIC to quantify and assess the residual expansion strain on a local micro-scale proved unsuccessful. Demec gauge readings were then used to capture strain readings on a micro-scale.

4.14.2 Demec gauge approach

Table 4-11 presents the total strain recorded over 112 days for the various concrete discs with respect to the established 90° strain rosette arrangement.

Table 4-11: Total strain ($\times 10^{-6}$) measured over 112 days

Concrete Disc ID	Strain measured relative to 90 °rosette arrangement		
	0	45	90
1B	-	-	-
1D	600	800	775
2B	450	525	525
2D	400	425	400
6B	350	150	450
6D	300	475	600
7B	325	275	300
7D	350	500	475
9B	325	350	350
9D	375	325	450
12B	275	350	425
12D	375	500	650

The values from Table 4-11 were used to determine the principal strains (normal and shear) and the orientation of the respective strains determined for each disc of concrete, which have been overlaid on reference concrete images, in Figure 4-59 to Figure 4-64. The strain state of the concrete disc corresponds to the last readings taken over 112 days. For each respective core, “B” refers to the discs close to the exterior surface while “D” refers to discs taken further into the bulk of the concrete.

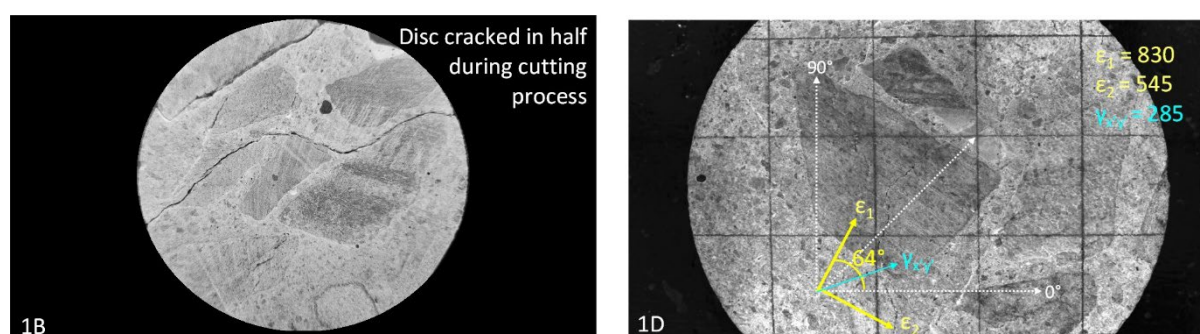


Figure 4-59: Potential residual principal normal and shear strains generated in concrete from C1- Abutment 1, Interior Wall (moderately damaged area)

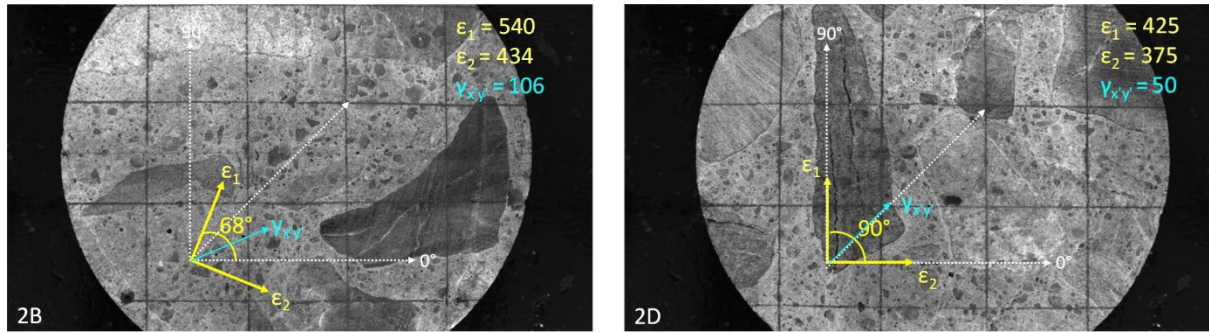


Figure 4-60: Potential residual principal normal and shear strains generated in concrete from C2 - Abutment 1, Interior Wall (moderately damaged area)

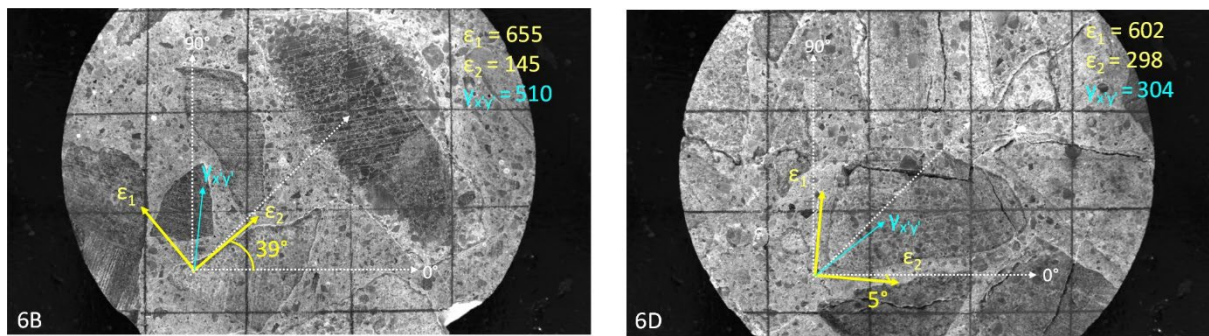


Figure 4-61: Potential residual principal normal and shear strains generated in concrete from C6 - North facing view of Pier 1 outer walls (severely damaged area)

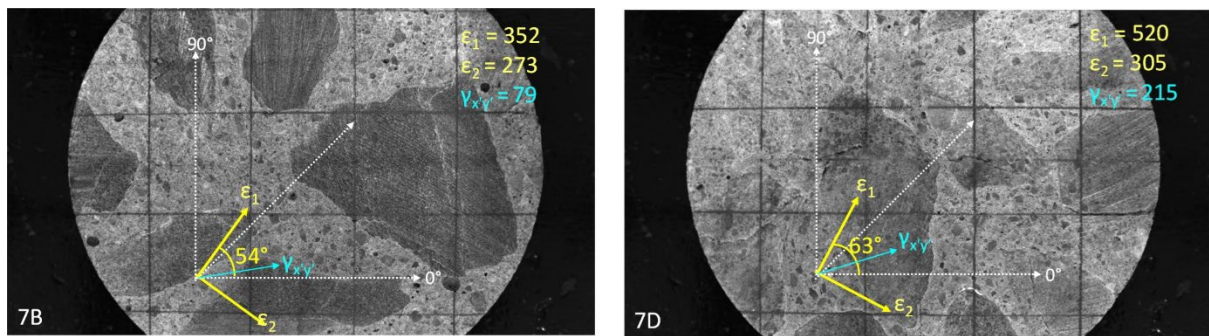


Figure 4-62: Potential residual principal normal and shear strains generated in concrete from C7 - Pier 1, Interior wall (undamaged area)

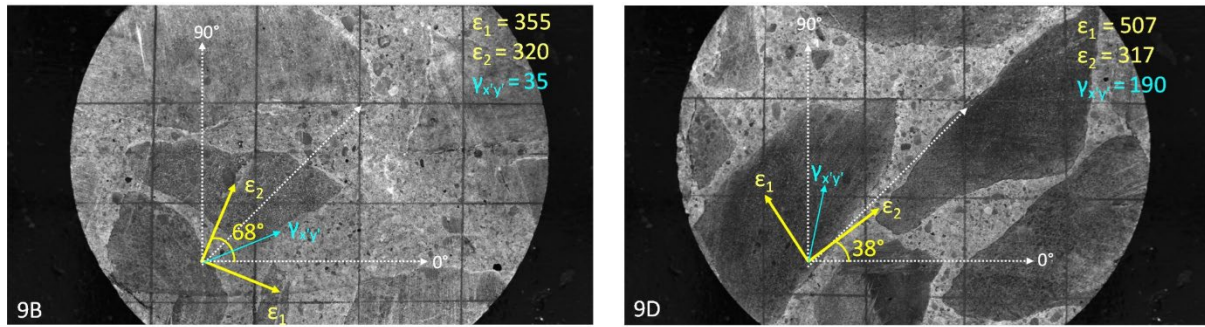


Figure 4-63: Potential residual principal normal and shear strains generated in concrete from C9 - Pier 1, Interior wall (undamaged area)

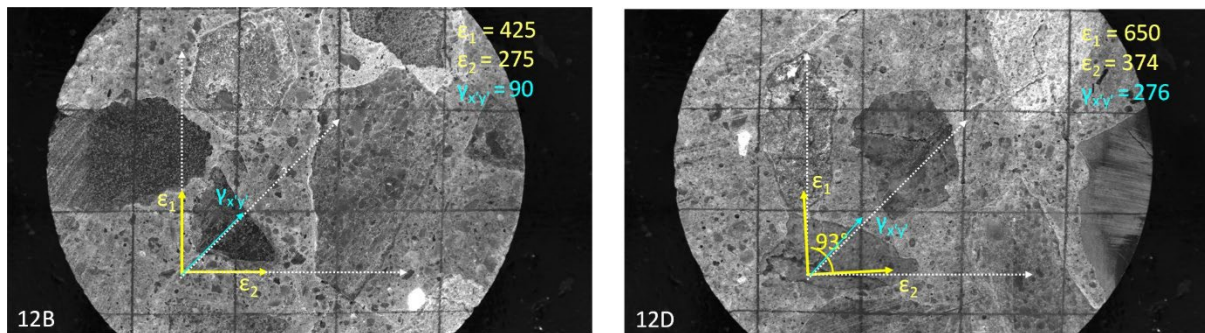


Figure 4-64: Potential residual principal normal and shear strains generated in concrete from C12 - South facing view of Pier 1 outer walls (severely damaged area)

The limits of traditional strain measurements are evident in Figure 4-59 to Figure 4-64. The respective axis (0° , 45° , 90°) on which the strain measurements were taken fell across various phases of the concrete matrix, yet only a single value was obtained. Mohr's circle graphically illustrated and compared the potential residual strain generated from the respective concrete discs at 112 days, as seen in Figure 4-65.

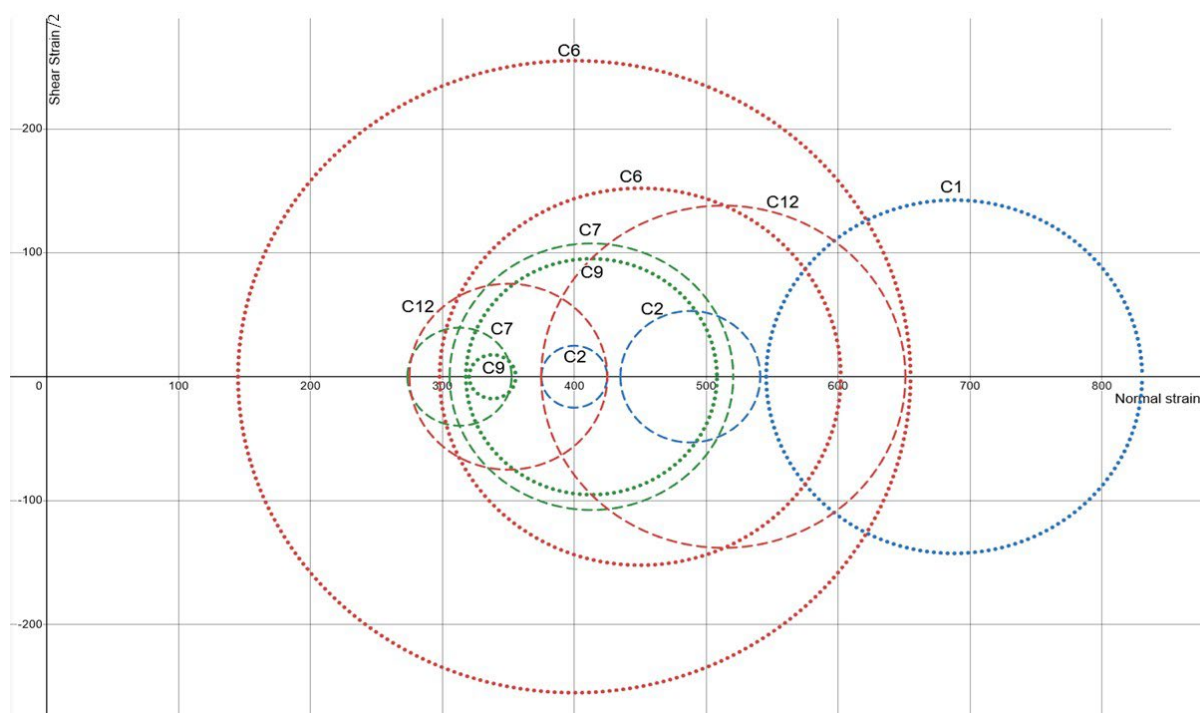


Figure 4-65: Mohr's circle comparing potential residual strain of concrete subject to ASR

Figure 4-65 shows that all concrete specimens experienced tensile strains, as expected in concrete subject to ASR deterioration. The Mohr's circles for the state of strain for concrete from undamaged, moderately and severely damaged areas are plotted in green, blue and red, respectively. The normal strain ranges from 830 $\mu\epsilon$ to 145 $\mu\epsilon$, whilst the shear strain ranges from 510 $\mu\epsilon$ to 50 $\mu\epsilon$ (the shear strain plotted is divided by two). Core C1 produced the greatest normal strain, whilst core C6 produced the greatest shear strain.

Figure 4-66 to Figure 4-68 illustrate the state of strain for concrete discs from the various locations of the RC railway bridge. Each plot also included the approximate tensile strain limit of 200 $\mu\epsilon$ for concrete (shown in orange).

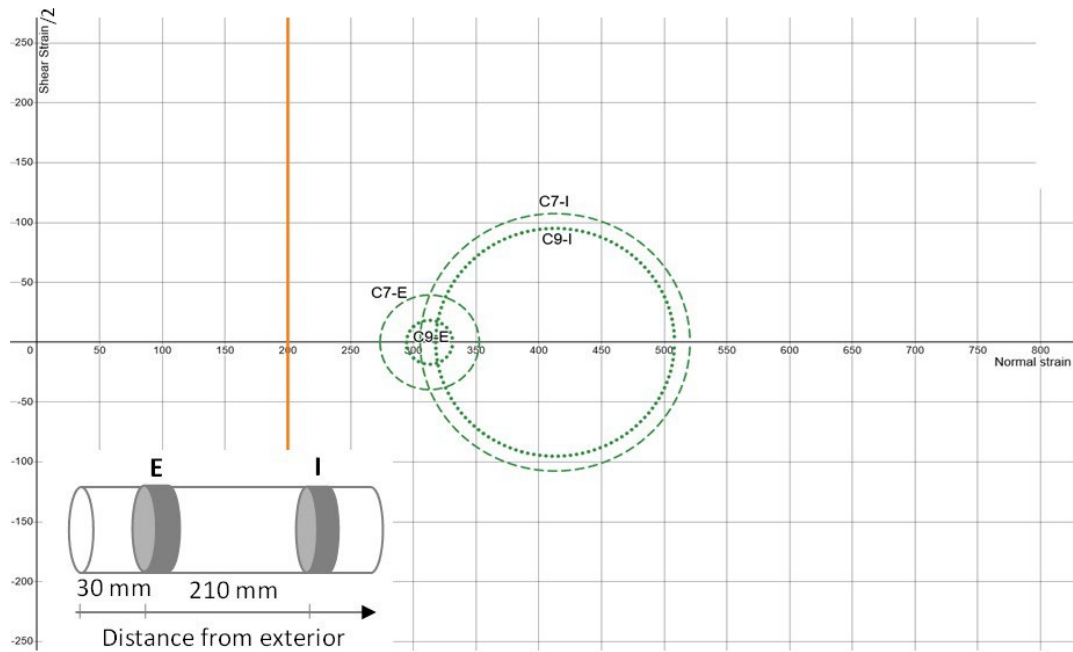


Figure 4-66: Mohr's circle comparing potential residual strain of concrete subject to ASR from undamaged region

Considering the state of strain for concrete from the undamaged region in Figure 4-66, it can be seen that the maximum normal strain was $507 \mu\epsilon$ while the maximum shear strain was $351 \mu\epsilon$. In both cores C7 and C9, the interior discs experienced greater normal strains, which were 47% and 44% higher than those closer to the exterior surface. The shear strain was also greater in the inner disc than the exterior discs for Core C7 and C9 by 2.72 and 5.39 times, respectively. Normal principle strains (ϵ_1 and ϵ_2) in all four concrete discs exceeded the approximate tensile strain limit of $200 \mu\epsilon$ for concrete.

