ASSESSMENT OF THE DURABILITY PERFORMANCE OF AGED REINFORCED CONCRETE WATER RESERVOIRS IN GAUTENG

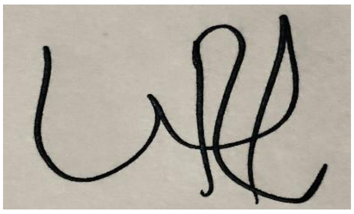
Marcia Retlogetswe Maidi

A research report submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, in fulfillment of the requirements for the degree of Master of Science in Engineering

Johannesburg, 2020

DECLARATION

I declare that this research report is my own unaided work. It is being submitted for the Degree of Master of Science to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any University.



……………………………………………………………………………………

(Signature of Candidate)

23rd day of **September** year **2020**

ABSTRACT

This study assessed the durability performance of aged reinforced concrete reservoirs. The objectives were to conduct audits/surveys of Rand Water reservoirs and how they have performed over time through assessments of the selected structures with regard to durability performance and aggressiveness of the in-service exposure environment of the reservoirs.

The assessment was conducted in phases: preliminary assessments and detailed assessments. The preliminary assessments were used to characterise the nature of the deterioration and the possible mechanism and gave guidance in planning for the detailed assessments, based on visual inspections. The detailed assessments were then used to confirm the cause of deterioration and to quantify the severity of the deterioration mechanisms.

Cores were extracted from the reservoirs, and on-site visual assessments and laboratory tests were carried out. The tests included surface hardness assessment using the Schmidt hammer, compressive strength, cover depth and carbonation depth. An attempt to quantify the impact and effect of durability performance using durability indices and corrosion indices was also conducted.

The results from the durability indices tests, indicate that the concrete is of poor quality. The Water Sorptivity Index (WSI) test results ranged between 3.16 and 12.79, signifying concrete susceptible to moisture ingress by absorption for some reservoirs. The Oxygen Permeability Index (OPI) values ranged between 8.02 and 9.43, signifying poor quality concrete with regard to permeability for all reservoirs. The Chloride Conductivity Index (CCI) values ranged between 0.58 and 1.62, signifying susceptibility to chloride penetration leading to corrosion for some reservoirs. The indices characterise the near surface properties of concrete and measure its resistance to fluid and ionic transport mechanisms; therefore, a reduced quality in the surface concrete results in its inability to protect the reinforcement against corrosion.

The study indicated that all reservoirs tested for durability performance experienced poor concrete quality and were prone to corrosion due to carbonation, Alkali Silica Reaction (ASR), insufficient cover layer and aggressive waters.

DEDICATION

In loving memory of my Grandmother, **Ema**.

She who believed in the education of the Girl child.

Maria Mashushu Maidi

ACKNOWLEDGEMENTS

I would like to express my gratitude to the following people and organisations for their assistance in my research:

- My supervisor, Dr Mike Otieno, for his assistance and continued guidance throughout this process. I would like to express my gratitude to you Mike, for not giving up on me when all hope was lost. This has been a journey that you have walked with me for a long time and I am very grateful.
- Prof Yunus Ballim for his guidance.
- Rand Water for allowing me the opportunity to conduct this research.
- Everyone who has contributed to, and assisted me with this research; my colleagues, the lab technicians, the contractors and consultants who made this possible-thank you.

To my loving family and my daughter, Lesedi - thank you for the unwavering support, belief and love - *ke leboga go menagane*.

CONTENTS

List of Figures

List of Tables

LIST OF ABBREVIATIONS

AI	Aggressivity Index
ASR	Alkali Silica Reaction
BS	British Standards
$CaCO_3$	Calcium carbonate
$Ca(OH)_2$	Calcium hydroxide
CCI	Chloride Conductivity Index
CEM I	Portland Cement Type I
DI	Durability Index
GFIP	Gauteng Freeway Improvement Project
H_2CO_3	Bicarbonate
HCP	Hardened Cement Paste
ITZ	Interfacial Transition Zone
LCSI	Leaching Corrosion Sub Index
SCSI	Spalling Corrosion Sub Index
M/	Megalitres
MPa	Mega Pascals
NDT	Non-Destructive Testing
NGL	Natural Ground Level
OPI	Oxygen Permeability Index
RW	Rand Water
SANRAL	South African National Roads Agency Limited
SANS	South African National Standards
SCCI	Spalling Corrosion Sub Index
SLS	Serviceability Limit State
TWL	Top Water Level
ULS	Ultimate Limit State
W/B	Water Binder
WRS	Water Retaining Structures
WSI	Water Sorptivity Index

CHAPTER 1 - INTRODUCTION

1.1 Background

Rand Water® is a bulk clean (potable) water supplier with reinforced and pre-stressed concrete water reservoirs ranging between 5 and 640 million litres in South Africa. It has a network traversing throughout four of the nine provinces (Gauteng, Free State, Mpumalanga, and parts of the North West) and operates a network of 62 reservoirs, all with different lifespans experiencing similar problems of concrete deterioration leading to unacceptable rates of leakages and reduced ability to maintain their serviceability. The lifespan of the chosen test structures ranges between 24-84 years.

Concrete structures as many other engineering structures are subjected to deterioration that affect their integrity, stability and safety, faced with the importance of the damages noted, the current choices are directed towards the repair of the existing structures rather than towards demolition and construction of new ones (Chandramouli, 2009). Before any repair work can be done, it is common practice to study the existing structures by analysis after ascertaining the quality of the material and its durability as all structures are designed to be functional and durable for a specific period of time.

"*Durability can be defined as the capability of maintaining the serviceability of a product, component assembly or construction over a specified time. Hence durability is a material performance concept rather than an intrinsic material property, associated with the deterioration of the material over the intended service life of the structure in a given environment*" (Ballim et al., 2001).

Deterioration of concrete structures is often associated with the ingress of aggressive agents, through the cover layer. The quality of the concrete cover layer and its ability to resist the ingress of aggressive and deleterious materials from the environment is a major controlling factor for durability (Alexander et al., 2008). In the case of water reservoirs, attack can also be from internal sources due to aggressiveness of the contained liquid.

The aggressiveness of the contained liquid is dependent on the chemicals used at each stage of the chemical treatment process, some with more detrimental effects on the durability of the concrete reservoirs than others in-service. The exposure conditions that the Rand Water network of reservoirs is subjected to are as follows:

- Raw water: This is the source of water to the entire operation directly from the Vaal Dam;

- Chemically dosed water: different chemicals (e.g. polyelectrolyte, silica, lime, ferric chloride, carbon dioxide) are used throughout the different processes to get the water to the acceptable level of consumption;
- Potable water: This is the final product produced, dosed with prescribed limits of disinfectants (i.e. chlorine and ammonia).

Potable water reservoirs are an important part of the distribution network, as they represent the final point at which the water quality can be checked and corrected before it reaches the consumer. It is becoming increasingly important to design and manage concrete reservoirs carefully so as to ensure safe drinking water right up to the consumer's tap.

In addition to the aggressiveness of the contained liquid, damage to reinforced concrete structures due to carbonation and chloride induced corrosion is widespread in South Africa. The damage to concrete structures is usually found in environments where carbon dioxide levels are high. South Africa is ranked the twelfth largest generator of CO_2 in the world (Ikotun, 2017). The source of the emissions arises from the electrical, steel and transport industries. There is expectation that these emissions will increase as South Africa is highly dependent on these industries to pursue economic growth and development, until such developmental and economic goals are achieved. This is also expected to have a considerable effect in reducing the service life of reinforced concrete structures.

To assess long-term durability performance and aid in the control of the quality of concrete structures, three durability index tests were developed in South Africa. The tests measure parameters related to the transport mechanisms in concrete. The tests comprise of the oxygen permeability for gas permeation, chloride conductivity for diffusion, and water sorptivity for absorption. Durability indices were developed to provide a sufficient guide to describe the quality of concrete. These indices are comprised of key parameters that will enable service life model predictions; however service life models are also influenced by parameters like steel reinforcing and environmental class. An empirical link between service life models and durability index test results has been established. Once the indices are quantified, they can then be utilised in construction specifications to achieve the necessary concrete quality required to sustain the service life of the structure (Alexander et al., 2008).

Thus, a framework is available for a performance-based approach to both design and specification, using the durability indices. The implementation of the Durability Index (DI) approach as part of durability design and specification was successfully implemented in South Africa by the South African National Roads Agency (SANRAL). As part of the literature review, the pros and cons of this will be assessed. Appropriate test methods, and interpretation of the test results from the DI approach, form the basis of a performance approach to design and specification of concrete structures. This is essential to ensure durability and a minimum lifespan of the concrete structure (Alexander et al., 2008).

For the purposes of this study, concrete deterioration of some of the Rand Water reservoirs will be studied. Selection criteria for the review and assessment of the concrete reservoirs had to be developed. The following three selection criteria were used:

- Age vs. design Life: how old are the structures? Have structures actually performed against originally intended design lives?
- Location and environment to which the selected reservoirs are exposed.
- Aggressivity of the contained liquid i.e. chemically dosed water.

1.2 Problem statement

1.2.1 Defects in concrete structures resulting in reduced service life

Civil structures are susceptible to various kinds of defects and deterioration mechanisms. These defects may present in forms such as cracking, corrosion, honeycombing, voids and delamination of cover.

Defects in the concrete elements may have long-term or serious implications for the serviceability of water reservoirs. These defects may exist for a number of reasons namely: poor design/specification; construction workmanship; operational issues; or incorrect use (e.g. overloading), durability, and the structure reaching the end of its service life. It is for this reason that the investigation was initiated.

1.2.2 Quality of concrete resulting in reduced service life

"*The quality of concrete has traditionally been assessed by measuring its compressive strength at a pre-determined age. However, it has since been discovered that this approach does not give an adequate indication of the quality of the surface zone of the concrete*" (Ballim et al., 2001).

The main factors controlling the surface or the cover zone of concrete are a) deterioration rate, b) constituent materials, c) construction quality, and d) the aggressiveness of the exposure environment (Ballim et al, 2009).

The ability of the cover layer to protect steel reinforcement is hindered by the ingress of aggressive agents such as chloride ions or acidification from the surrounding environment, and this remains a critical challenge for engineers, contractors and concrete producers (Alexander et al., 2008). The ability to quantify the quality of the cover zone in terms of parameters that can be measured, (such as durability indices), form the basis of performance-based specifications that may assist in improving overall quality of reinforced concrete structures, (Alexander et al., 2008). Strategies are required to cater for durability design to address the adequacy of the cover zone.

1.2.3 Lack of specifications that may result in reduced service life of structures

Reinforced and/or pre-stressed concrete water-retaining structures are commonly used in South Africa for the purposes of storage. The bulk of Rand Water reservoir designs and construction methodologies are based on prescriptive specifications. The structures are all designed as per the prescriptions of the BS 8110 and BS 8007 as a South African code pertaining to the design of concrete water-retaining structures does not currently exist. This approach has proved invaluable in determining durability requirements for tailor-made concrete, since prescriptive-based approaches alone have been found to be inadequate. The same deterioration modes of the concrete structures have been assessed over time, affecting all the different types of reservoirs within the network of reservoirs. This has necessitated the need for new ways in which reservoirs have to be designed and specified through the cover layer and the cementitious materials.

Currently Rand Water has no concrete durability specifications dedicated for its water-retaining structures; therefore, the study will address and look into the need for a performance-based design approach and specification, for various durability properties. The need for performance based design approaches arises from an influx of new cementitious materials and an increase in the rate of deterioration in reinforced concrete structures. Efforts need to be made to enable the quantification of in-situ performance of reinforced concrete structures. The study will serve as a basis to promote a deeper understanding of concrete durability, performance-based specification, as well as the implementation of the DI Approach within the Rand Water system.

1.2.4 Aim and objectives of the research

The aim of this study is to lay a foundation for the development of performance-based design approaches and specifications, taking into account the various durability parameters that affect the design and the construction of the structures within the Rand Water system.

The objectives of the study are as follows:

(i) Conduct audits/surveys of Rand Water reservoirs and how they have performed over time, through preliminary and detailed assessments of the selected structures with regard to durability. The following aspects will be looked at:
- Have they performed as per their design requirements?
- How much longer can they be in service, considering the deterioration mechanisms observed on site?

(ii) Look into the design considerations and parameters that have an impact on the durability of the reservoirs, as per the prescriptions of the design codes mentioned earlier, i.e. BS 8007, and their adequacy for concrete durability design in water retaining structures.

(iii) Assess or quantify the aggressiveness of the in-service exposure environment of the reservoirs.

(iv) Conduct laboratory testing to assess the durability performance and behaviour of the mature reservoirs in service and initiate modifications to the current specification utilised for reinforced concrete structures.

It is envisaged that the effectiveness to the modifications to the Rand Water specification that will be initiated as a result of this study will be tested through implementation in future construction projects, its viability assessed throughout, and that will form the basis for further study.

1.3 Implementation of research methodology

The study was conducted in three phases to achieve the objectives of the research. The first phase consisted of the literature review and review of Rand Water reservoir network which resulted in the selection of test structures and their exposure environment, the second phase consisted of preliminary investigations which entailed the visual assessment, core extraction and on-site testing. The last phase involved detailed assessments which entailed laboratory testing and analysis. The analysis also included a review of records of water quality samples, five to ten years of water quality analysis results from Rand Water was used in the analyses.