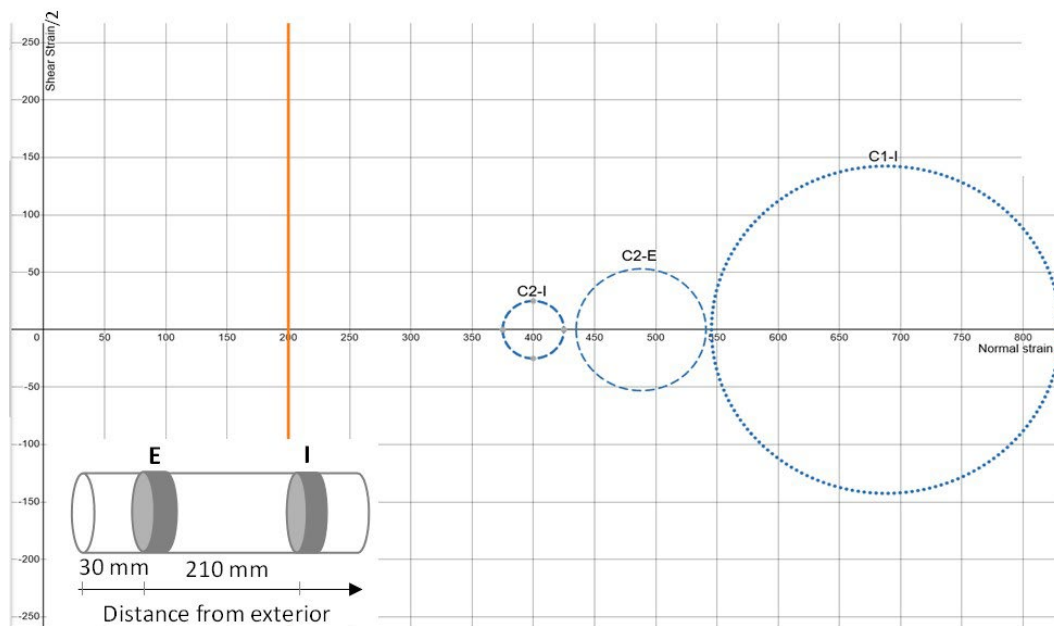


Figure 4-67: Mohr's circle comparing potential residual strain of concrete subject to ASR from the moderately damaged region

In contrast to concrete from the slightly damaged region, in Figure 4-67, it can be seen that the maximum normal strain was $830 \mu\epsilon$ while the maximum shear strain was $285 \mu\epsilon$ in core C1-1. In the case of core C2, the disc closer to the exterior surface displayed the greatest normal and shear strain. This trend does not align with what was observed in concrete from the undamaged region. Normal principal strains in all three concrete discs exceeded the tensile strain limit of $200 \mu\epsilon$ for concrete; however, in this instance, the principle strain ϵ_1 was 4.15, 2.7 and 2.1 times greater than the tensile strain criteria of $200 \mu\epsilon$ for discs C1-I, C2-I and C2-E respectively.

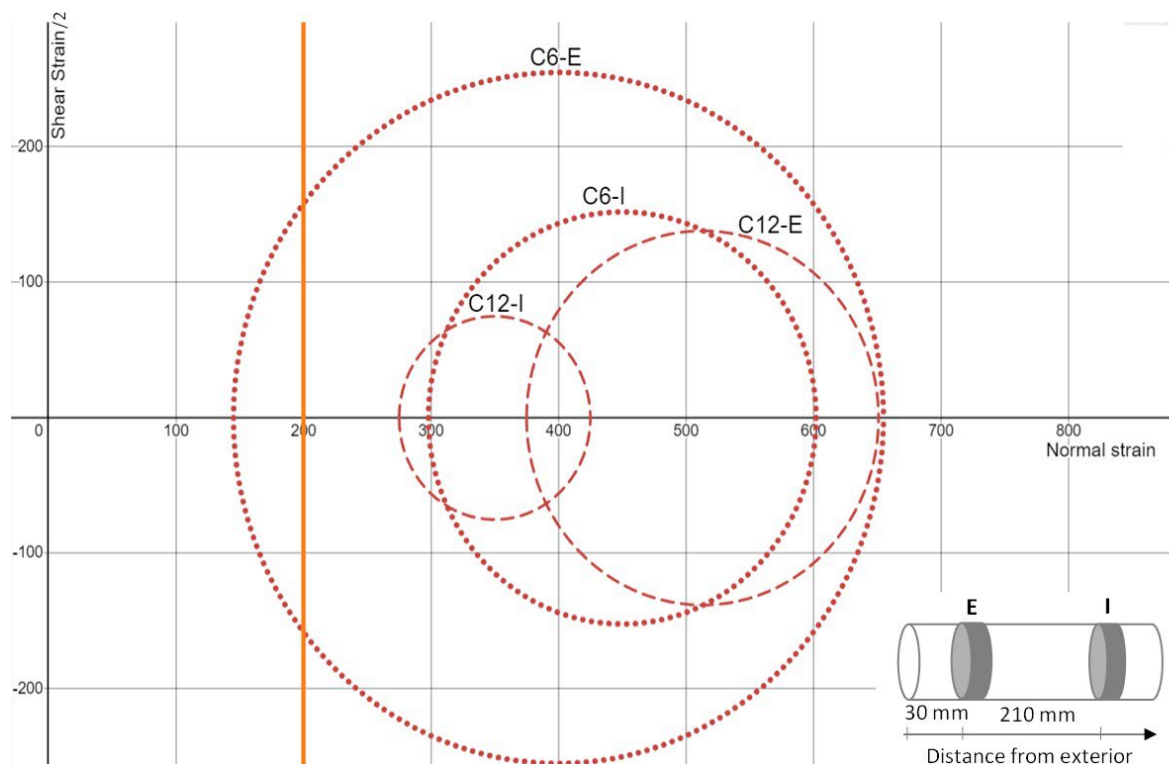


Figure 4-68: Mohr's circle comparing potential residual strain of concrete subject to ASR from the damaged region

Lastly, considering the strain in concrete from the damaged region, Figure 4-68 showed that the maximum normal strain was $655 \mu\epsilon$ while the maximum shear strain was $510 \mu\epsilon$. In both cores C6 and C12, the exterior disc experienced greater normal and shear strains than the interior disc. Normal strains in all four concrete discs exceeded the tensile strain limit of $200 \mu\epsilon$ for concrete; however, in this instance, the principle strain ϵ_1 was 3.2, 3.0, 2.1 and 3.2 times greater than the tensile strain criteria of $200 \mu\epsilon$ for discs C6-I, C6-E, C12-I and C12-E respectively.

Surprisingly, the analysis established a correlation between higher potential residual strain and more extensive damage in areas like Pier 1 (North and South). Conversely, lower potential residual strains were generated in areas with minimal damage, as observed in the inner walls

of Abutment 1 and Pier 1. This result may be attributed to the increased permeability of the damaged concrete disc, which, owing to the presence of cracks and thickness of the disc, facilitated the ingress of moisture and caused the swelling of existing ASR gel in the concrete.

Furthermore, it was noted that despite lower sodium and potassium alkali reserves in the concrete from the damaged region (Section 4.12), the concrete still exhibited the highest residual potential expansion. These results do not follow trends in literature, where higher expansion levels are associated with concrete taken from undamaged areas. However, the same trend was observed in the bulk residual potential expansion results.

Similarly, Figure 4-69, Figure 4-70, Figure 4-71, Figure 4-72, Figure 4-73 and Figure 4-74 assessed the development of local strain in concrete subject to ASR deterioration over 112 days. For a given concrete core, the disc closer to the exterior surface was drawn in green, and the disc closer to the bulk of the concrete was drawn in red.

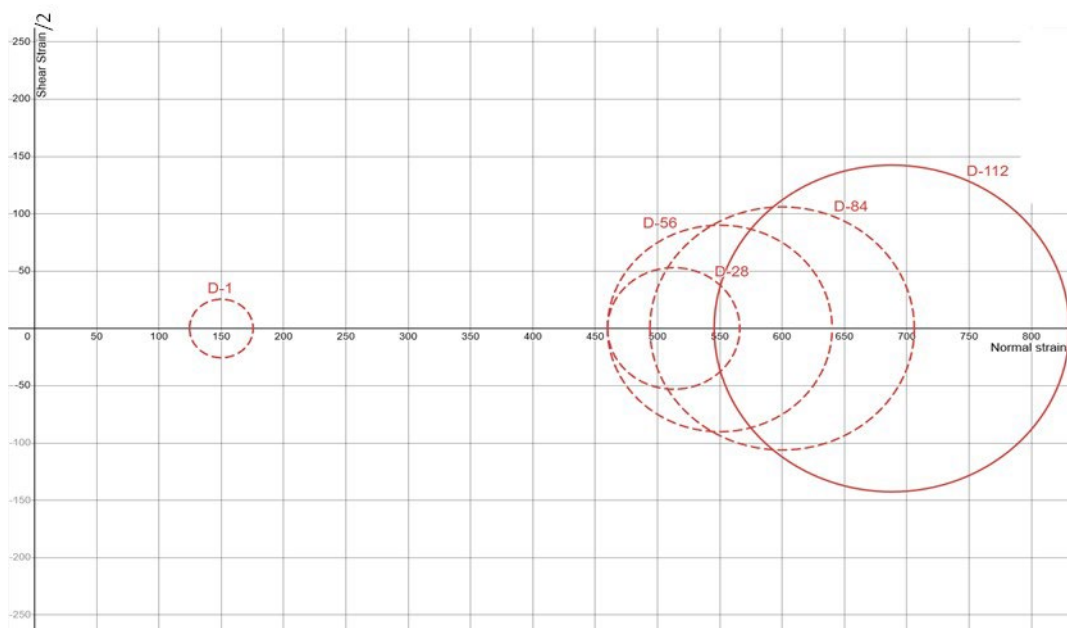


Figure 4-69: Development of local potential residual strain of concrete subject to ASR at days 1, 28, 56, 84 and 112 in core C1

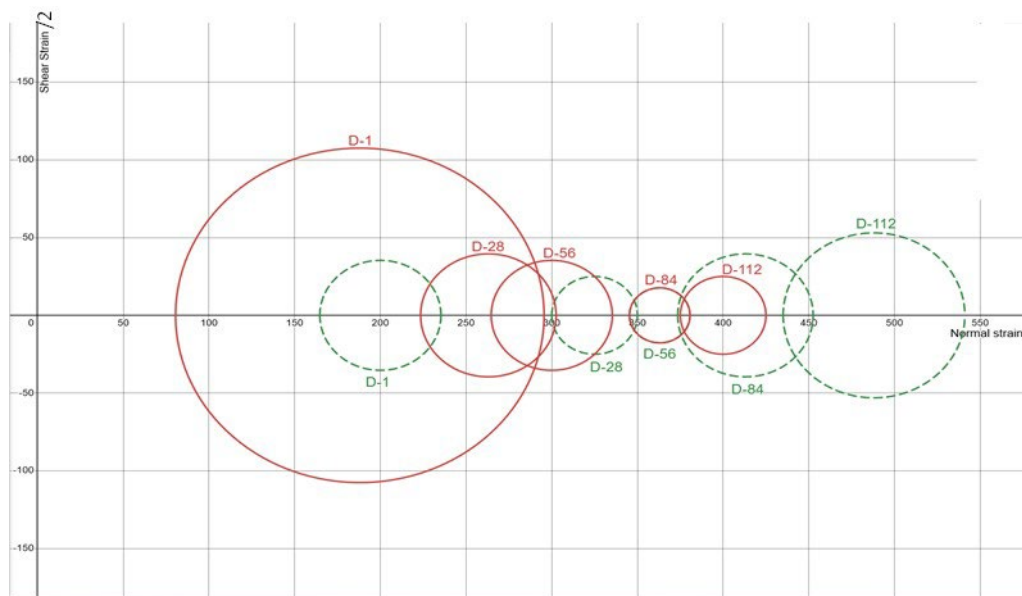


Figure 4-70: Development of local potential residual strain of concrete subject to ASR at days 1, 28, 56, 84 and 112 in core C2

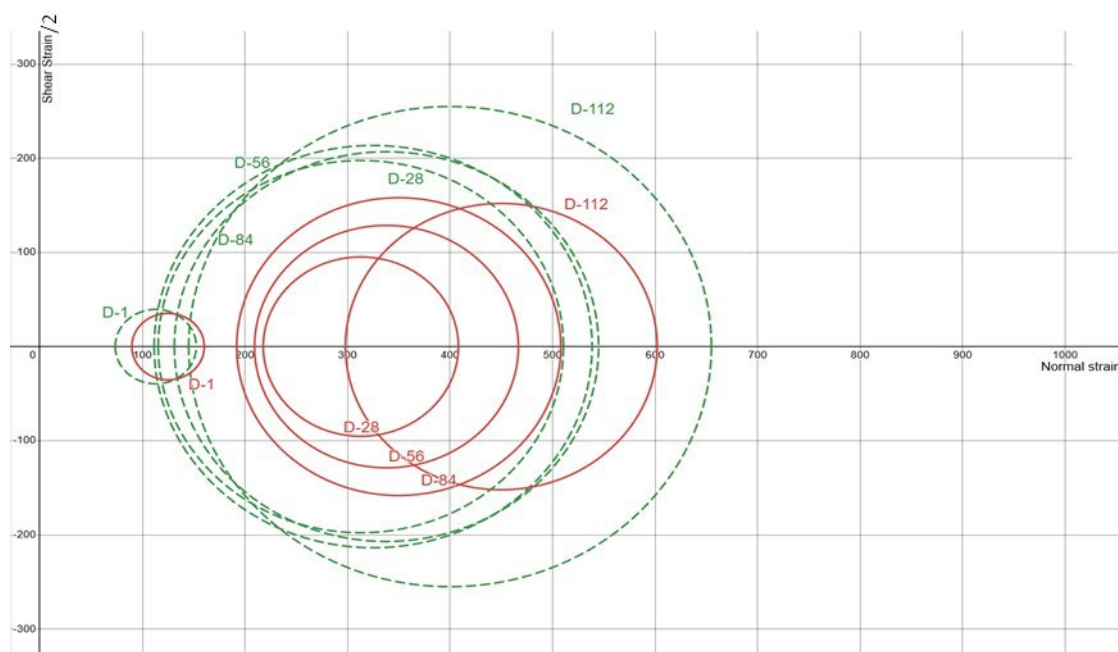


Figure 4-71: Development of local potential residual strain of concrete subject to ASR at days 1, 28, 56, 84 and 112 in core C6

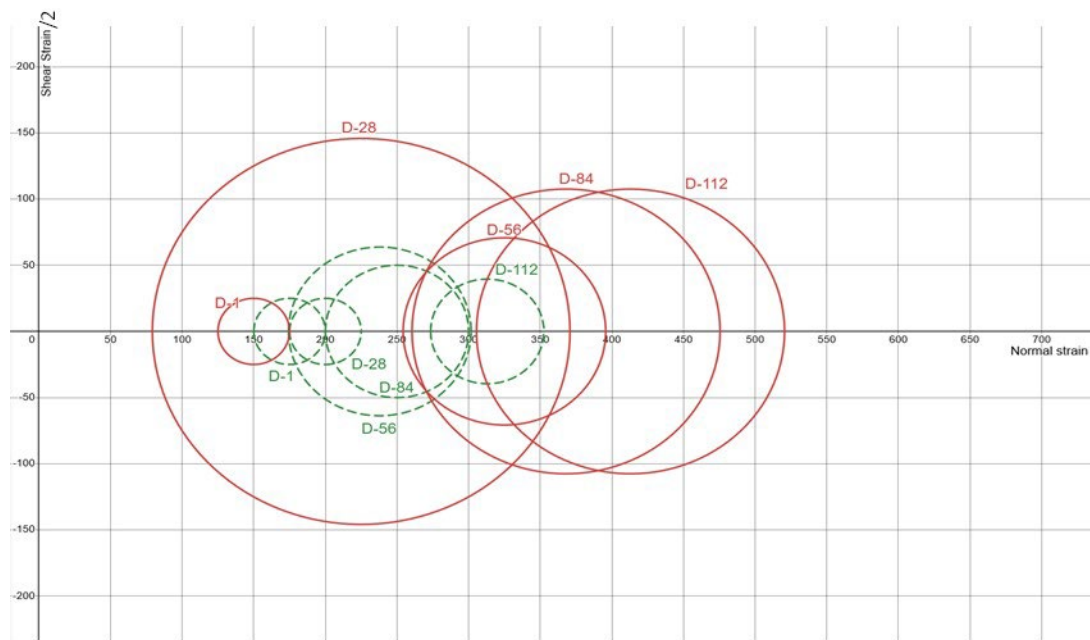


Figure 4-72: Development of local potential residual strain of concrete subject to ASR at days 1, 28, 56, 84 and 112 in core C7

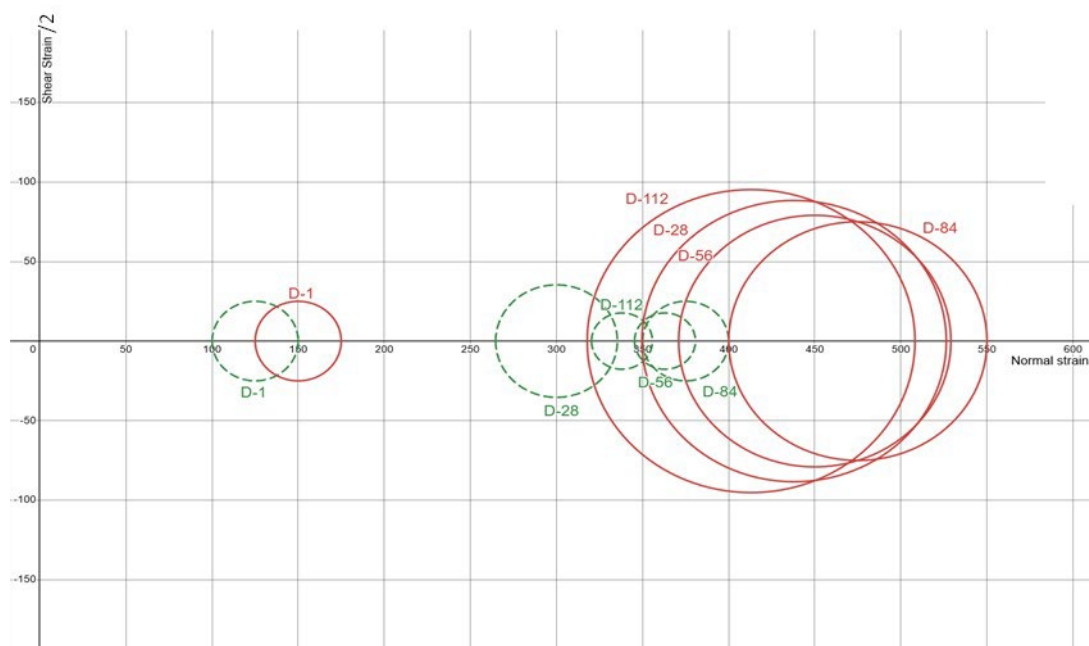


Figure 4-73: Development of local potential residual strain of concrete subject to ASR at days 1, 28, 56, 84 and 112 in core C9