1.3.1 Literature review

An extensive review of literature regarding durability and performance models of existing water retaining structures in South Africa was carried out. The following was part of the review:

- Concrete durability and the factors influencing durability.
- Deterioration mechanisms and their effects on durability.
- Review of the existing Rand Water reservoirs, their design and specification aspects.
- Approaches and specification for durability performance and their usefulness.
- Review of performance based specification on existing or mature structures.
- Importance of durability design and improvements to future design codes of practice.
- The benefits of a performance specification to Rand Water structures.

1.3.2 Preliminary field assessments and detailed assessments

Durability can be assessed by conducting various tests methods that allow for the deterioration process likely to affect the structure to be studied, understood and quantified. The preliminary assessments were used to characterise the nature of the deterioration and the possible mechanism and give guidance in planning for the detailed assessments based on visual inspections and the detailed assessments were used to confirm the cause and quantify severity of the deterioration mechanism (Otieno, 2010).

Visual inspection was conducted on the test structures and cores were extracted on specified locations, for further testing. All tests conducted were utilised to characterise the durability performance of the test structures.

The following onsite and laboratory tests were conducted on the extracted cores:

- Surface hardness
- Compressive strength
- Cover depth surveys
- Carbonation
- Durability index tests
- Water quality test results analysis

Water quality tests on the reservoirs are conducted by the Scientific Services Division of Rand Water and the results were made available to conduct the assessment of aggressivity and analysis using Basson indices.

1.4 Scope and limitations

The scope of the study focused only on durability as described quantitatively by the measurable parameters on the DI Approach and the SANS acceptable standards. The basis of the DI Approach was to enable the quantification of the relevant parameters in order to yield a suitable description of concrete quality. The following were also identified as challenges, as far as the study was concerned:

(i) Access to sites for visual inspections and detailed assessments, due to operational requirements;
(ii) Inability to conduct petrographic analysis for the test reservoirs. This would have assisted in analysis of the durability performance of the reservoirs;
(iii) Unavailability of existing records regarding the test reservoirs i.e. bulk properties of the concrete, material used during construction, and challenges that were faced during design and construction;

(iv) Inconsistencies in monitoring of key parameters required to compute aggressivity analysis.

Due to the age of the reservoirs and the poor concrete quality, the coring phase of the study was prolonged and complex; some cores would break in the middle of the core extraction exercise.

1.5 Report outline

This research report is divided into five chapters as follows:

The first chapter, the introduction, gives a brief framing of the whole study. It includes a background of deterioration mechanisms in concrete, the motivation and significance of the research, the objectives, and scope limitations.

The second chapter presents the literature review which focuses on durability performance of mature structures, and properties impacting durability.

The third chapter describes the experimental investigations conducted. It highlights all the test procedures utilised in preliminary assessments i.e. visual assessments and onsite testing and detailed assessments i.e. laboratory testing.

The fourth chapter presents the results, analyses and discussion of the experimental investigations described in Chapter 3.

The fifth chapter presents the general conclusion of the chapters and recommends further studies, followed by a list of all the references and appendices, that were used in this study.

CHAPTER 2 - LITERATURE REVIEW

2.1 Introduction

Concrete is a composite material consisting of coarse aggregate bonded together with hardened cement paste. The bonding medium is a mixture of water, cement and supplementary cementitious materials. The microstructure of concrete is composed of the following components:

- "*Hydrated cement paste: the hydration reactions between water and the calcium silicates and calcium aluminates present in Portland cement produce the binding medium known as the cement paste. The paste itself can be regarded as a system of constituents forming a microstructural framework through which the transfer of loads takes place" (Angelucci, 2013).*
- *"Interfacial transition zone (ITZ): formed between aggregate and cement paste and is often characterized by a microstructure with increased porosity, relatively weak crystalline phases and increased permeability" (Tragardh, 1999).*
- *"Aggregates: are used as diluents of the cement paste in order to make the concrete dimensionally stable, they are characterized by their porosity, absorptivity and dimensional properties. The dimensional stability of the concrete is influenced by aggregate properties such drying shrinkage, thermal volume change, wetting expansion, elastic stiffness and creep" (Grieve, 2001).*

Many factors have been reported in literature to influence the microstructure and the properties of concrete. According to Tragardh (1999), such factors include Water Binder (w/b) ratio, type and particle size distribution of aggregates and cement, permeability and construction practices such as curing, amongst others.

Durable concrete is therefore quality concrete, and a quality product is one that meets its desired expectations over its intended lifespan. To achieve the quality desired, therefore, it is important to take into consideration the characteristics of the cement, aggregates and mixing water, as well as their proportions (Basson, 1989).

2.2 Concrete durability

"In the context of structural concrete, durability can be defined as the ability of a structure or component to withstand the design environment over the design life, without undue loss of serviceability or need for major repair" (Ballim et al, 2009) further states that durability therefore relates to the concept of material performance and cannot be regarded as an intrinsic material property, but can be regarded as the deterioration of a material over the intended service life of the structure in a given environment, as concrete durable in one environment may not be durable in another.

According to Richardson (2000) the bulk of the durability problems arise from the following:

- change in exposure conditions;
- poor material choice;
- inadequate designs;
- inadequate reinforcement detailing;
- construction methodology and/or practices i.e. poor workmanship affects quality, lack of understanding of specifications, inexperienced workforce;
- not adhering to specifications as stipulated in the drawings or tender documents;
- over and under specification, i.e. over specification is wasteful of resources and unjust to the client, while under-specification leads to costly repair works.

Engineers need to consider durability in the design and specification of their structures for adequate service life.

2.3 Factors influencing the durability of concrete

Concrete durability is an interaction between the concrete as a system and its environment. According to Ballim et al (2009) the concrete system will influence the ability of the concrete to resist deterioration, while the environmental factors influence the degree of aggressiveness that the concrete has to withstand. This system interaction is demonstrated in Figure 2.1 (Ballim et al, 2009). The factors which are most likely to dominate in any given durability assessment are binder type, water-binder ratio for intrinsic factors (Ballim et al, 2009).

Curing and early age temperature history are the dominant extrinsic factors as stated in Ballim et al (2009); it is very important to permit proper strength development, aid moisture retention to ensure hydration process occur completely. The most critical aspect of the environmental factor is the aggressiveness of agents attacking the concrete. The effects of cracking speed up the environmental deterioration as cracking may accelerate ingress of harmful substances into the concrete (Ballim et al, 2009).

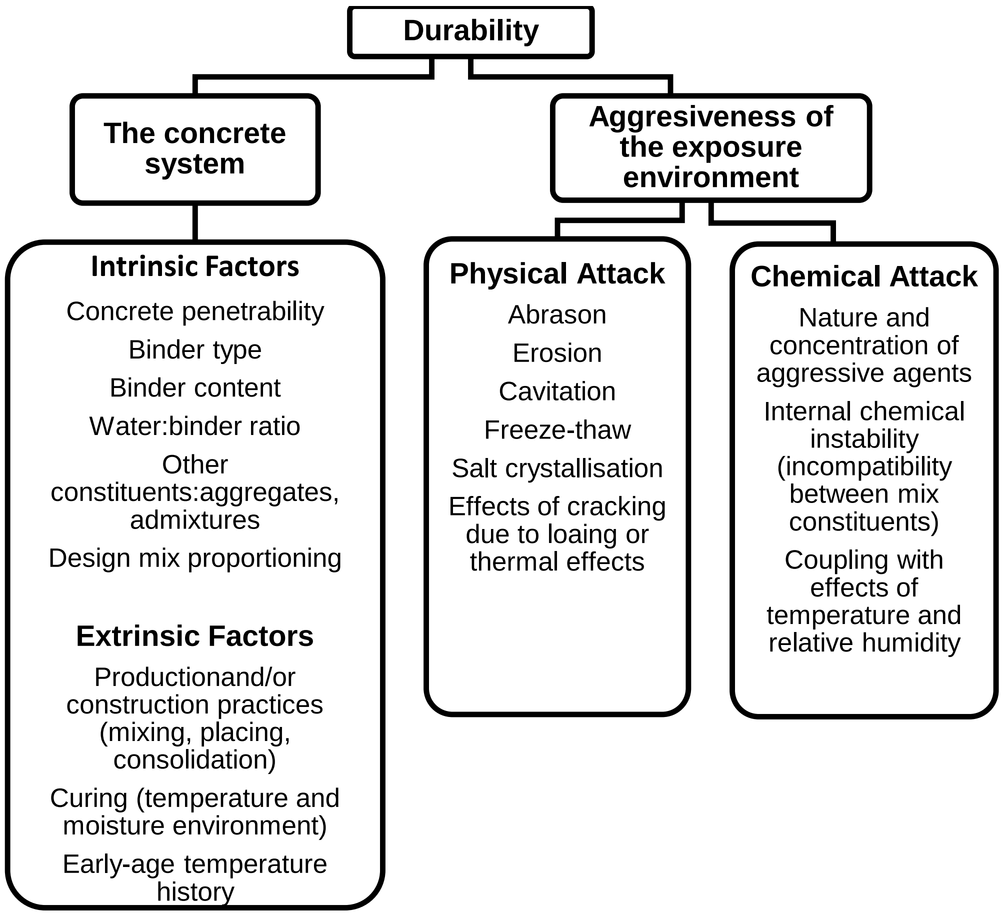


Figure 2.1: Concrete and Environment: Factors influencing durability (Ballim et al, 2009)

Amongst other factors affecting the durability of concrete are the concrete cover, permeability and penetrability of the cover layer. Higher permeability is usually caused by higher porosity. Concrete as a whole system contains voids that can also be caused by inadequate compaction. There are three main factors controlling the quality of the surface zone of the concrete and its deterioration rate namely, the constituent materials, construction quality and the aggressiveness of the exposure environment (Alexander et al., 2008).

Lampacher (2000) asserts that “fundamental to an understanding of durability is the need to visualise a typical cross-section of a reinforced concrete member as having two distinct zones”. This concept is shown schematically in the Figure 2.2.

The area of concrete between the outer surface and the shallowest reinforcing steel is called the covercrete and its role is to act as a protective barrier between the external environment, the underlying reinforcement steel and inner zone of concrete called the heartcrete. The role of the heartcrete is to provide the bulk properties of a structural member, such as its compressive strength and modulus of elasticity, while the characteristic bulk properties, accounted for by the heartcrete, are very important for the structural behaviour of a reinforced concrete member, it is the transport properties i.e. permeation and absorption of that concrete forming the covercrete zone that are of particular relevance to durability from a steel protection point of view (Lampacher, 2000).

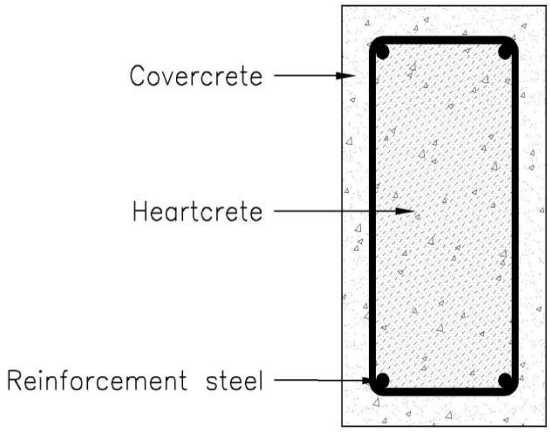


Figure 2.2: Typical Concrete Cross Section

2.3.1 Transport mechanisms

Transport mechanisms play a major role in predicting the durability of concrete. The transport systems are the key drivers of any deterioration mechanism, as they permit the passage of potentially aggressive ions, in the form of liquids and gases, into the concrete microstructure. The penetrability of the concrete thus manifests through diffusion, absorption, permeation and migration (Ballim et al, 2009). Transport mechanisms and properties of the cover layer measured in-situ and in the laboratory, can be used to characterise durability of the concrete. The cover layer has a direct impact on durability by controlling the movement of aggressive ions from the environment into the concrete (Alexander, 2008).

2.3.2 Deterioration mechanisms

There are a number of mechanisms that cause the deterioration of concrete namely:

- Chemical mechanisms: carbonation; alkali silica reaction; sulphate attack; chloride attack;
- Corrosion of steel;
- Physical mechanisms as stated in Figure 2.1.

2.3.2.1 Deterioration mechanisms experienced by Rand Water reservoirs

Rand Water conducts assessments on its reservoirs every five years, for maintenance and legislative purposes. Defects and deterioration mechanisms that have been observed in all the structures include cracks due to expansive forces, tension, products of corrosion; spalling;

inadequate cover to reinforcement; exposed reinforcement, efflorescence; carbonation; allultimately leading to corrosion of reinforcement. All these defects have led to secondary serviceability problems, such as leakages.

The composition of the water that is used by Rand Water is subject to chemical reactions that may be privy to cat-ion and ion-addition. “The salts dissolved in water are almost invariably dissociated into their components ions which are then free to participate independently of their original partners in various chemical reactions. The most important of these in the context of durability of concrete, mainly corrosion of concrete, are those ion-exchange reactions where an ion present in the liquid phase is exchanged for one in the solid phase” (Ballim & Basson, 2001).