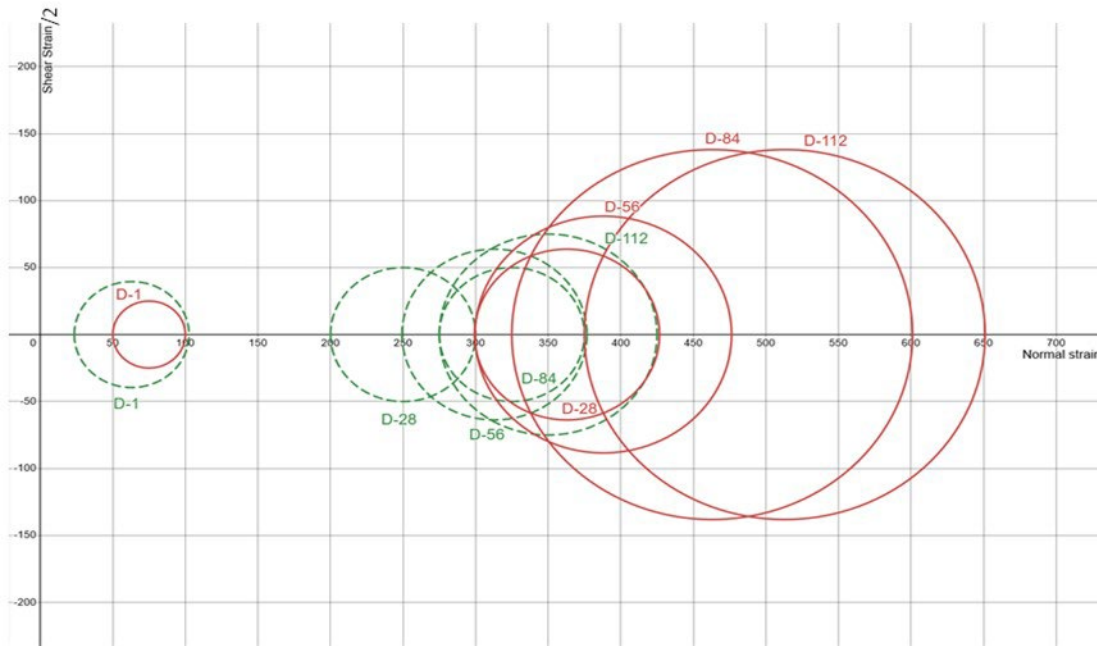


Figure 4-74: Development of local potential residual strain of concrete subject to ASR at days 1, 28, 56, 84 and 112 in core C12

Over the course of 112 days, the normal tensile strains of all the concrete specimens examined increased with time, as evidenced by the shift of their respective Mohr circles along the x-axis. However, with respect to shear strains, there was no clear pattern, with some concrete showing an increase in strain and others showing a decrease over time. The increase in shear strain may be attributed to the increased ASR swelling pressures leading to an increase shear strain at the ITZ. The decreased in shear strains may be attributed to the subsequent development of cracks, owing to the increased ASR swelling pressures leading to a reduction in shear strain at the ITZ. Furthermore, the configuration of aggregates in the concrete matrix, the location of ASR gel and restraint imposed by steel rods may all contribute to strain experienced in the concrete. Further investigation is required to validate this hypothesis.

4.15 Micro versus macro potential residual strain

Figure 4-75 and Figure 4-76 display plots of local normal and shear strain equivalents alongside bulk expansion strain in concrete subjected to ASR deterioration. The comparison between local strain equivalents and bulk strain was determined using a line with a slope equal to 1 (equality line).

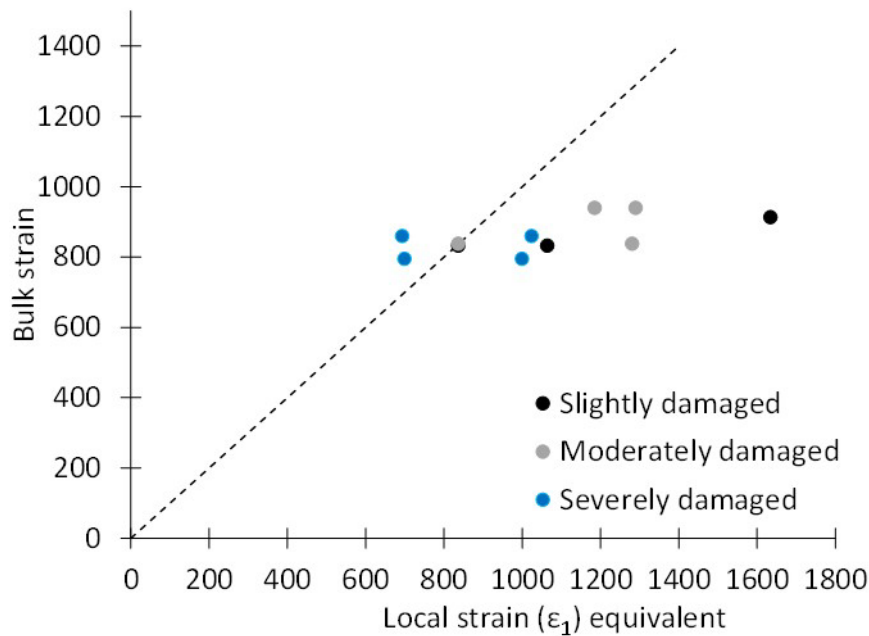


Figure 4-75: Potential residual local normal strain equivalent versus potential residual bulk strain

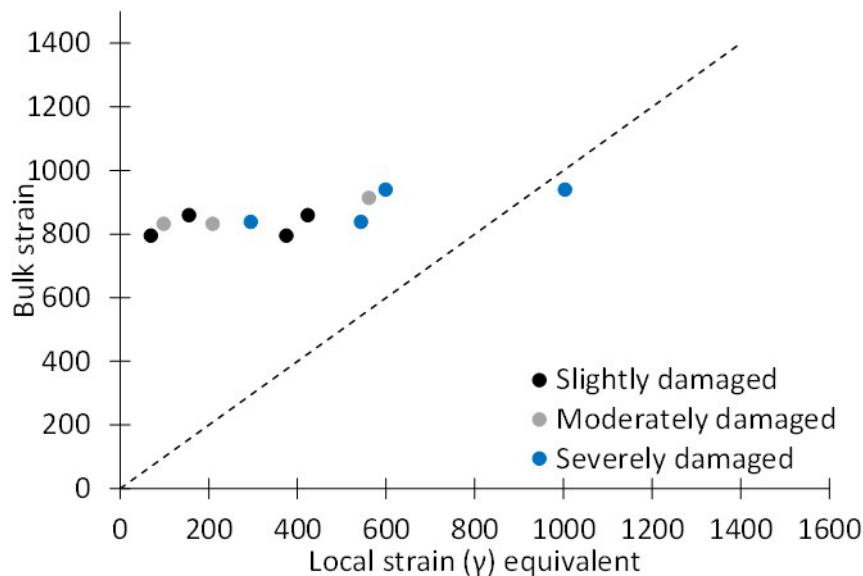


Figure 4-76: Potential residual local shear strain equivalent versus potential residual bulk strain

Based on the data presented in Figure 4-75, there is a positive correlation between potential residual local normal equivalent strain and bulk strain. Seven out of eleven data points were located to the right of the equality line, indicating that local equivalent normal strains were generally higher than respective bulk expansion strains.

The highest difference observed was that the local equivalent normal strain was 1.8 times greater than the bulk expansion strain.

In contrast, when analysing the local shear equivalent and bulk strains in Figure 4-76, ten of the eleven data points were located to the left of the 45-degree line. This implies that local equivalent shear strains were generally lower than respective bulk strains. The highest difference observed in this scenario was that the local equivalent shear strains were 11.4 times lower than the respective bulk expansion strains.

In summary, the analysis indicates that local normal equivalent strains tend to be higher than bulk expansion strains, while local shear equivalent strains are generally lower than bulk strains. These findings suggest a significant difference in how normal and shear strains behave locally compared to their bulk material, this highlights the importance of considering local strain variations in material analysis.

4.16 Summary of Part 3 - Results

In this section, a summary of the findings of Phase 3 is presented, and a number of trends and conclusions are generalised to draw useful information.

- i. The bulk expansion method used in this study has shown that it is possible to assess and compare the extent of potential residual expansion of concrete taken from various locations of an RC structure. The analysis correlated higher potential residual strain values and more extensive damage in areas like Pier 1 (North and South). Conversely, lower potential residual strain values in areas of minimal damage, as observed in Abutment 1's inner and Pier 1's.
- ii. The attempt to evaluate local strain development over a 2 cm square region of concrete affected by ASR using digital image correlation technique was unsuccessful.
- iii. The strain rosette arrangement using 0, 45, and 90 degrees has shown that it is possible to quantify and compare the extent of local potential residual expansion of concrete taken from various locations of an RC railway bridge. A similar correlation to the bulk expansion was established, where higher potential residual strain values correlated to more extensive damage in areas like Pier 1 (North and South).

- iv. Concrete extracted from a moderately damaged region exhibited the highest degree of local potential residual normal strain, while concrete extracted from a damaged region exhibited the highest degree of local potential residual shear strain.
- v. Lastly, a non-linear positive correlation was established between local normal equivalent strain and bulk strain, similarly a non-linear positive correlation between local shear equivalent strain and bulk strain.

4.17 Synthesis of findings - phase 1 to phase 3

The results from the three phases of this study are synthesised to obtain a holistic view of the interrelationships between the extent of ASR deterioration in the 78-year-old RC structure investigated. Figure 4-77 illustrates the five assessments considered (and the order in which they were conducted) in this study to characterise the extent of ASR deterioration. Additional assessments/tests were also performed, such as petrography and the use of a micro X-ray fluorescence spectrometer. However, these tests were focused on confirming ASR as the underlying form of deterioration in the structure.

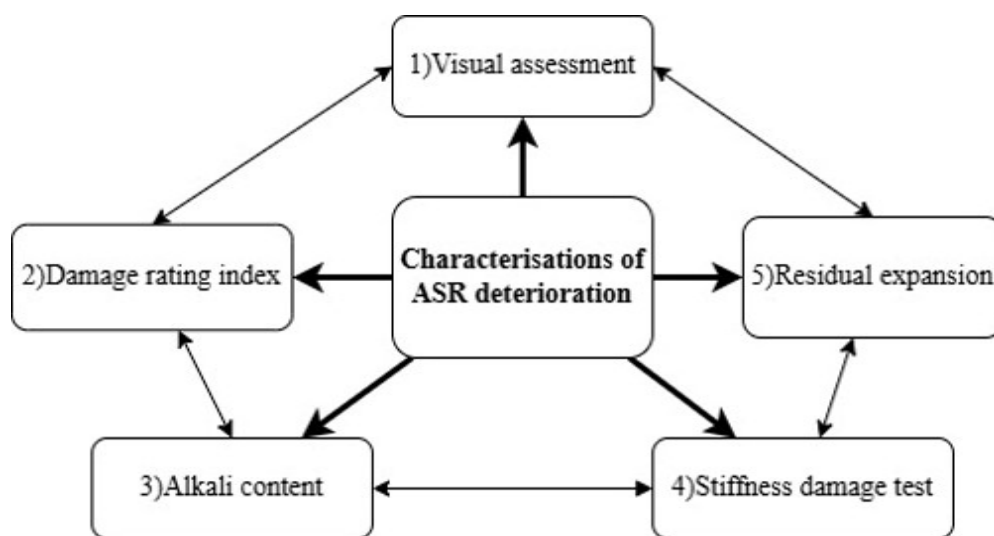


Figure 4-77: Schematic illustrating five assessments conducted to characterise the extent of ASR deterioration in a 78-year-old RC structure

Based on the literature, there is a clear correlation between the outcomes of the five specified tests and the level of deterioration owing to ASR in a structure, as detailed below. Firstly, the visual assessment of a RC structure subject to ASR deterioration differentiates between concrete with no signs of visual distress and concrete with severe signs of visual distress. Subsequently, when concrete is extracted for testing from the two distinct areas of distress, higher DRI values are obtained for concrete taken from areas showing severe signs of distress compared to areas with no distress. With respect to the alkali content (Na and K) present in the concrete, concrete exhibiting distress shows lower alkali content compared to concrete which shows no signs of distress due to the consumption of alkalis in facilitating ASR. In the damage stiffness test, concrete exhibiting distress demonstrates a lower elastic modulus and higher stiffness damage index and plastic deformation index values. Finally, concrete displaying significant distress presented lower residual potential expansion results compared to concrete, which showed no signs of ASR distress.

However, when assessing the extent of ASR deterioration of the 78-year-old RC structure in this study, Figure 4-78 illustrated which assessment results/trends did not agree (shaded in yellow and red) with that noted in the literature.

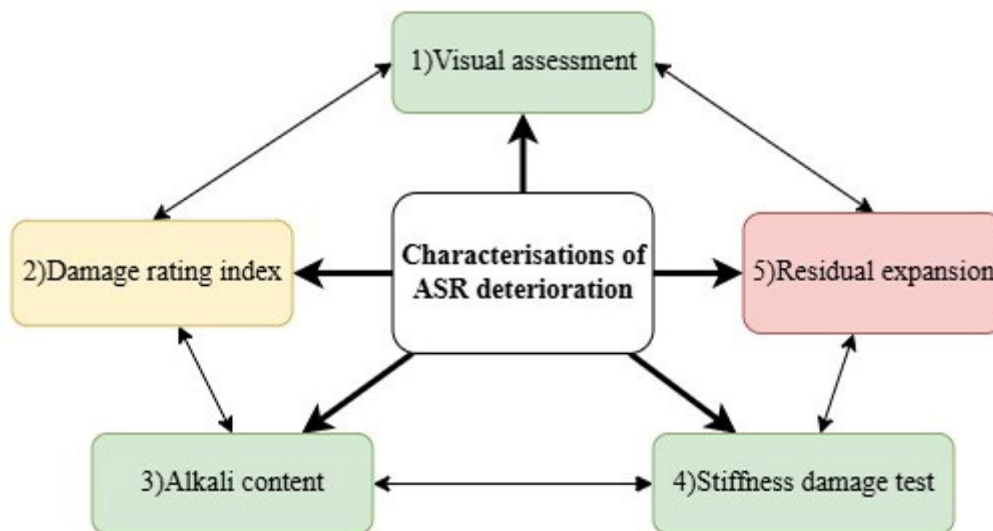


Figure 4-78: Results of five assessments conducted to characterise the extent of ASR deterioration in a 78-year-old RC structure

The DRI results showed a shared form of internal distress owing to ASR in concrete specimens from different locations, regardless of the level of damage observed. The modified approach used an increased magnification and grid size, which demonstrated that it was not sensitive enough to detect all the petrographic features in the concrete. As a result, it was ineffective in accurately characterising the extent of ASR deterioration in various areas of the RC railway bridge.

It was noted that the residual potential results, both on the macro and micro levels, did not align with the established trends in the literature. These findings emphasise the crucial role of moisture in facilitating future expansion and damage in the concrete due to ASR. It is essential to take preventive measures to stop moisture from entering the RC railway bridge and prevent potential future damage. Despite the highly reactive Witwatersrand aggregates, the deterioration of the RC railway bridge may have been suppressed due to Johannesburg's relatively low relative humidity conditions. However, the structure remains at risk of moisture ingress from external sources, such as inadequate drainage after heavy rain. Thus, the onus is on the engineer to specify that surface film coatings such as polyurethane coatings and water-repellent agents, e.g., water-based silicates, be applied to the exterior concrete of the entire RC railway bridge (Thomas *et al.*, 2013).

Chapter 5 Conclusion and recommendations

5.1 Introduction

This research aimed to quantify and interpret the expansion exhibited (with respect to the swelling pressure and subsequent strain development) by concrete subject to ASR deterioration on two scales (micro-scale and macro-scale). The study intended to (i) identify a field structure subject to ASR deterioration, (ii) determine the free alkali content in the concrete, (iii) determine the concrete stiffness properties owing to ASR deterioration, (iv) understanding of local and bulk mechanics of induced strain as a result of ASR gel formation and subsequent swelling (v) determine if there is a correlation between the localised micro-strain and the expansion measured on the bulk of the concrete (macro-strain)

To achieve the aims and objectives of the study, the experimental method was designed in three sequential phases as follows:

- Phase 1- Site identification and assessment
- Phase 2- ASR confirmation and concrete characterisation
- Phase 3- Assessment of residual potential expansion

The experimental details were presented in Chapter 3, while the results were presented, analysed and discussed in Chapter 4. This chapter deals with the general conclusions, practical implications, and recommendations for further study.