“Water is a ubiquitous corrodent of concrete” (Ballim et al, 2009). The Rand Water reservoirs contain water treated with some form of chemical i.e. lime, ferric chloride, sodium silicate, carbon dioxide and chlorine, depending on the stages of the treatment process. The minerals used in water treatment processes could have an impact on the durability of the concrete. The impact of this will be explored further in later chapters.

The foremost line of defence against aggressive waters, however, is to use a sound, dense impervious concrete, made from the appropriate materials and properly compacted and cured (Basson, 1989). In the case of highly aggressive waters even the best concrete may not be sufficiently resistant to the corrosive forces that are present, and supplementary protection by means of coatings becomes mandatory to achieve adequate durability of the completed structure.

2.3.2.2 Deterioration mechanisms on mature structures

Although reinforced concrete structures can be expected to show good durability, many structures are experiencing deterioration caused by reinforcement corrosion. These are observed all over the country. In a study conducted by (Alexander et al, 1996), they observed that the underlying reasons for the deterioration of the structures could possibly be linked to lack of understanding of the corrosion mechanisms and inadequate specification for achieving durable concrete at the design stage, poor workmanship at construction stage and a lack of knowledge and application of maintenance management strategies by owners of the structures.

The factors that play an important role and influences on the rate of deterioration of the structures is the external environment i.e. the amount of chlorides and carbon dioxide in the air which cause concrete to deteriorate more rapidly. The cover to reinforcement, the strength of concrete used, the location of the element in the structure in question and whether the structure has been coated, painted and repaired before, are other important factors that lead to the deterioration of the structures (Alexander, 1996). The lack of as-built information (i.e. cover to reinforcement details, design strength and water-cement ratios) in the assessment of the mature structures is always cited in the studies, as an obstacle, to conduct proper diagnostics.

Studies conducted in the coastal provinces of the country i.e. KwaZulu Natal and Western Cape, it was observed that most structures in the coastal region begin cracking and spalling significantly due to chloride ingress, between the age of 20 - 25 years and those in a severe exposure climate at 15 years (Alexander et al, 1996). Some of these structures were found to be in poor condition and in need of repairs. It is of greater concern that structures that are midway to the design lifespan are exhibiting premature deterioration. Contributing factors to this early damage includes bad designs, inadequate specifications and poor construction practice. In the studies conducted by Lampacher (2000), in the inland Gauteng province, indicated that carbonation is the main deterioration mechanism that leads to corrosion of reinforcement and the rate of carbonation was found to be rapidly high. This implies that comparatively early depassivation of the steel and the inherent risk of steel corrosion.

2.4 Effect of deterioration mechanisms on durability

2.4.1 Carbonation

Literature and research by different institutions suggests that carbonation is the primary deterioration mechanism that leads to corrosion of reinforcement in Gauteng province, South Africa and its surrounding areas. Some institutions have gone as far as releasing carbonation models for their infrastructure and such models will form the basis of this research. The studied carbonation models were looking into bridge infrastructure and therefore the design parameters will be different to water tanks as water tanks are designed as water retaining structures with stringent requirements. The test structures for this research, i.e. Rand Water reservoirs are all based in the Gauteng region, and therefore are most likely to experience carbonation, alkali silica reactions and sulphate attack, all leading to reinforcement corrosion. According to Basson (1989), Gauteng’s inland location and warm climate promote rapid carbonation. The influence of high temperatures, optimum humidity and significant atmospheric pollution levels combine to make this region vulnerable to carbonation (Lampacher, 2000).

Carbonation of concrete is a chemical reaction between alkaline hydroxides of concrete and carbon dioxide gas both dissolved in concrete pore water. The reaction product is calcium carbonate, which lowers the pH of the pore water (and concrete) gradually to a level where steel can corrode. As the alkaline hydroxide reservoir in concrete is limited, it is eventually used up leading to the neutralisation of concrete as stated by (Pakkala et al., 2014).

Carbonation has been found to involve a number of processes, from the aggressive environment of gas diffusion to the beginning of the corrosion itself, and there are several other parameters whose variability cannot be ignored. In a study conducted by Pakkala (2014) it was found that the rate of carbonation of concrete varies a lot according to the properties of concrete, because concrete is not a homogenous material, the durability properties varied due to e.g. compaction and curing of fresh concrete. In general, the carbonation rate of concrete is slow in dense, low w/b ratio concrete, where curing has been careful. Permeability, mix constituents and environmental conditions, including climate change, are major factors influencing carbonation rate.

2.4.1.1 Effects of carbonation

In addition to the associated implications of steel corrosion and reducing the pH of concrete, carbonation has been found to have the following additional effects:

- *"*Carbonation shrinkage: it has been observed that carbonation can cause significant shrinkage and increase creep. This form of shrinkage is typically evident in smooth "off-shutter" concrete as surface crazing. It is probably caused by dissolving of crystals of $Ca(OH)_2$ while depositing under a compressive stress, imposed by the drying shrinkage and depositing $CaCO_3$ in spaces free from stress*"* (Lampacher, 2000).
- Reduced porosity and permeability: carbonation reduces porosity and may reduce permeability. Lampacher (2000) indicated that this is because the volume of $CaCO_3$ formed exceeds the volume of the original $Ca(OH)_2$ by approximately 12%.
- Increased strength, surface hardness and elastic modulus: as carbonation progresses, these parameters increase in the carbonated zone.

2.4.2 Corrosion to reinforcement

Corrosion of steel in concrete is a complex phenomenon, influenced by many internal and external factors. These include: the pH of the concrete pore solution; temperature; internal stresses; stray currents and electrolytic potentials (Mackechnie, 2001). Steel reinforcement is protected from corrosion by high alkalinity of concrete pore solution. This alkalinity is over time neutralised by carbon dioxide in the surrounding air or chlorides penetrating the concrete cover leaving the reinforcement susceptible to corrosion.

Corrosion of reinforcement in concrete structures is divided into two phases: an initiation period during which little damage occurs as ions diffuse towards the reinforcement; and a propagation phase in which damage occurs progressively once the corrosion threshold has been exceeded and corrosion has been initiated as shown in Figure 2.3.

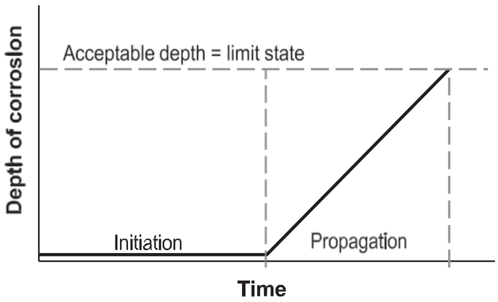


Figure 2.3: The two-phase model of corrosion of reinforcement in concrete

Reinforcement cast into concrete is initially rendered passive by a protective layer of ferric oxide that forms under normal alkaline conditions (pH12.5 – 13.5). The ferric oxide layer may be disrupted by the presence of sufficient chloride ions, or by a reduction in pH of concrete below 10.5 due to carbonation or acidification (Alexander, 2008).