5.2 Phase 1

An RC railway bridge was identified as a field structure based on its location in Southern Johannesburg and extensive map cracking exhibited on the concrete surface. A visual inspection was done on thirteen railway bridge elements (piers, abutments, arches) to diagnose the structure's potential for ASR deterioration. The structure, overall, was characterised as having a medium to high probability of being subject to ASR deterioration. This was based on several visual signs of concrete deterioration owing to ASR deterioration. Twelve concrete cores were extracted from various locations of the Railway Bridge, visually showing varying degrees of ASR deterioration: undamaged, moderately and severely damaged. The modified DRI method was used to assess and compare the extent of damage to concrete taken from various locations of an RC railway bridge, using 2D global images of the cylindrical surface

of the concrete core. The DRI results indicate a shared form of internal distress across concrete specimens from various locations, irrespective of the observed damage levels.

5.3 Phase 2

Petrographic analyses of the concrete confirmed various diagnostic features of ASR, such as reactive aggregates, limited ASR gel presence, various crack types, and intricate crack networks. Micro X-ray fluorescence spectrometry results also affirmed that the primary cause of the deterioration of the RC railway bridge was an alkali-silica reaction. The technique has shown that it is possible to locate and characterise ASR gel, which is impossible with the naked eye, based on the elemental distribution (silica, potassium, and calcium) in the concrete matrix.

Free alkali metal concentrations were determined in the laboratory to investigate and quantify the reserve of alkali hydroxides available in concrete for further ASR development and expansion. The cold water extraction method was used to determine the free alkali metal content inventory with respect to Na^+ , K^+ and Ca^{2+} for concrete extracted from various damaged regions. These results were used to determine the equivalent sodium oxide content for concrete, which showed that it is possible to assess and compare the extent of ASR deterioration damage of concrete based on the consumption of various alkali metals. Concrete from the interior undamaged region exhibited the highest corrected sodium oxide equivalent content of $3.2 \text{ kg/m}^3 \text{ Na}_2\text{O}_{\text{eq}}$ compared to concrete from the interior damaged region with a corrected sodium oxide equivalent content of $0.1 \text{ kg/m}^3 \text{ Na}_2\text{O}_{\text{eq}}$.

In the absence of initial reference alkali data from the cement and concrete mix design when the bridge was initially constructed, the theoretical pore solution composition was determined on the values from the corrected sodium oxide equivalent content undamaged region. The theoretical pore solution for the concrete amounts to 3.16 kg/m^3 , based on sodium oxide equivalent content and taking into account the alkali release from aggregates. Compared to the stipulated alkali limits for controlling expansion, this value falls above the 2.0 kg/m^3 limit for Witwatersrand aggregates, validating the concrete's susceptibility to ASR deterioration.

The stiffness damage test showed that it is possible to assess the stiffness and compare the extent of damage of concrete taken from various locations of a RC railway bridge using various test outputs: modulus of elasticity (E-Mod), hysteresis area (HA), stiffness damage index (SDI), plastic damage index (PDI), and non-linearity index (NLI). The modulus of elasticity was determined to be 18.0 GPa, 14.0 GPa and 12.4 GPa for concrete tested from undamaged,

moderately and severely damaged regions, respectively. The decreased stiffness was noted in conjunction with an increase in hysteresis area and plastic deformation, with the increased extent of ASR damage. Additionally, the non-linearity index of less than one indicated that the cracks in the concrete were predominantly parallel to the direction of loading instead of perpendicular, as previously seen in 2D global images of the concrete core cylindrical surface.

5.4 Phase 3

Lastly, in the third phase of the study, residual expansion tests were carried out in the laboratory to investigate and quantify micro and macro potential residual strain in concrete subject to ASR deterioration. It can be seen that the maximum longitudinal strain recorded for the bulk expansion over the test duration ranged from 830 $\mu\epsilon$ to 700 $\mu\epsilon$ over 112 day test period. The analysis shows a correlation between higher potential residual strain values and more extensive damage in areas like Pier 1 (North and South). Conversely, lower potential residual strain values in areas of minimal damage, as observed in Abutment 1's inner and Pier 1's. These results did not follow trends in literature, where higher expansion levels are associated with concrete taken from undamaged areas.

With respect to local strain measurements, the attempt to quantify local strain development over a 2 cm by 2 cm square region of concrete affected by ASR using digital image correlation technique was unsuccessful. This result was attributed to three main reasons. Firstly, the use of a 10 mm thick concrete disc, in conjunction with a square region of interest of 2 cm. Furthermore, the increasingly difficulty to return the specimen holder precisely to its original position for image capture. The traditional strain measurement approach showed that the normal strain ranged from 830 $\mu\epsilon$ to 145 $\mu\epsilon$, whilst the shear strain ranged from 510 $\mu\epsilon$ to 50 $\mu\epsilon$ over 112 day test period. Surprisingly, the analysis established a correlation between higher potential residual strain and more extensive damage in areas like Pier 1 (North and South). Conversely, lower potential residual strains were generated in areas with minimal damage, as observed in the inner walls of Abutment 1 and Pier 1. These results do not follow trends in literature, where higher expansion levels are associated with concrete taken from undamaged areas. However, the same trend was observed in the bulk residual potential expansion results. Furthermore, it was noted that despite lower alkali reserves of sodium and potassium in the concrete from the damaged region (Section 4.11), the concrete still exhibited the highest residual potential expansion. This may be attributed to the increased permeability of the

damaged concrete, which facilitated moisture ingress and caused swelling of existing ASR gel and the formation of new ASR gel in the concrete.

Lastly, a non-linear positive correlation was established between local normal equivalent strain and bulk strain, similarly a non-linear positive correlation between local shear equivalent strain and bulk strain. The results of this phase showed that the potential residual expansion of concrete from the RC railway bridge, subject to varying degrees of ASR deterioration, displayed similar magnitudes of bulk potential residual and micro potential expansion despite varying degrees of surface damage and the limited availability of alkalis in certain concretes.

5.5 Recommendations for further research

Based on the findings of this study, the following recommendations for future research are given.

- This study was limited to the findings of twelve concrete cores taken from only Piers and abutment wall elements, showing no distress (damage), moderate damage and severe damage. Due to the limited specimens available for testing from each damage area, it was impossible to quantify the variability in the results obtained for each test conducted for a given area. Thus, it is recommended that additional areas be identified on the RC railway bridge structure (at least two additional regions per damage severity), and additional concrete cores be extracted, and the respective tests be conducted to provide a more comprehensive evaluation of the RC railway bridge.
- Furthermore, as noted in the previous recommendation, concrete cores were only extracted from the RC railway bridge structure's pier and abutment wall elements. Concrete extracted from these locations is primarily subjected to compressive stresses, which are known to restrain the concrete, subsequently suppressing ASR expansion and deterioration. It is recommended that concrete cores be extracted from elements such as the deck, which is subject to less restraint but displayed signs of severe map cracking. In addition, further research is needed on the influence of restraint conditions in RC structures and its subsequent impact on the extent of ASR deterioration, particularly with regard to long-term effects.
- In light of the role of moisture in facilitating ASR, it is recommended that a visual assessment of the RC railway structure be conducted after a heavy downpour. This will

highlight areas subject to water ponding and the ingress of external sources of moisture. This may explain why areas such as the underside of the bridge arches are subject to map cracking. Following the previous recommendation, the moisture content at various depths of concrete cores from various locations should be determined. This can provide useful information on the internal moisture availability in the concrete, which may assist in understanding if moisture is possibly trapped in the bulk of the concrete. This may provide a possible explanation for the distress seen in the bulk of the concrete cores.

- It is established that the reactivity of siliceous aggregates depends not only on the mineralogy but also on the mechanics of the formation of the aggregate and the degree of deformation of the quartz. It is recommended to determine if there is any potential relationship between the degree of straining of Quarts grain particles in the Witwatersrand quartzite and the reactivity of aggregate susceptible to ASR deterioration.
- Owing to the larger scan area of the Micro X-ray fluorescence spectrometer, it is proposed that the instrument be used to create elemental maps to analyse the presence of alkalis along the depth of the concrete core to help validate results obtained from the cold water extraction method.
- In this study, local strain measurements over a 20 mm square region of concrete affected by ASR using the digital image correlation technique were unsuccessful. Thus, using a larger area of interest, 100 mm by 100 mm, is advisable when using DIC to obtain deformation and potential residual strain readings. Furthermore, careful consideration must be taken with respect to the camera resolution required for DIC setup and the strain magnitude under investigation. In this particular instance, a higher resolution camera may be better suited to capture micro-level strain measurements.
- In addition, it is proposed that the DIC rig set-up may have to be re-designed to accommodate multiple specimens but limit the specimen's movement, thus ensuring successful image tracking over a specified time period.
- Durability index tests, in particular oxygen permeability tests, are recommended to be conducted on concrete specimens taken from the RC railway bridge. This knowledge

may validate the hypothesis that areas with a greater extent of damage resulted in the highest degree of expansion despite the limited availability of alkalis.

- It is recommended that the RC railway bridge be modelled using FEM to determine the likely consequences of design elastic modules versus the reduced concrete stiffness. This provides insight into potential effects on the stress development in the structure, particularly the top portion of the arches of the railway bridge. Furthermore, it would be valuable to quantify the dynamic effects of a train moving over the RC railway subject to ASR deterioration.
- Lastly, owing to the lack of information (either lost or destroyed) on the RC railway bridge, it is recommended to assess several other RC structures constructed using Witwatersrand aggregates subject to ASR deterioration in the greater Johannesburg region. With respect to visual signs of deterioration, characteristics of concrete properties and likely potential future expansion. This will aid the creation of a database of knowledge of structures affected by ASR deterioration, which will assist Agencies with the management of such structures going forward.

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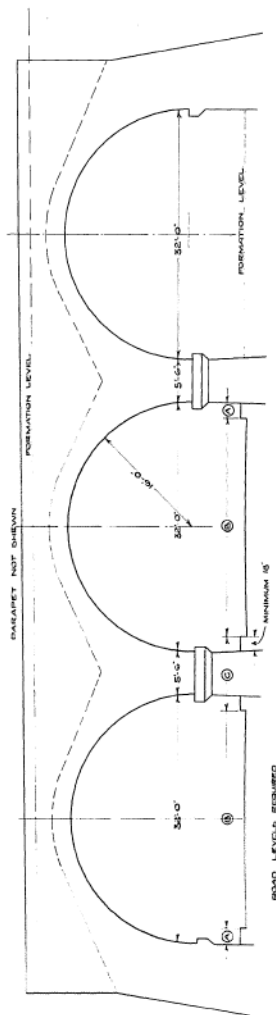
LONGITUDINAL SECTION ALONG PROPOSED ARCH DAM SHAFT
SCALE: 1" = 20'

PLAN VIEW
SCALE: 1" = 100'

PROPOSED ARCH DAM
TRUCK ROAD - RAMP MINERAL LINE

JOHANNESBURG, N.W. WORK
SITE PLAN FOR 3150' ARCHES ON DEVIATION N° 1:
TRUCK ROAD - RAMP MINERAL LINE

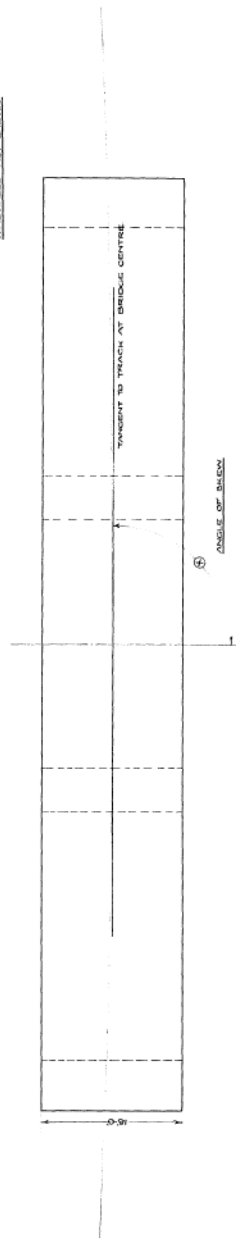
PLAN NUMBER
JNW 326 11.5'



ROAD LEVELS REQUIRED ALONG CENTRE LINES OF PIERS AND ABUTMENTS

PART ELEVATION

MINIMUMS NOT SHOWN



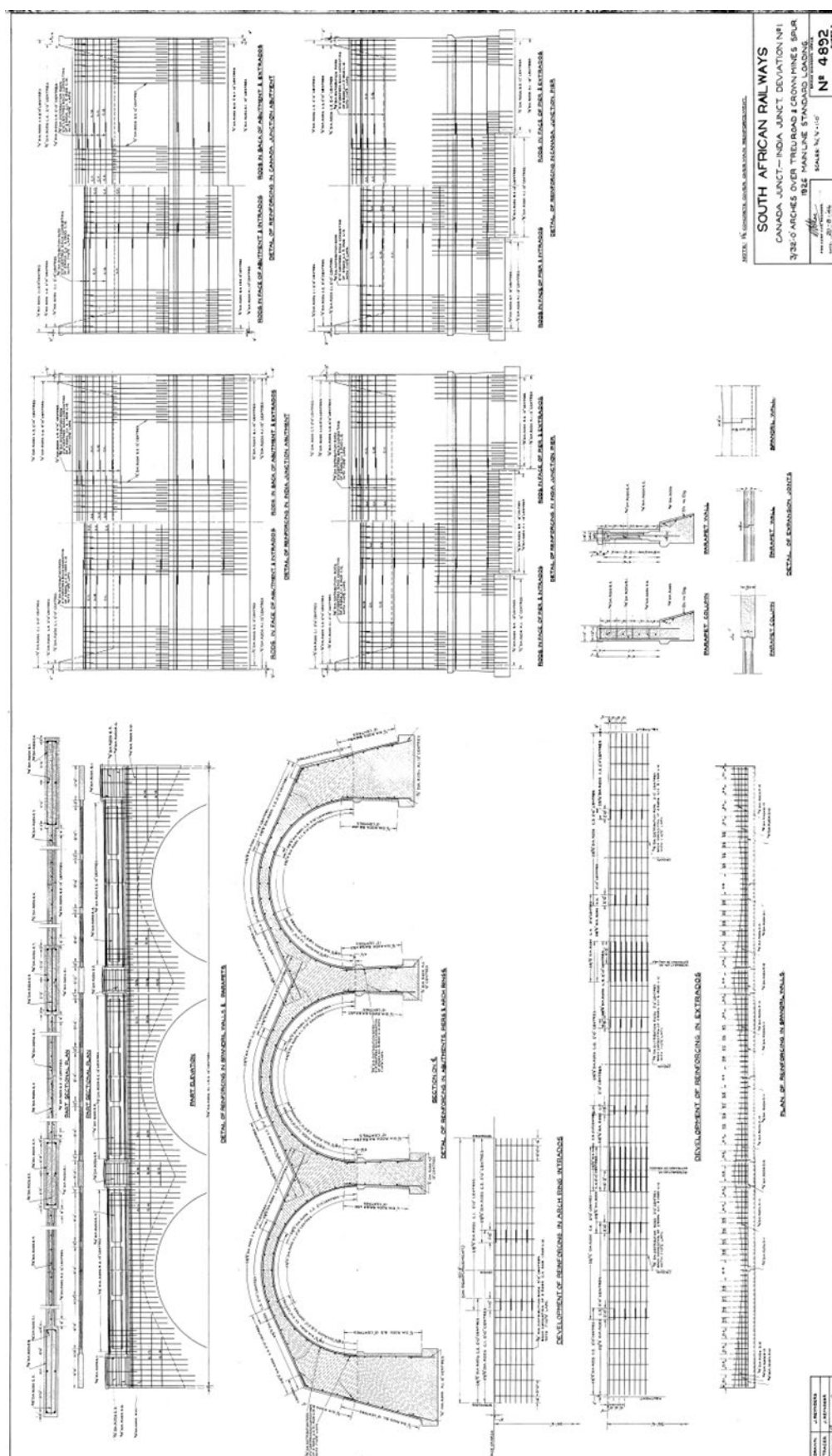
PART PLAN

DIMENSIONS A, B, & C ALSO ANGLE X REQUIRED

SOUTH AFRICAN RAILWAYS.
CANADA JUNCT - INDIA JUNCT DEVIATION No 1.

5/32' O' ARCHES AT TREE ROAD
CHIEF ENGINEER
2/5/46
B.E. No 4892 P

DRAWN BY
CHECKED BY
APPROVED BY



Appendix B - Visual assessment report of RC railway bridge located on True Road, in the Crown Mines Spur

Details of the structure

GPS Coordinate location

Latitude			Longitude		
DD	MM	SS.s	DD	MM	SS.s
26	13	13.4	28	01	31.1

Town: Johannesburg

Province: Gauteng

Type of structure: Reinforced concrete arch railway bridge

Owner of asset : Passenger Rail Agency of South Africa (PRASA)

Date of construction : 1946 Age: $\pm$ 78 years old

Nature of exposure and loading condition

Description of exposure environment : Inland

Ambient temperature (Avg.): 10 °C in June to 21 °C in January

Relative humidity (Avg.): 65% RH in Summer months, 30% RH in winter months

Rainfall (Avg.):

700 millimetres per year and is concentrated in the warm period, from October to March

Soil (foundation conditions):

Loading conditions: Structure subject to dynamic loading from passing railway carriages

Existing records

Existing drawing:

BE-4892_SHT_A – Site plan for 3/32' Arches on Deviation No1

BE-4892_SHT_P – Canda Junction – India Junction Deviation No1

BE-4892_SHT_1 – Canda Junction – India Junction Deviation No1, (Construction drawing)

BE-4892_SHT_2 - Canda Junction – India Junction Deviation No1, (Construction drawing)

Existing reports (e.g. maintenance/repair/etc. reports): None

Additional notes

- Owing to the vandalism and theft of overhead cables, the railway corridor was not in operation thus the bridge was not subjected to any railway traffic/loading.

General overview of structure



Figure 1: North-facing view of Railway bridge located on True Road and in the Crown Mines Spur



Figure 2: South-facing view of RC railway bridge located on True Road and in the Crown Mines Spur

Additional comments:

- Steel pipe gantry structure running alongside the Southwards facing side of the railway bridge.

Reference schematic used to identify various railway bridge element

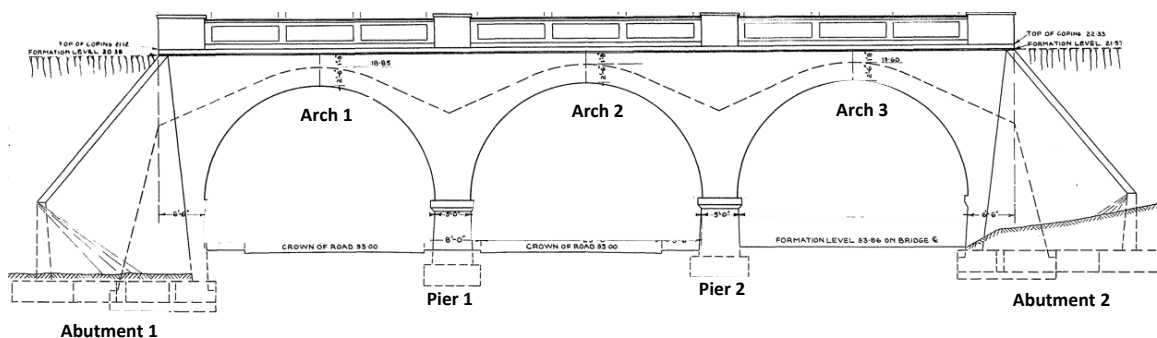


Figure 3: Elevation view looking southwards onto the RC railway bridge located on True Road and Crown Mines Spur in Johannesburg

Element: Abutment 1



Figure 4: Abutment 1 – Northwards facing side



Figure 5: Abutment 1 – Southwards facing side

Exposure: Outdoor environment (rain and sun)

South-facing abutment partly sheltered due to adjacent pipe gantry and surrounding foliage

Drainage: Weep hole positioned approximately one-third from the top of the abutment

Surface condition of concrete:

- Extensive map cracking on the upper surface, cracks visible through surface coating
- Rust straining below the drainage area
- Lower surface subject to graffiti, painted advertisements and posters

Element: Abutment 2



Figure 6: Abutment 2 – Northwards facing side



Figure 7: Abutment 2 – Southwards facing side

Exposure: Outdoor environment (rain and sun)

South-facing abutment partly sheltered due to adjacent pipe gantry and surrounding foliage

Drainage: Weep hole positioned approximately one-third from the top of the abutment

Surface condition of concrete:

- Rust straining below the drain area
- Cracks present around the drainage area
- Lower surface subject to graffiti, painted advertisements and posters

Element: Abutment 1 – Interior Wall



Figure 10: Abutment 1 – Interior wall

Exposure: Sheltered, outer edges of wall exposed to rain

Drainage: None

Surface condition of concrete:

- Extensive map cracking, primarily on the surface above the abutment construction joint
- Random areas of dampness and rust straining are present along the abutment lip
- Lower surface subject to graffiti, painted advertisements and posters

Element: Pier 1 & 2 – Outer Walls



Figure 4 (a, b):

North and South facing view of Pier 1 outer walls



Figure 5 (a, b):

North and South facing view of Pier 2 outer walls

Exposure: Outdoor environment (rain and sun)

Drainage: Weep hole positioned at the top section of pier element

Surface condition of concrete:

- Extensive map cracking on pier, haunches and spandrel wall surfaces, cracks visible through surface coating

- Dampness strains originating from cracks and conjunction joints along arch, haunches and spandrel wall
- Whitish gel exudations originating from cracks
- Lower surface subject to graffiti, painted advertisements and posters

Element: Pier 1 – Interior Wall



Figure 11: Pier 1 – Interior wall, facing eastwards



Figure 12: Pier 1 – Interior wall, facing westwards

Exposure: Sheltered, outer edges of wall exposed to rain

Drainage: None

Surface condition of concrete:

- Extensive map cracking on the surface, primarily above the pier construction joint
- Random areas of dampness and rust straining present along the interior wall
- Continuous line of dampness running along the construction joint between the haunch and arch section
- Lower surface subject to graffiti, painted advertisements and posters

Element: Pier 2– Interior Wall (East)



Figure 13: Pier 2 – Interior wall, facing eastwards



Figure 14: Pier 2 – Interior wall, facing westwards

Exposure: Sheltered, outer edges of wall exposed to rain

Drainage: None

Surface condition of concrete:

- Extensive map cracking on the surface, primarily above the pier construction joint
- Random areas of dampness and rust straining were observed running longitudinally between various concrete pier and haunch construction joints
- Continuous line of dampness running laterally along the construction joint between the haunch and arch section
- Lower surface subject to graffiti, painted advertisements and poster

Element: Abutment 2 – Interior Wall



Figure 15: Abutment 2 – Interior wall

Exposure: Sheltered, outer face exposed to rain

Drainage: None

Surface condition of concrete:

- Extensive map cracking, primarily on the surface above the abutment construction joint
- Random areas of dampness and rust staining originating from cracks
- Lower surface subject to graffiti, painted advertisements and poster

Element: Arch 1, 2 and 3



Figure 16: Arch 1 (View from North side looking Southwards)



Figure 17: Arch 2 (View from the North side looking Southwards)



Figure 18: Arch 3 (View from the North side looking Southwards)

Exposure: Sheltered, extreme ends exposed to rain

Drainage: None

Surface condition of concrete:

- Extensive map cracking on arch surface
- Random areas of dampness and straining originating from cracks
- Whitish gel exudations originating from cracks

Appendix C - “As-received” core log

Core log: Cores (12 of) extracted from RC railway bridge located on True Road and in the Crown Mines Spur, in Johannesburg

Core Label	Reference position on Railway bridge	Diameter (mm) ^a			Total length (mm) ^a	Weight (g)	If broken, location (distance from top, mm) ^a	Reinforcement				Aggregate				Other comments ^f
		d ₁	d ₂	Avg.				Rebar type ^b	Diameter ^c (mm) ^c	Location (distance from top, mm) ^a	No. of bars	Type	Max. size (mm)	Shape ^d	Distribution ^e	
C1	Abutment 1 - Inner wall	93.7	94.1	93.9	304	5202	-	-	-	-	-	Quartzite, Slate	60	Angular	Well graded	- Greyish coating present on all concrete cores exterior surfaces; - cracks predominant in coarse aggregates; - Cracks appear to occur along entire length of concrete core;
C2	Abutment 1 - Inner wall	93.9	93.7	93.8	274	5247	-	-	-	-	-	"	"	"	"	
C3	Abutment 1 - Inner wall	93.8	94.0	93.9	276	5125	-	-	-	-	-	"	"	"	"	
C4	Abutment 1 - Inner wall	94.3	94.0	94.2	264	5052	-	-	-	-	-	"	"	"	"	
C5 (a, b)	Pier 1 - North-facing view	94.0	94.3	94.2	286	5047	183	-	-	-	-	"	"	"	"	
C6 (a, b)	Pier 1 - North-facing view	94.1	93.8	94.0	306	5316	222	-	-	-	-	"	"	"	"	
C7	Pier 1 - Inner wall West-facing side	93.8	94.2	94.0	254	4814	-	-	-	-	-	"	"	"	"	
C8	Pier 1 - Inner wall West-facing side	93.7	93.5	93.6	223	4504	-	-	-	-	-	"	"	"	"	
C9	Pier 1 - Inner wall West-facing side	94.3	94.1	94.2	184	3435	-	-	-	-	-	"	"	"	"	
C10	Pier 1 - Inner wall West-facing side	94.0	94.1	94.1	306	5578	-	-	-	-	-	"	"	"	"	
C11	Pier 1 - South-facing view	94.1	94.2	94.2	189	3378	-	-	-	-	-	"	"	"	"	
C12	Pier 1 - South-facing view	94.1	93.5	93.8	274	4829	-	-	-	-	-	"	"	"	"	

*** Additional Notes**

^a: Record to one decimal place

^b: State if deformed or ribbed

^c: Try and conform to standard rebar diameters e.g. 8, 10, 12, 16, 20, 25, 32, 40 mm

^d: Use conventional terms e.g. flaky, angular, rounded

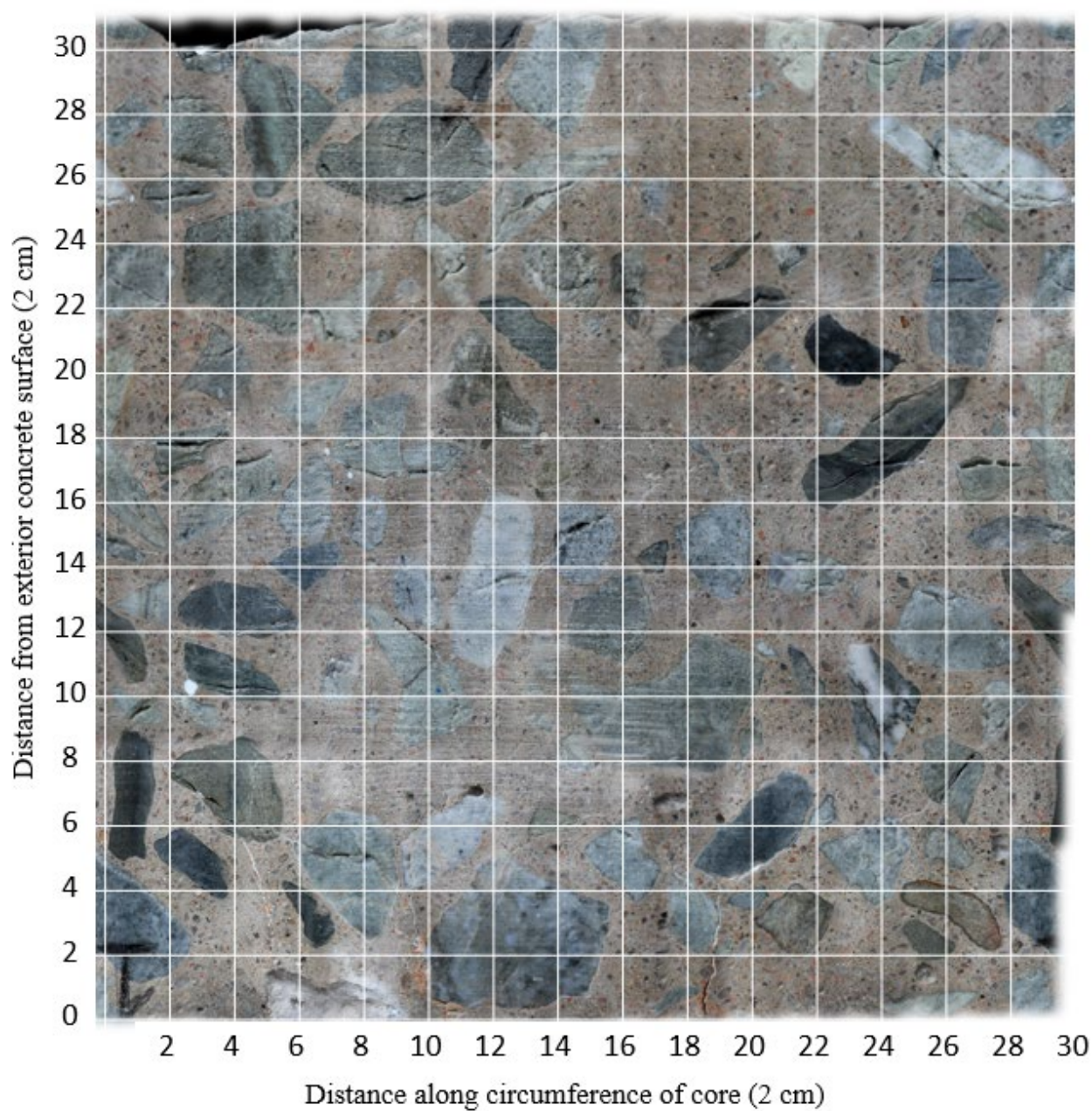
^e: Distribution w.r.t. (i) fine and coarse aggregate in the core, and (ii) ratio of fine to coarse aggregate

^f: This may include non-standard details like presence of a cold joint (note its location), colour of aggregate/concrete, friability (easily crumbles) of the concrete, etc

Appendix D - 2D global images of cores' cylindrical surface

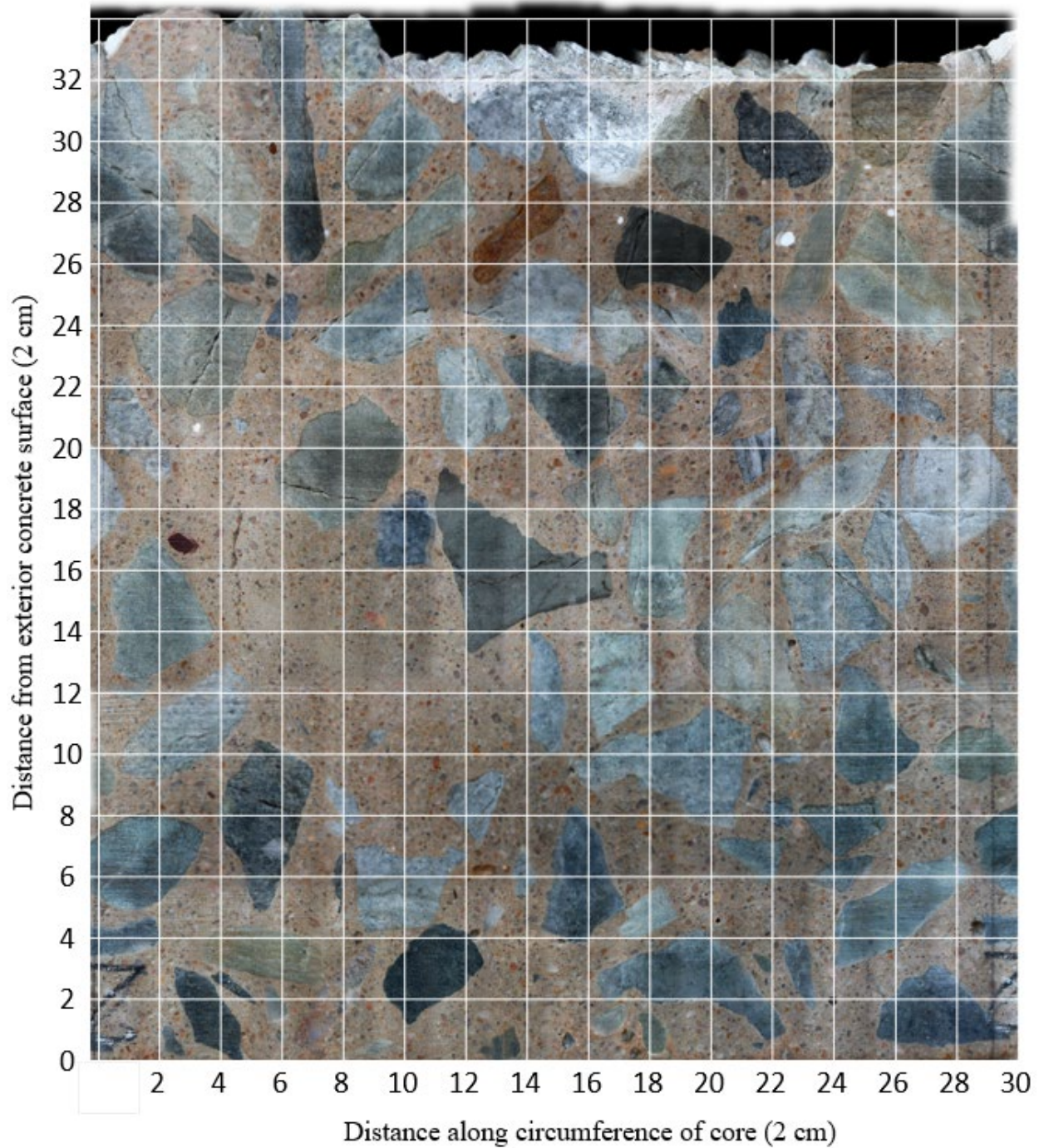
Core #1 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