Pakkala (2014) explains the process of corrosion as shown in Figure 2.3, where he states that corrosion of steel embedded in concrete is an electrochemical reaction where anode and cathode areas are formed on the steel surface as a result of differences in pH and moisture. Anode areas release positively charged hydrated ions in oxidation reaction into the pore water acting as an electrolyte. The excess electrons flow through metal from the anode to cathode where they are consumed in reduction reaction by hydrogen ions or by dissolved oxygen. Once de-passivation has occurred, corrosion of steel is possible as shown schematically in Figure 2.4, as depicted in Mackechnie (2001).

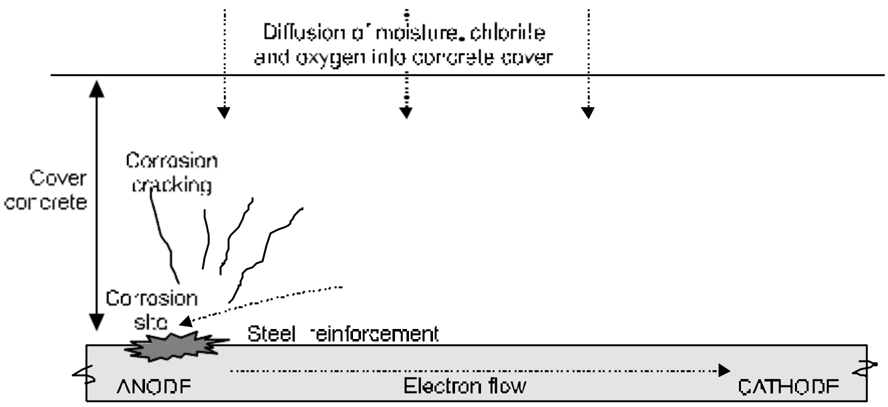


Figure 2.4: Schematic diagram of corrosion of reinforcement in concrete

The corrosion propagation phase is now appreciated as a significant component in the service life of reinforced concrete structures and a good understanding of the propagation process is paramount. Various models have been developed to simulate and/or predict the propagation phase, as stated by Pakkala (2014).

2.4.2.1 Factors influencing the rate of corrosion

Cracking: Cracking occurs in structures where the reinforcement is placed quite near the concrete surface and the pressure generated by corrosion products is sufficient. Cracking accelerates the penetration of harmful agents in concrete, and causes visual defects in concrete facades, ultimately leading to spalling of areas of concrete.

Carbonation: corrosion of relatively small reinforcement with normal cover depths can go on for a remarkably long time without showing any visual signs, thus making it difficult to diagnose.

Reduction of steel cross section: this affects the structural capability and integrity of the structure.

Although chloride-induced corrosion is considered to be worse than carbonation-induced corrosion, 2/3 of all structural concrete is exposed to environmental conditions that favour carbonation-induced corrosion, as stated by Pakkala (2014).

Durability concerns in concrete structures, often linked to reinforcement corrosion, have raised awareness of the inadequacy of current provisions in construction specifications. The control of quality required in these specifications is traditionally based largely on a measure of compressive strength. In regard to adequate protection of the reinforcing steel, durability is, however, not directly related to strength but dependent on the quality of the concrete cover – its penetrability and thickness (Alexander, 2008).

2.4.3 Reinforced concrete structures subject to chemical attack

Improving the concrete's intrinsic resistance by choosing the correct type of cement, mix design, sacrificial allowance, by applying protective coating and modifying the environment by various means to make it less aggressive to the concrete exposed will protect the concrete structure against chemical attack and these are the only strategies available to mitigate chemical attack (Ballim & Basson, 2001).

The decision to choose the correct strategy required is dependent on the aggressiveness of the environment the concrete is exposed to. The source of the chemical attack arises mainly from water, either as a main corrodent or as the carrier of corrodent in an aqueous solution. The main source of Rand Water chemical attack arises from both. According to Ballim and Basson (2001) the aggressiveness of water is influenced by the following factors:

- **Temperature:** the rate of reaction between water and concrete is strongly influenced by the temperature of the water;
- **pH:** the impact potential water has to dissolve the hardened cement paste. A low pH indicates higher aggressiveness;
- **Softness:** also referred to as calcium hardness, potential of the water to leach out $Ca(OH)_2$ from the hardened cement paste;
- **Turbulence and stagnance:** movement of water causes erosion and continuous removal of deteriorated material, which exposes the fresh surfaces of concrete to attacks;
- **Wet/dry cycling conditions:** this is a phenomenon applicable to structures in tidal zones, therefore not applicable to this study.

To determine the impact aggressiveness of water has on concrete structures, an aggressiveness index was developed by (Basson and Ballim ,2001). The quantification of the aggressiveness, through corrosion indices, then determines the dominant mode of attack, that will in turn provide guidance on how the concrete structures exposed to this environment can be protected. There are only two modes of chemical attack: leaching corrosion; and spalling corrosion. All water containing salts has the potential to either dissolve material, thus weakening the structural strength, or reduce the carrying capacity of the structure through precipitation. To counter the effects of the corrosion, the following measures are proposed in Table 2.1, Table 2.2 and Table 2.3.

Table 2.1:Counter measures against leaching corrosion (Ballim & Basson, 2001)

LCSI	Other applicable factors	Class of concrete	**Coating
Not greater than 300	SCSI not greater than 300	1 (Min binder content = 340 kg/m3) & (Max water-binder ratio = 0.55)	-
	SCSI 300 – 500	2 (Min binder content = 380 kg/m3) & (Max water-binder ratio = 0.50)	-
400 - 700		3 (Min binder content = 420 kg/m3) & (Max water-binder ratio = 0.45)	A
800-1000		3 (Min binder content = 420 kg/m3) & (Max water-binder ratio = 0.45)	B C D E
➢ 1100		3 (Min binder content = 420 kg/m3) & (Max water-binder ratio = 0.45)	B D E
Binder type may be OPC, RHPC, SRPC or Blended OPC with extenders (GGBS, FA, CSF)* ** 50%GGBS, 30% FA, 15% CSF* * Coating refer to Table 2.2*			

Table 2.2:Counter measures against spalling corrosion (Ballim & Basson,2001)

SCSI	Other applicable factors	Class of concrete	**Coating
Not greater than 300	LCSI not greater than 300	1 (Min binder content = 340 kg/m3) & (Max water-binder ratio = 0.55)	-
	SCSI 300 – 500	2 (Min binder content = 380 kg/m3) & (Max water-binder ratio = 0.50)	-
400 - 700	LCSI not greater than 500	3 (Min binder content = 420 kg/m3) & (Max water-binder ratio = 0.45)	-
800-1000	N6 not greater than 800	3 (Min binder content = 420 kg/m3) & (Max water-binder ratio = 0.45)	B C D E
➢ 1100	N6 > 800	3 (Min binder content = 420 kg/m3) & (Max water-binder ratio = 0.45)	B D E

**** Resistance to spalling corrosion may be enhanced**
by using SRPC; 70 % PC +30 % FA; or blends of PC and GGBS with at least 65 % GGBS.