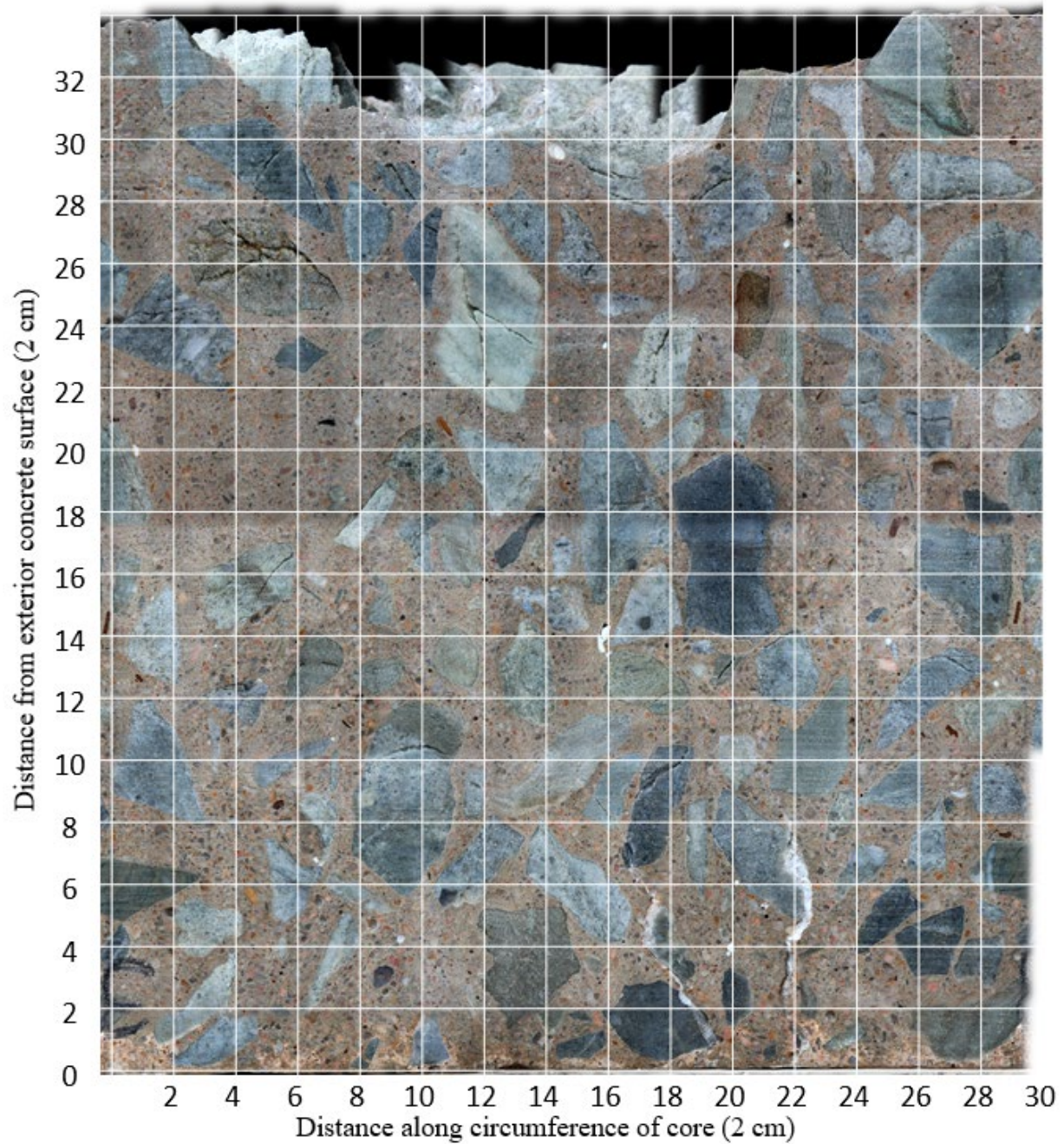
Core #2 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



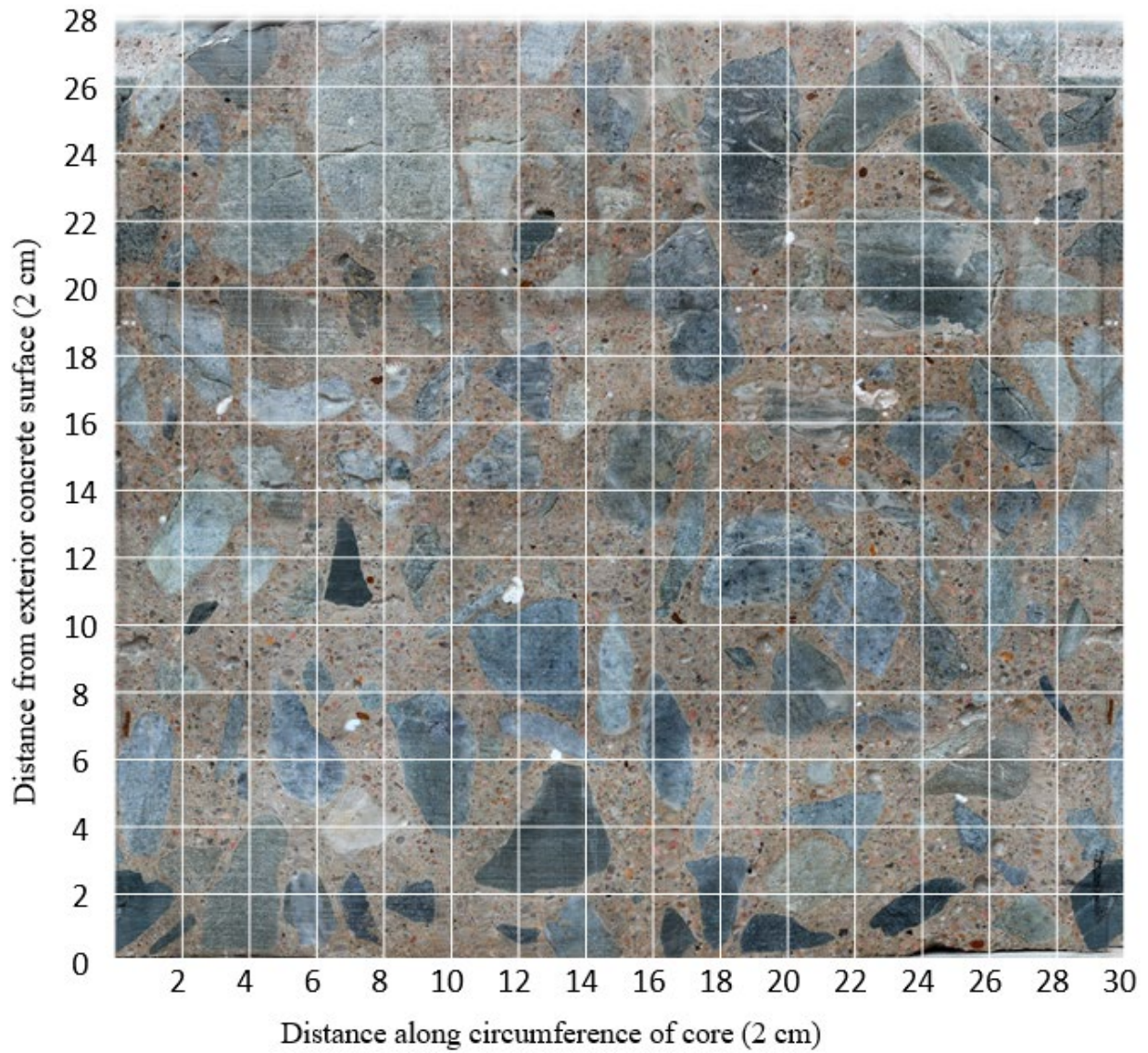
Core #3 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



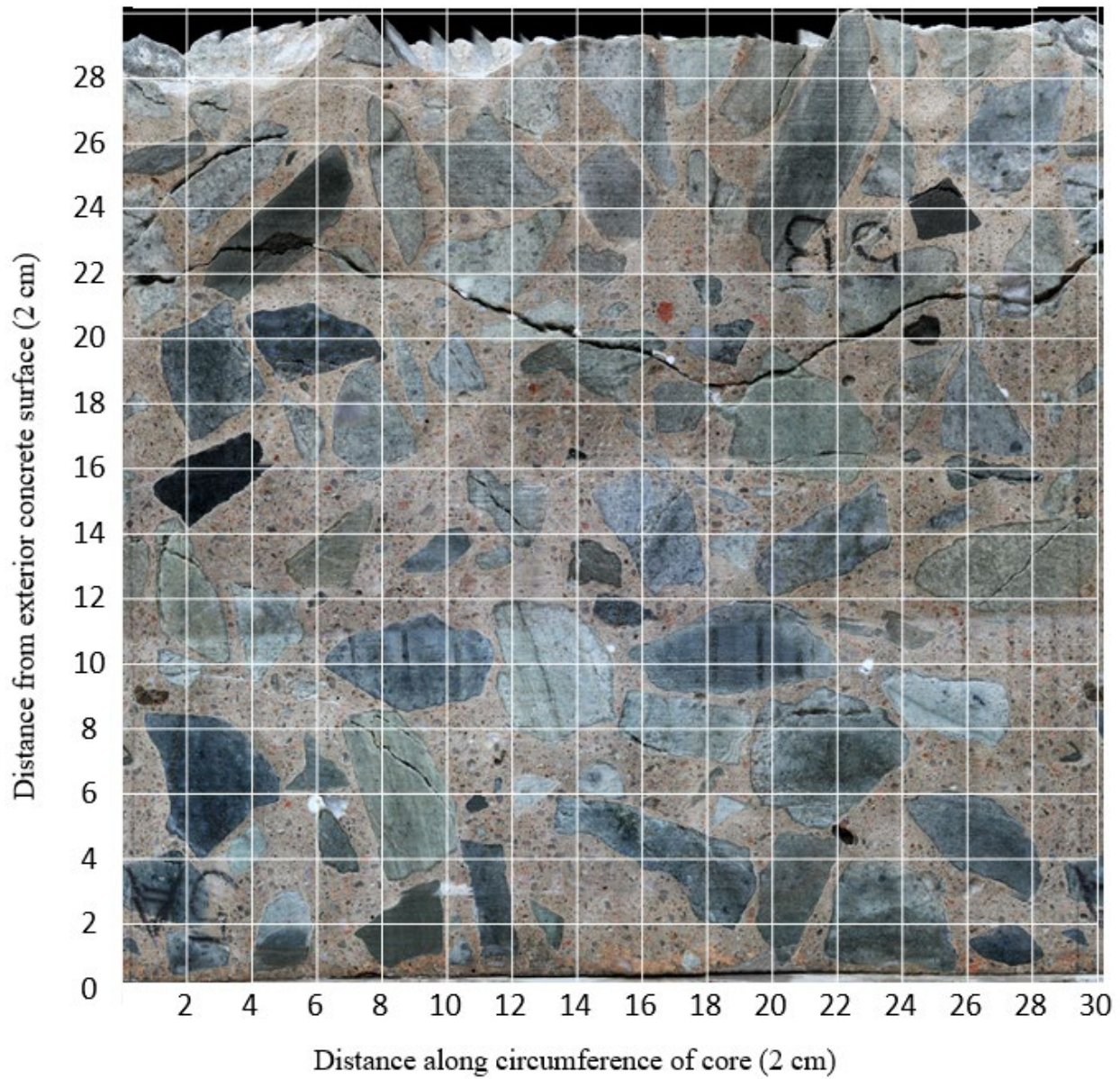
Core #4 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



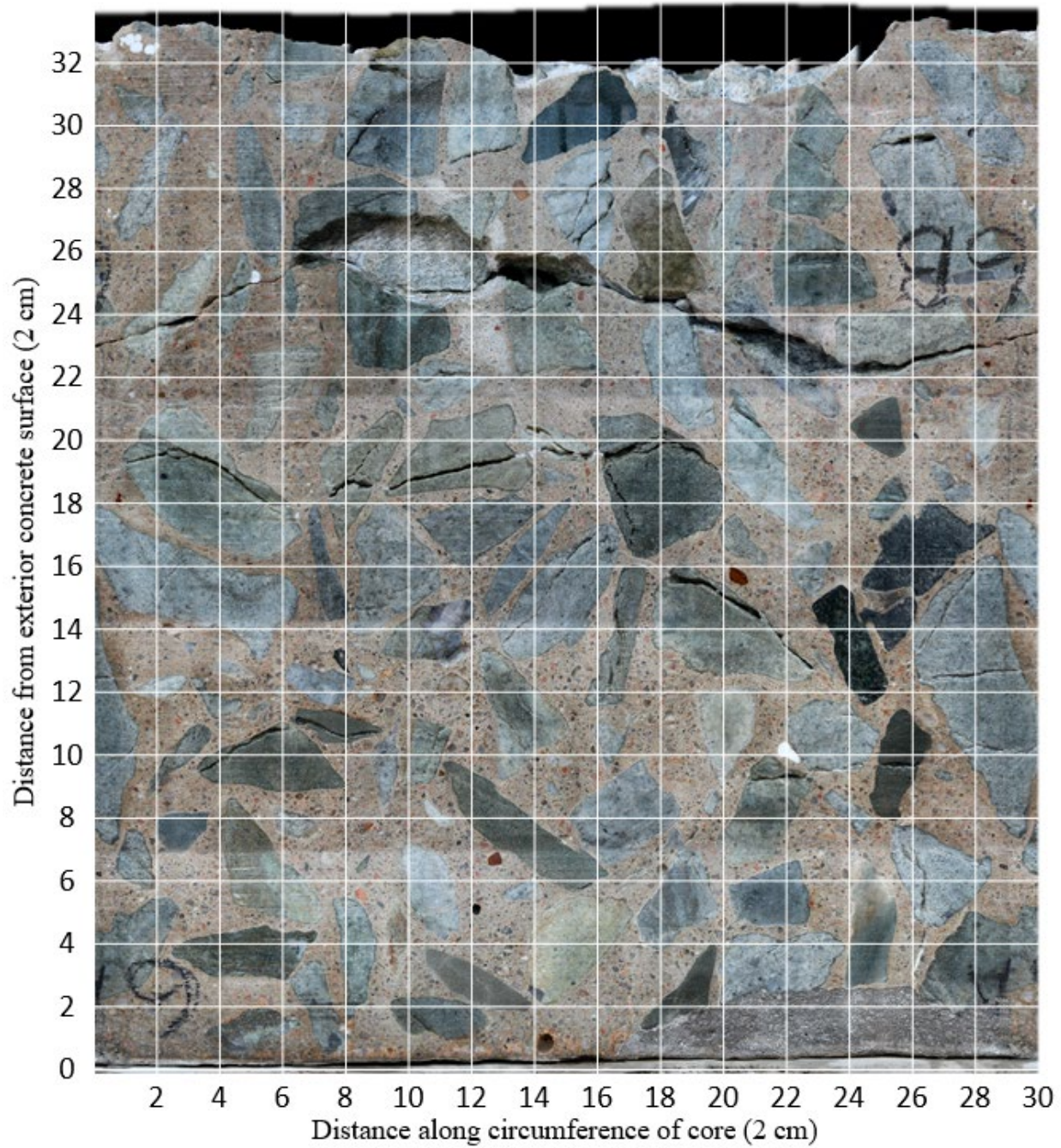
Core #5 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



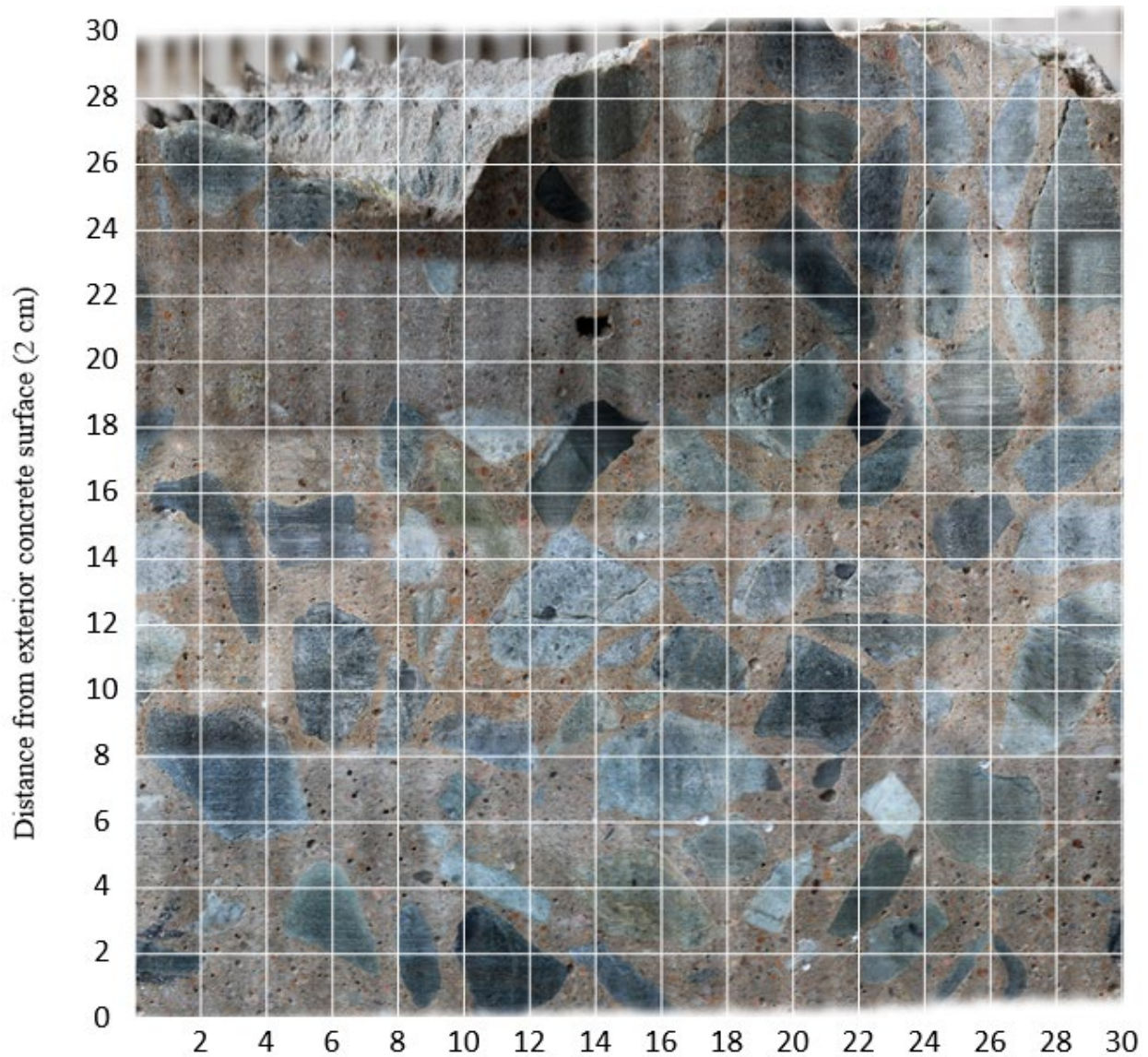
Core #6 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