Table 2.3: Coating for concretes in aggressive waters (Ballim &Basson, 2001)

Type	Category	Some examples	Minimum coating thickness	Remarks
A	Inorganic	Extra thickness of base concrete as sacrificial allowance. Plaster coat	20 – 50 mm	Use under mild to moderate leaching conditions only (i.e. LCSI> 500)
B	Preformed polymeric liner	Polyethylene Polyvinyl chloride Polychloroprene	0.2 – 1 mm	Available in light, regular and heavy duty gauges
C	Emulsion-based waterborne organic	Polyacrylic Polyacrylonitrile	150 μm	Tolerant of wet surfaces
D	Solvent-based organic	Chlorinated rubber Vinyl Vinylidene	150 μm	High-build, thixotropic types available
E	Catalyxed organic	Epoxy tar Solventless and water based epoxies Polyurethane	150 – 300 μm	Accurate mixing and time scheduling required

2.5 Rand Water reservoirs design aspects and practices

All Rand Water reservoirs are located in areas of high altitude due to the pumping head that is always required to allow supply. All the reservoirs are buried into the ground. Figure 2.5 is an illustration of the biggest reservoirs Rand Water operates, i.e. Klipriviersberg Reservoirs 1 and 2.

Figure 2.5: Klipriviersberg reservoirs buried, 13.5m into the ground and 20m apart

Figure 2.6: Daleside reservoir without the embankment, wall construction

The bulk of Rand Water reservoirs are buried into the ground, heights generally limited the depth of water to 13.5m because of difficulty in ensuring adequate sealing of joints at greater depths. The sealing of joints, if not installed properly, might result in durability problems. Although the reservoirs are buried, the design of details should be able to cater for any construction defects that are likely to occur.

Large rectangular reservoirs of this depth incorporate a sloping concrete lined basin to reduce the height of the wall. (Figure 2.7) shows the construction of the 100 million litres Rustenburg reservoir indicating the sloped floors from the retaining wall looking at the detail of the floor panels. The construction of the panel can easily result in poor workmanship if the methodology of installation as stipulated in the design drawings and specifications is not understood and followed accurately. As mentioned earlier, poor construction practices might result in durability issues.

Figure 2.7: Construction of Rustenburg reservoir

2.5.1 Design practice

The current design practice is to follow the guidelines and procedures set out in BS8007 in conjunction with BS8110, as a South African code of practice for water retaining structures does not exist at present. Actions in design include self-weight, imposed loads, geotechnical actions, seismic actions, liquid loads and temperature and restrained shrinkage effects. Liquid loads are considered as permanent by BS8007 and SANS10160. Seismic actions are only allowed for in seismically-sensitive areas of South Africa. Some of the old Rand Water reservoirs were designs under the BS8007 predecessor, i.e. BS 5337.

Three main structural elements are considered in the design of a Rand Water reservoirs: walls; floor slabs and foundations; and the roof slab.

2.5.1.1 Walls

The shape and proportions of the structure and the type of construction, determine the load cases and its effects. The most common shapes are circular and rectangular. Wall configuration for both shapes and conventions, are either continuous or jointed vertically with horizontal joints, generally being construction joints and expansion/contraction joints.

The walls of the partially or fully buried reservoirs will encounter soil action on the exterior face of walls. Rectangular reservoir walls tend to be designed as cantilever walls with the top of the wall either free or propped (McLeod, 2013). The base of the wall is generally fixed however, flexible joints can be utilized as well. Ignoring the self-weight of the structure, a cantilever wall (top free) has a bending moment induced about a horizontal axis (max at base of wall) by the liquid pressure, this is at a maximum when the reservoir is full and without soil backfill in the case of buried reservoirs (McLeod, 2013).

An equally critical load case is the reservoir empty with soil pressure on the exterior face of the wall, this load case is also utilised to check for flotation. Any flexural cracking induced in both load cases would be perpendicular to the vertical reinforcement provided to resist the ultimate bending moment (McLeod, 2013).

2.5.1.2 Floor slabs and foundations

The bulk of the Rand Water reservoirs are rectangular therefore the reinforced concrete slabs are cast in square panels, whilst a radial casting pattern is used for circular reservoirs. “Panel casting sequence, joints and reinforcement are used to control the cracking due to the dominant action of serviceability limit state (SLS) restrained deformations due to temperature and moisture effects. Where required, the ultimate limit state (ULS) case of uplift due to groundwater is checked” (McLeod, 2013).

2.5.1.3 Roof slab

Currently, the most common type of construction for the roof slab of a reservoir is a flat slab supported on columns, with heads if necessary to resist punching shear. The roof slab is designed as a flat slab in accordance with SANS 10100 and BS8007. The loads included self-weight, the permanent load for stones, water and imposed loads due to construction and later maintenance inducing bending moments. Serviceability limit state restrained deformations due to temperature and moisture effects may occur, although the slab is usually relatively unrestrained at edges, allowing some movement due to these effects.

The roof also incorporates a drainage layer at the top, if by any means cracking develops due to thermal expansion, deterioration or inadequate mix design, incorrect aggregates, the water will penetrate into the concrete and eventually reach the reinforcement, and the only sign of this moisture effect is black stains seen underneath the roof slabs and the signs of efflorescence.

Figure 2.8: Typical internal configurations for Rand Water reservoir

2.5.1.4 Material properties specified

The design parameters are applied as per BS 8007 requirements. Durable concrete has the required strength and impermeability necessary for the proper functioning of the reservoir. The specified 28-day characteristics cube strength at 35 MPa is adequate, and to ensure adequate protection to reinforcement, the nominal cover is chosen to be 40mm - 50mm against soil.

A minimum cement content of 375 kg/m^3 of cementitious material is the accepted value for good-quality concrete. The maximum cement content, however, is limited to 500kg/m^3 to control thermal shrinkage. The water-cement ratio is also key to ensuring concrete that has the specified strength and impermeability, with a maximum ratio of 0.50 usually specified. To limit shrinkage, fly ash is blended with Portland cement and the maximum water-cement ratio of 0.45 to ensure that the concrete has the specified strength and impermeability. “To limit shrinkage, slagment in cement is generally avoided, as is rapid-hardening cement” (McLeod, 2013).