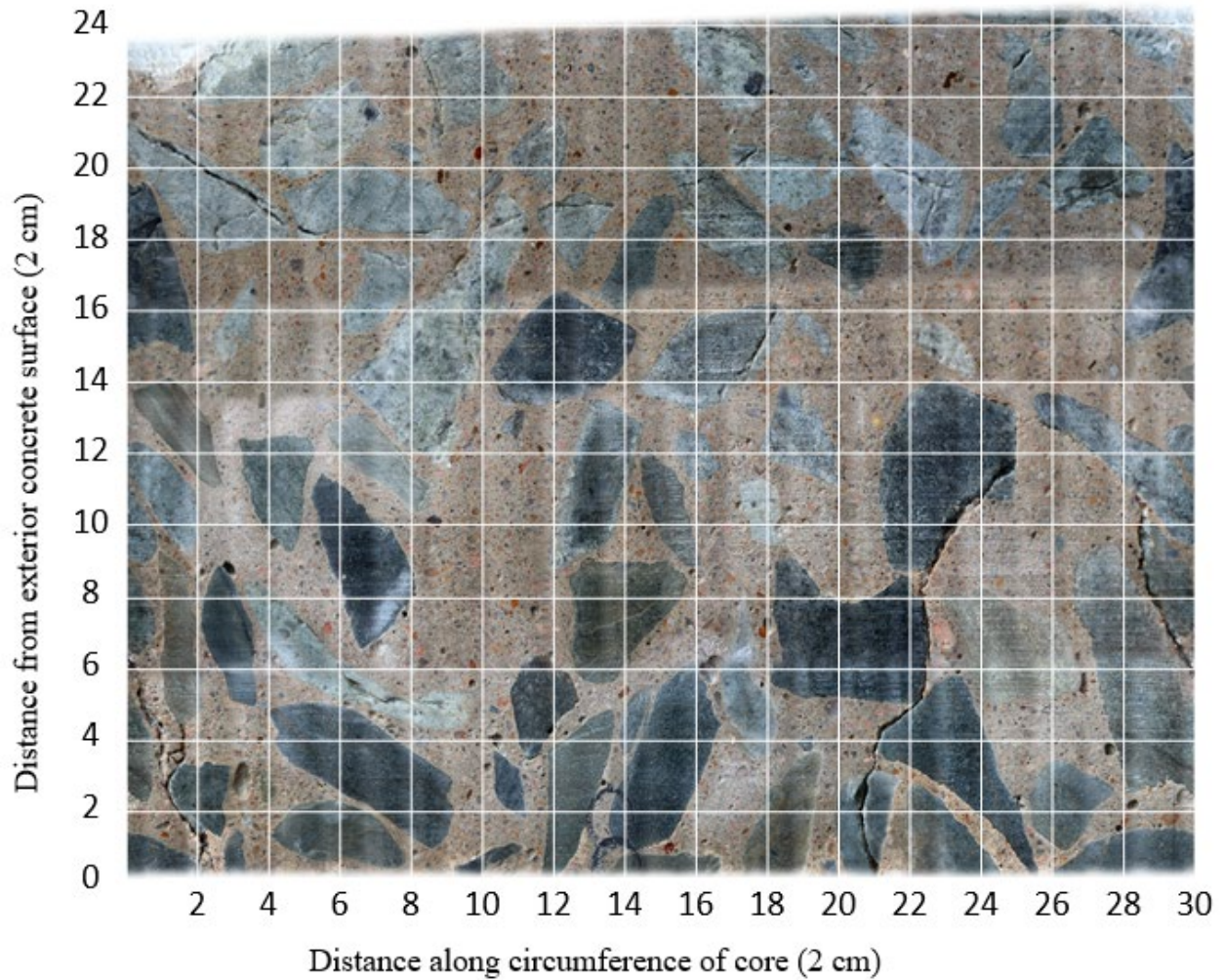
Core #7 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



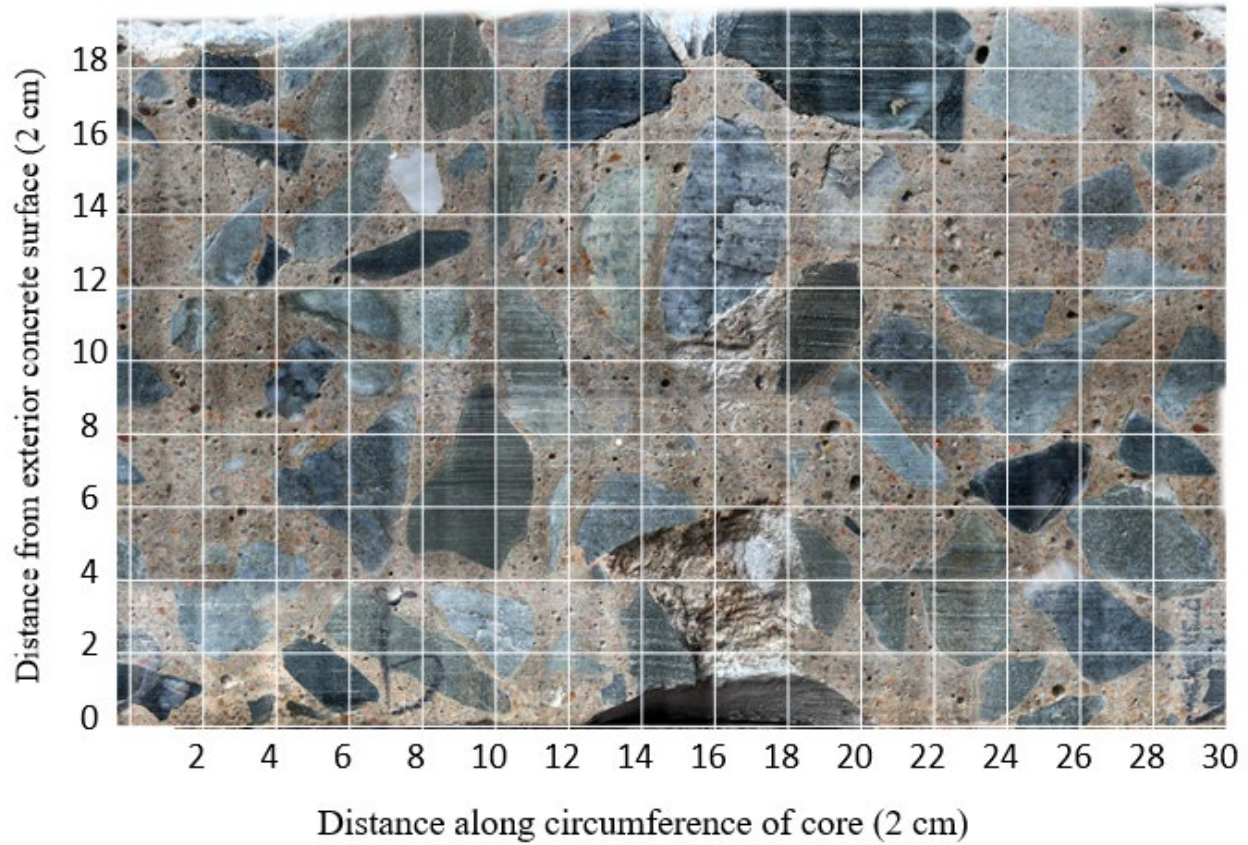
Core #8 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



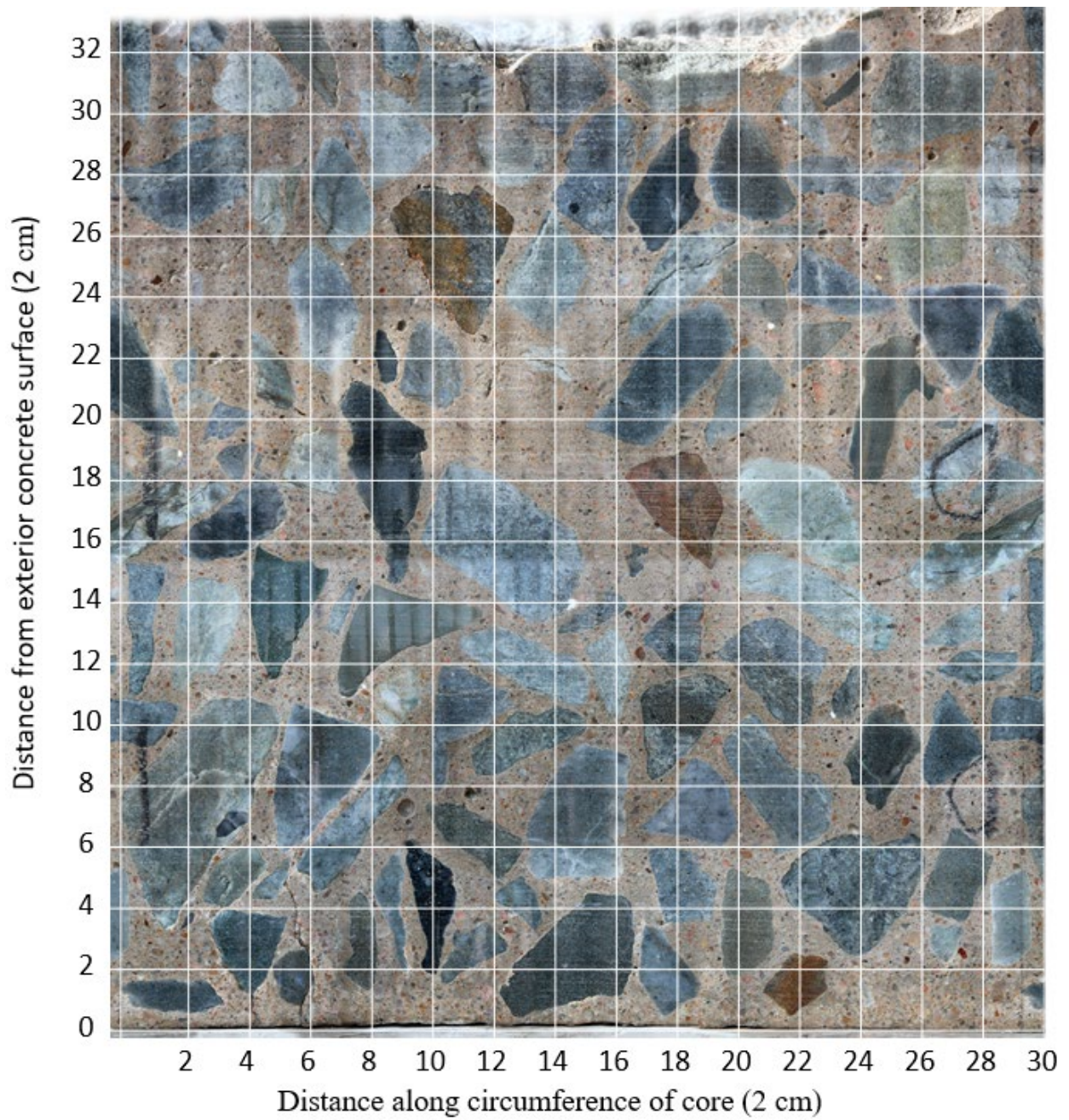
Core #9 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



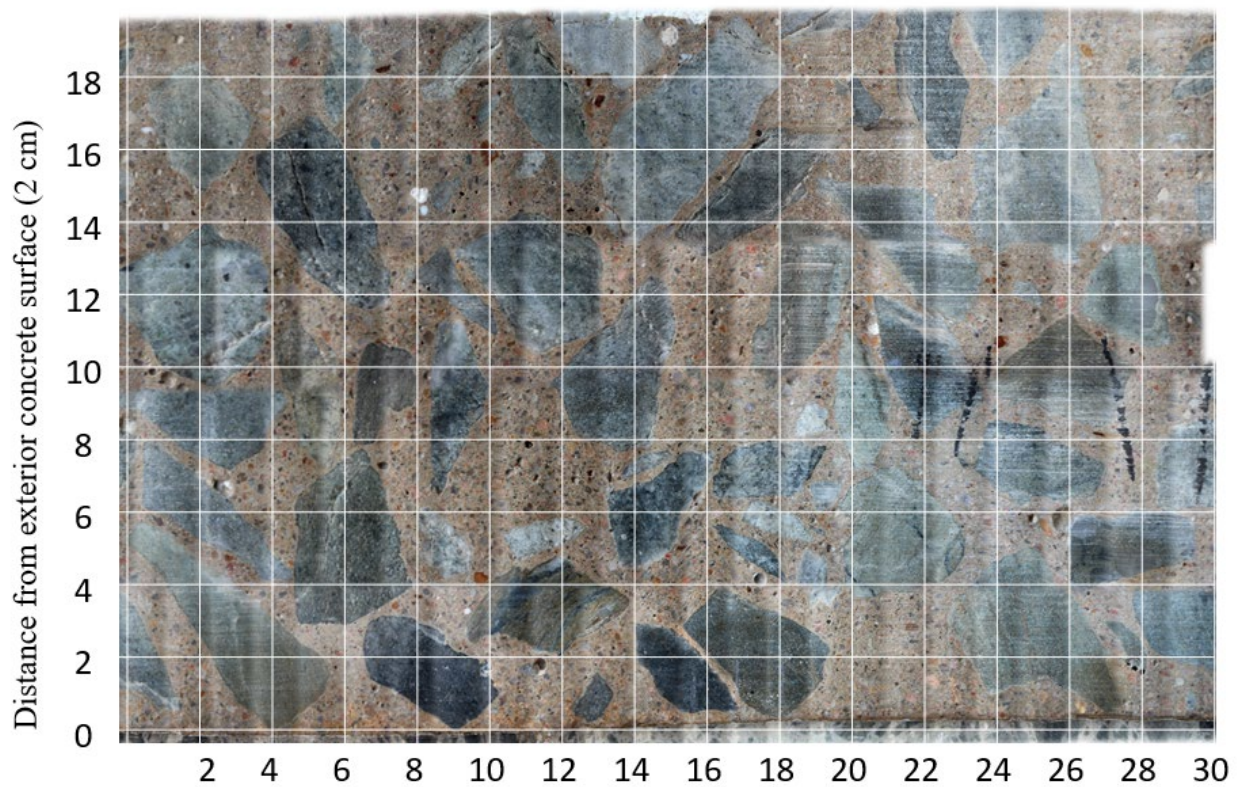
Core #10 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



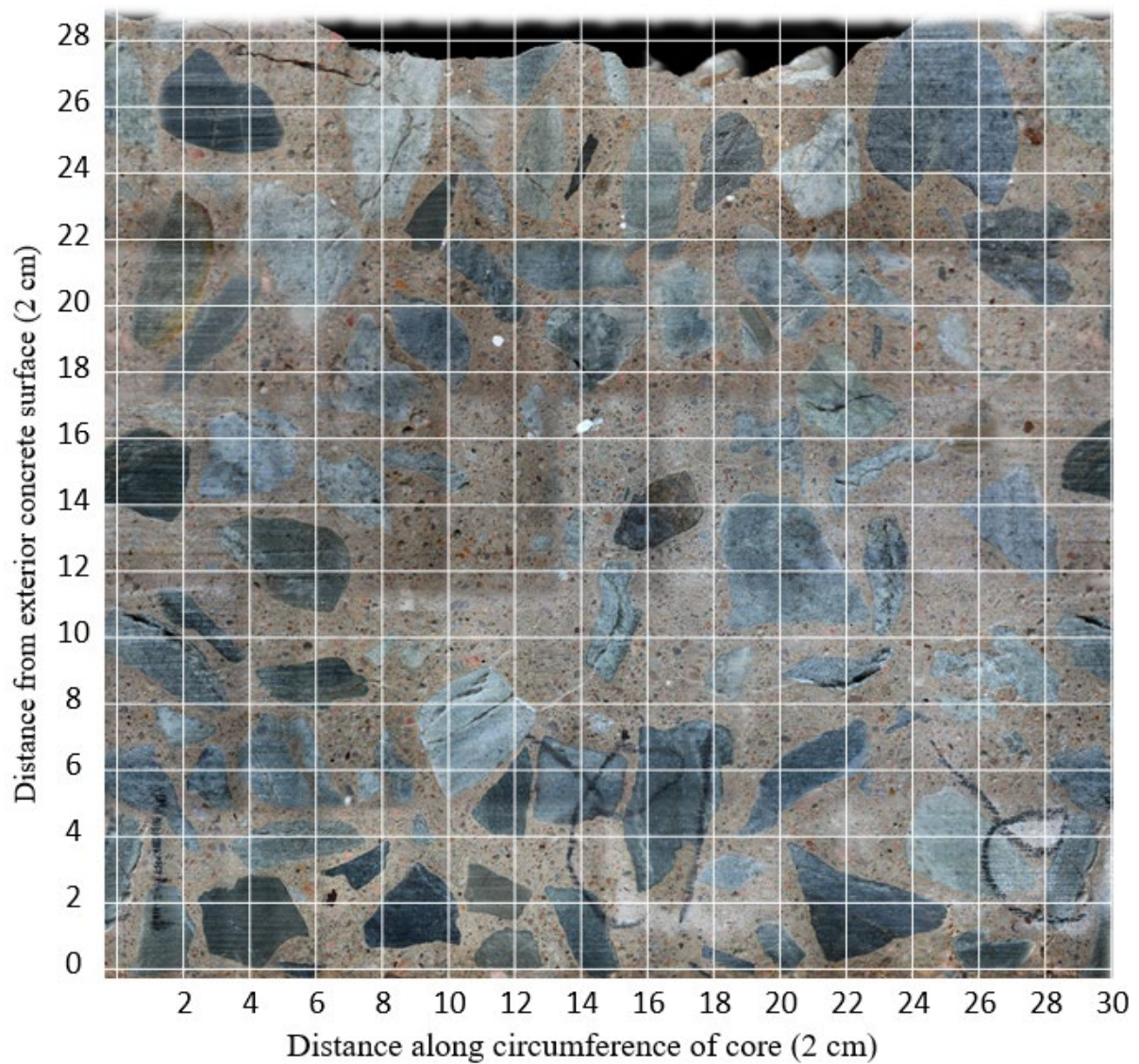
Core #11 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



Core #12 – Cylindrical surface map

* Note the image has been scaled down to fit on an A4 page, with an actual grid size of 2 cm²



Appendix E - Free alkali metal content

Free alkali content of concrete

Variables:	
Dilution Factor	10.14
Amount of pore solution in sample (ml)	1
Deionized water (ml)	20
Constant	0.001
Powdered material used (g)	20
Sodium Molar mass (g/mol)	22.99
Potassium Molar mass (g/mol)	39.1
Calcium molar mass(g/mol)	40.08
Density of concrete kg/m ₃	2400
M(Na ₂ O)	61.98
M(NaOH)	39.997
M(KOH)	56.1056
% of aggregates	70

Sodium (Na)										
	Specimen ID	Initial reading (mg/l)			AVG	Max	Min	AVG (g/l)	Na (mmol/kg)	Depth
		1	2	3						
Moderately “damaged”	1A	53	47	46	49	53	46	0.0487	21	15
	1E	87	92	82	87	92	82	0.0870	38	265
Severely “damaged”	6A	15	16	15	15	16	15	0.0153	7	15
	6E	10	11	7	9	11	7	0.0093	4	265
“Undamaged”	7A	31	30	31	31	31	30	0.0307	14	15
	7E	101	102	105	103	105	101	0.1027	45	265
Aggregate	A	9	9	9	9	9	9	0.0090	4	-
Potassium (K)										
	Specimen ID	Initial reading (mg/l)			AVG	Max	Min	AVG (g/l)	K (mmol/kg)	Depth
		1	2	3						
Moderately “damaged”	1A	38	38	38	38	38	38	0.0380	9.9	15
	1E	13	14	14	13	14	13	0.0135	3.5	265
Severely “damaged”	6A	20	21	21	21	15	15	0.0205	5.3	15
	6E	15	15	15	15	37	37	0.0148	3.8	265
“Undamaged”	7A	37	37	37	37	37	37	0.0372	9.7	15
	7E	20	20	21	20	21	20	0.0205	5.3	265
Aggregate	A	247	276	276	266	276	247	0.2663	69.1	-
Calcium (Ca)										
	Specimen ID	Initial reading (mg/l)			AVG	Max	Min	AVG (g/l)	Ca (mmol/kg)	Depth
		1	2	3						
Moderately “damaged”	1A	15.9	16.5	16.2	16.2	16.5	15.9	0.0162	4.1	15
	1E	6.9	7.1	7.1	7.0	7.1	6.9	0.0070	1.8	265
Severely “damaged”	6A	7.1	7.2	7.3	7.2	0.1	0.1	0.0072	1.8	15
	6E	0.1	0.1	0.1	0.1	2.7	2.5	0.0001	0.0	265
“Undamaged”	7A	2.5	2.7	2.7	2.6	2.7	2.5	0.0026	0.7	15
	7E	1.9	1.9	2.1	2.0	2.1	1.9	0.0020	0.5	265

Appendix F – Stiffness damage test results

The data presented in the table below is a summary of various indices obtained using the raw data of stiffness damage test.

Wet Cycle #1	Undamaged												
	Cycle	↑	↓	HA	SDI	SEC E1	SEC E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	2746	-1558	1188	0.433	14.7	18.0	14.5	21.7	0.0002	0.0008	0.2857	0.859
	2	2169	-1639	530	0.244	16.2	18.0	19.3	16.9				
	3	2253	-1900	353	0.157	16.2	17.0	19.3	16.9				
	4	2199	-1894	305	0.139	16.2	17.0	19.3	16.9				
	5	2240	-1981	259	0.116	16.2	16.2	16.9	16.9				
				2635	1.088	16.2	17.0	18.7	16.9				
	Moderately												
	Cycle	↑	↓	HA	SDI	SEC E1	SEC E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	3021	-2338	683	0.226	12.9	14.5	14.5	14.5	0.0001	0.0008	0.1429	0.847
	2	2681	-2382	299	0.111	13.8	14.0	12.1	12.1				
	3	2708	-2478	230	0.085	13.5	14.0	14.5	12.1				
	4	2750	-2609	140	0.051	13.5	13.5	12.1	14.5				
	5	2645	-2486	159	0.060	13.5	13.5	14.5	12.1				
				1510	0.533	13.6	13.8	13.3	12.7				
	Severely												
	Cycle	↑	↓	HA	SDI	SEC E1	SEC E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	3639	-2238	1401	0.385	10.1	13.5	9.7	12.1	0.0004	0.0011	0.3077	0.789
	2	3113	-2622	491	0.158	11.5	12.4	12.1	14.5				
	3	2833	-2497	336	0.119	12.0	12.4	12.1	14.5				
	4	2804	-2478	326	0.116	12.0	12.4	12.1	14.5				
	5	2791	-2503	288	0.103	12.0	12.0	14.5	12.1				
				2842	0.881	11.9	12.3	12.7	13.9				

Dry Cycle #1	Undamaged												
	Cycle	↑	↓	HA	Si	SEC E1	SEC E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	352	-	169	0.31	10.9	12.3	15.7	13.3	0.000	0.001	0.171	0.95
		3	1828	5	7					2	0	4	1
	2	316	-	792	0.14	11.6	12.1	12.0	12.1				
		5	2373	3	3								
	3	345	-	789	0.12	11.2	11.6	14.5	10.9				
		5	2666	9	9								
	4	346	-	707	0.11	11.2	11.6	13.3	10.9				
		2	2755	4	4								
	5	353	-	708	0.11	11.2	11.2	10.9	12.1				
		2	2824	1	1								
				469	0.81	11.3	11.6	12.7	11.5				
				1	4								
	Moderately												
	Cycle	↑	↓	HA	Si	SEC E1	SEC E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	537	-	128	0.13	6.9	7.5	8.4	8.4	0.000	0.001	0.169	0.73
		7	4090	7	6					3	6	8	7
	2	493	-	106	0.12	7.0	7.2	7.2	9.7				
		4	3870	4	1								
	3	461	-	726	0.08	7.0	7.2	7.2	8.4				
		1	3886	5	5								
	4	373	-	146	0.24	7.0	7.1	8.4	8.4				
		4	2274	0	3								
	5	439	-	625	0.07	7.0	7.3	7.2	10.9				
		7	3772	7	7								

Wet Cycle #2	516 2 0.66 2 7.0 7.2 7.5 9.4												
	Severely												
	Cycle	↑	↓	HA	Si	SEC E1	SEC E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	666 1	- 3454	320 6	0.31 7	7.0	9.0	7.2	9.7	0.000 4	0.001 6	0.232 1	0.94 5
	2	499 9	- 3688	131 1	0.15 1	8.1	8.3	8.4	10.9				
	3	491 4	- 3808	110 5	0.12 7	7.7	8.1	8.4	9.7				
	4	473 7	- 3753	984 6	0.11 6	7.9	7.7	8.4	10.9				
	5	491 7	- 4019	898 0	0.10 0	7.3	7.5	7.2	9.7				
				750 4	0.81 1	7.7	7.9	8.1	10.3				
	Undamaged												
	Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	2512	-2144	367	0.079	11.4	12.1	14.5	21.7	0.0001	0.0006	0.0952	0.937
	2	2315	-2299	17	0.004	17.0	17.0	16.9	16.9				
3	1960	-1956	4	0.001	17.0	17.0	16.9	21.7					
4	2336	-2420	0	-0.018	16.2	17.0	16.9	19.3					
5	2303	-2357	0	-0.012	17.0	17.0	16.9	16.9					
			388	0.054	16.8	17.0	16.9	18.7					
Moderately													
Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI	
1	2570	-1866	704	0.159	17.0	18.1	16.9	14.5	0.0000	0.0006	0.0476	0.965	
2	2192	-1912	280	0.068	17.1	17.7	19.3	16.9					
3	2543	-2359	184	0.037	16.7	15.4	19.3	14.5					
4	2451	-2320	132	0.028	15.4	16.2	14.5	14.5					
5	2420	-2282	138	0.029	16.2	16.2	16.9	16.9					
			1436	0.321	16.4	16.4	17.5	15.7					
Severely													
Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI	
1	4171	-2591	1581	0.234	9.5	11.5	12.1	9.7	0.0002	0.0011	0.2051	0.856	
2	3338	-2674	664	0.110	10.8	11.1	12.1	14.5					
3	3674	-3029	645	0.096	10.8	10.8	12.1	12.1					
4	3622	-2998	624	0.094	10.4	10.3	9.7	12.1					
5	3677	-3065	612	0.091	9.9	10.3	12.1	14.5					
			4126	0.625	10.5	10.6	11.5	13.3					
Dry Cycle #2	Undamaged												
	Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	2378	-1689	689	0.169	15.4	20.2	14.5	16.9	0.0003	0.0008	0.3214	0.811
	2	2290	-1848	443	0.107	17.0	17.9	16.9	16.9				
	3	2382	-1985	397	0.091	17.0	19.0	16.9	16.9				
	4	2303	-1990	313	0.073	17.0	17.9	14.5	16.9				
	5	2319	-2069	251	0.057	17.0	17.0	19.3	14.5				
				2092	0.497	17.0	18.0	16.9	16.3				
	Moderately												
	Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	2971	-2036	935	0.187	12.9	18.0	14.5	19.3	0.0004	0.0009	0.4063	0.826
	2	2457	-2096	361	0.079	15.4	18.0	16.9	17.6				
	3	2411	-2155	257	0.056	16.2	17.0	16.9	16.9				
4	2297	-2115	182	0.041	16.2	17.0	16.9	16.9					
5	2278	-2159	119	0.027	16.2	17.0	16.9	16.9					
			1854	0.390	16.0	17.3	16.9	17.1					
Severely													
Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI	
1	3597	-2846	752	0.117	10.8	11.6	9.7	12.1	0.0001	0.0009	0.1613	0.817	
2	3318	-2925	393	0.063	11.2	11.6	9.7	14.5					
3	3155	-2794	361	0.061	11.6	12.0	12.1	14.5					
4	3021	-2695	326	0.057	12.0	12.0	12.1	14.5					
5	2881	-2562	319	0.059	12.0	12.4	9.7	14.5					
			2150	0.356	11.7	12.0	10.9	14.5					
Undamaged													

	Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI
	1	2741	-2217	524	0.106	17.0	18.0	16.9	19.3	0.0000	0.0006	0.0500	1.040
	2	2290	-2157	134	0.030	18.0	18.0	16.9	16.9				
	3	2482	-2378	104	0.021	17.0	17.0	16.9	16.9				
	4	2407	-2343	65	0.014	17.0	17.0	16.9	14.5				
	5	2366	-2345	21	0.004	17.0	17.0	16.9	16.9				
				848	0.175	17.3	17.3	16.9	16.3				
Moderately													
Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI	
1	2706	-2146	560	0.115	15.4	17.0	14.5	16.9	0.0001	0.0006	0.0909	0.940	
2	2332	-2309	23	0.005	16.2	16.2	16.9	14.5					
3	2403	-2303	100	0.021	16.2	16.2	16.9	16.9					
4	2403	-2257	146	0.031	16.2	16.2	16.9	14.5					
5	2340	-2236	104	0.023	16.2	16.2	16.9	14.5					
			933	0.196	16.2	16.2	16.9	15.1					
Severely													
Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI	
1	3361	-2779	583	0.095	11.6	11.6	12.1	12.1	0.0000	0.0009	0.0000	0.836	
2	3364	-2996	367	0.058	11.2	11.2	12.1	12.1					
3	3301	-2958	342	0.055	11.2	11.2	12.1	9.7					
4	3259	-2921	338	0.055	11.2	11.2	12.1	12.1					
5	3205	-2883	322	0.053	11.1	11.1	12.1	12.1					
			1952	0.315	11.1	11.1	12.1	11.5					
Undamaged													
Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI	
1	2136	-1610	526	0.140	20.2	23.1	19.3	19.3	0.0001	0.0006	0.1579	0.902	
2	2036	-1798	238	0.062	20.2	20.2	19.3	19.3					
3	2061	-1931	129	0.032	19.0	19.0	19.3	16.9					
4	1981	-1823	159	0.042	19.0	20.2	19.3	19.3					
5	1890	-1718	171	0.047	20.2	20.2	19.3	21.7					
			1223	0.324	19.6	19.9	19.3	19.3					
Moderately													
Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI	
1	2382	-1802	580	0.139	19.0	20.2	19.3	16.9	0.0001	0.0005	0.1111	1.047	
2	1927	-1718	209	0.057	20.2	20.2	19.3	19.3					
3	1839	-1497	342	0.103	20.2	21.6	19.3	19.3					
4	1781	-1687	94	0.027	21.6	21.6	21.7	19.3					
5	1981	-1910	71	0.018	20.2	20.2	19.3	19.3					
			1297	0.344	20.6	20.9	19.9	19.3					
Severely													
Cycle	↑	↓	HA	Si	E1	E2	Tan E1	Tan E2	DI	DII	PDI	NLI	
1	2587	-1839	747	0.169	13.5	15.4	12.1	14.5	0.0001	0.0008	0.1481	0.765	
2	2679	-2119	560	0.117	14.1	14.7	14.5	14.5					
3	2503	-413	2090	0.717	14.7	14.7	14.5	12.1					
4	2328	-1960	367	0.086	14.7	14.7	14.5	14.5					
5	2395	-2128	267	0.059	14.1	14.1	14.5	16.9					
			4032	1.147	14.4	14.5	14.5	14.5					

Appendix G – Bulk Residual Potential Expansion

Day	1					2					6				
	Mass	A	B	C	D	Mass	A	B	C	D	Mass	A	B	C	D
0	3163.0					3325.9					2895.0				
30min	3226.3	944	961	964	953	3366.3	973	952	954	990	2960.9	965	950	954	958
7	3263.1	965	987	999	985	3401.2	996	977	982	1022	2991.4	985	984	979	982
14	3264.9	968	989	1002	990	3401.8	998	978	983	1027	2993.1	986	992	980	985
21	3265.3	970	990	1004	994	3401.3	999	980	984	1027	2994.7	987	993	982	988
28	3266.4	971	990	1005	997	3402.9	1000	981	984	1028	2995.4	988	993	985	990
35	3267.0	971	992	1006	998	3403.4	1001	981	985	1029	2996.2	989	995	990	990
42	3266.6	972	993	1010	998	3403.5	1002	981	989	1030	2995.9	990	998	992	991
49	3267.3	973	994	1012	1000	3403.6	1002	982	991	1031	2996.4	990	1000	992	992
56	3266.7	973	994	1012	1001	3403.2	1002	983	991	1032	2995.4	992	1001	993	992
63	3266.9	974	995	1013	1001	3403.2	1003	984	992	1033	2995.6	992	1001	994	992
70	3267.3	975	996	1013	1001	3403.7	1004	984	992	1033	2996.7	993	1002	994	992
77	3266.7	975	996	1013	1002	3403.1	1004	985	992	1034	2995.8	993	1003	995	993
84	3267.1	975	996	1013	1002	3403.4	1005	985	993	1034	2995.9	993	1004	995	993
91	3267.1	975	996	1014	1003	3403.8	1005	985	993	1034	2995.6	993	1004	995	993
112	3268.0	975	997	1015	1005	3403.8	1007	987	995	1035	2995.6	996	1006	996	997

Appendix I – Ethics Clearance



HUMAN RESEARCH ETHICS COMMITTEE (NON-MEDICAL)

Registration number: REC-101114-044

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING ETHICS COMMITTEE

Ethics clearance number: CEECW 0710524F 2019

1st April 2019

Re: Janina Prakash Kanjee (0710524F)

To whom it may concern,

Janina Prakash Kanjee (0710524F) is currently registered as a PhD student at the School of Civil and Environmental Engineering, University of the Witwatersrand, Johannesburg. This letter is to confirm that, at the time of writing, Janina Prakash Kanjee does not need ethical clearance for her study entitled '*Assessing the potential expansion of Alkali Silica Reaction (ASR) in existing reinforced concrete structures using Digital Image Correlation (DIC)*'. This decision has been reached based upon a description of the project supplied by Janina Prakash Kanjee to the School of Civil and Environmental Engineering Ethics Committee, constituted as a subcommittee of the University Human Research Ethics Committee (Non-Medical), which has been evaluated by the subcommittee chair. This decision has then been ratified by the University Human Research Ethics Committee (Non-Medical). If, however, Janina Prakash Kanjee changes the methods of data collection and analysis for this project, this decision may no longer be valid. If such changes take place, this should be communicated to the School of Civil and Environmental Engineering Ethics Committee.

Please feel free to contact me should you require any further information.

Thank you.

Yours sincerely,

A handwritten signature in black ink, appearing to read 'John Ndiritu'.

Prof John Ndiritu, School Ethics Chair

Shaun Schoeman (Senior Administrative Officer)

Solomon Mahlangu House, 10th Floor, Room 10004, Jorissen Street, Braamfontein, Johannesburg
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