These material specifications are as per the prescripts of the BS 8007 and it has since been discovered that these prescriptive specifications have durability problems.

2.6 Approaches and specifications for durability and their usefulness

The durability of a material is defined as its ability to withstand environmental deterioration. It is important to investigate the ability of a concrete structure to withstand environmental deterioration in order to enhance the service life of the reinforced concrete structure without having to incur high cost for maintenance or repairs. The quality of the concrete on durability aspect has to be evaluated through a series of concrete durability experiments before the service life model has taken place, rather than relying on the prescriptive approach.

Traditional standards and specifications for concrete have largely been prescriptive and can sometimes prevent attaining the expected performance by requiring limitations, eg. minimum cement content. Specification for durable concrete involves mutual understanding by both the specifier and the end user of what is meant by durability.

2.6.1 Prescriptive approach

Prescriptive specifications involve the selection of material through consideration of factors like the structure's environmental exposure class and service life span. They are "recipe-based specifications", where different limits are set on specific parameters i.e. minimum binder contents, maximum water/binder (w/b) ratio and minimum compressive strength to achieve desired durability for different exposure classes (Ballim et al., 2009).

Codes of practice include the BS 8007, the only code used in South Africa for water retaining structures as a South African equivalent is still in draft and under review i.e SANS 10100-3. A European equivalent, Eurocode 2, is also used in South Africa. As part of her research, McLeod (2013) explored the differences between the crack width models and calculations based on the two codes, and her findings could be used for the crack-width limitations in durability design. Current specifications for concrete durability in both South African Standards and European Standards follow the prescriptive type. However, the increasing argument among various engineers and researchers is that aspects like durability, which is a material performance concept for a structure in a given environment (Ballim et al., 2009), cannot be determined accurately through the simple prescription of mix parameters.

The basis for prescriptive specifications gave the following assurance to a designer, maximum water-binder ratio, minimum fines content and steel protection as stated in (Angelucci, 2013). However; the limits do have economical, technical and environmental implications, hence the swift move away from prescriptive specifications.

2.6.2 Performance-based approach

"Performance-based durability is thus based on durability transport properties instead of limiting particular ingredients, proportions or construction operations without predicting the service life of the structure" (Ballim et al., 2009).

Performance-based specifications seek to ensure adequate performance by taking into consideration exposure conditions and measured material characteristics. Their guiding principle is the relationship between measured durability parameters and the environmental load on the structure, thus enabling the quantification of structural deterioration through the use of appropriate models (Ballim et al., 2009).

The following codes are based on the abovementioned approach:

- EN 206-1;
- DIN 1045-2;
- BS 8500.

2.7 Durability index (DI) Approach performance based specification

"The basis of the DI Approach is to enable the quantification of relevant parameters in order to yield a suitable description of concrete quality. This is obtained through the measurement of transport-related properties of the cover layer. "The measurements characterize concrete quality through the use of durability indices, which are quantifiable parameters that are sensitive to material processing and environmental factors, thus highlighting both material potential and construction quality" (Alexander, 2008).

Durability Index (DI) tests were developed to characterise near surface properties of concrete and to measure its resistance to fluid and ionic transport mechanisms and the resistance to penetrability of cover concrete. (Alexander, 2008).

Ballim et al. (2009) stated that the durability index tests comprise of the three measurable parameters that are associated with transport mechanism within concrete namely:

- Oxygen permeability index (OPI): the principle behind this testing procedure is to measure the decay of oxygen pressure through the concrete sample with time by means of a falling head permeameter (Mackechnie et al., 2001);
- Sorptivity and porosity (water sorptivity index, WSI): the absorption rate and porosity of the concrete sample are determined by immersing the concrete samples in water at regular time intervals and then placing the samples in a vacuum allowing them to saturate for 18 ± 1 hour (Mackechnie et al., 2001);

- Conductivity (chloride conductivity index, CCI): diffusion is a transport mechanism that becomes of crucial importance, especially in marine concrete, and its rate is dependent on a variety of factors such as temperature, saturation level, type of diffusing substance and inherent material diffusion capacity (Mackechnie et al., 2001).

Details of testing procedure are further outlined in the Durability testing manual (Alexander, 2017).

The DI tests can be used for performance-based design and service life predictions. Ballim and Mackechnie established a strong correlation between OPI and carbonation depth, making the test applicable for the prediction of carbonation induced reinforcement corrosion as opposed to chlorine induced corrosion most likely experienced by marine structures (Nganga et al., 2013).

The DI tests have been in development for quite a while, and have undergone improvements through multiple stages of round robin tests, resulting in more refined test procedures (Angelucci, 2013). Although the tests have been in practical use for some time, there is still room for further incremental improvements, particularly regarding test variability and preconditioning of test specimens, so that they can be more confidently used in the construction industry.

2.7.1 DI Approach on existing structures.

Durability studies of concrete structures must ultimately relate to the behaviour of structures in service. There are several reasons for favouring a more analytical laboratory approach: site concrete is exposed to a multitude of influences that are difficult to quantify, design and construction data is often unreliable, and assessing the durability performance of a structure is relatively subjective (Alexander et al, 1996). Despite these practical limitations useful information can be obtained about the long-term behaviour of concrete under real service conditions. This information is essential for confirming trends established from early-age laboratory and field exposure studies. Case studies of concrete structures should therefore be regarded as the ultimate benchmark for concrete durability studies.

The Durability Index tests performed on mature structures can assist in describing the potential durability of the concrete and form the basis for future design parameters, construction methods and specifications, and inform repair strategies in the assessment and control of the quality of cover concrete. (Lampacher, 2000) recommended that in the assessment of mature structures, early age index tests should be conducted and followed by periodic direct performance monitoring and measurement of the extent of deterioration, using the same tests. A comparison of these tests over time, can provide an accurate assessment of the performance of the structure throughout its service life.

In a study conducted in the Kwazulu-Natal area, concrete structures built with Grade 35 concrete and made with high quality aggregates were found to have low values of permeability and sorptivity and the study concluded that such concretes provide satisfactory performance for water retaining structures, provided good construction practice is ensured.

Rand Water reservoirs are designed and maintained as water retaining structures and the use of assessing durability via the DI approach can provide an indication of the performance of the reservoirs, while they are in service. The results can also assist in crafting a specification that focuses on the following as per (Alexander et al, 2000) recommendation:

- High quality aggregates, that will allow dense packing of the concrete
- Cement binders,as concrete made with binders will continue to improve due to the longterm pozzolanic and cementing reactions
- Curing methods: as poor curing methods were considered detrimental to durability and increasing the risk of reinforcement corrosion. Active wet curing was recommended for promoting permeability concrete in the near surface regions.

2.8 Review of SANRAL current durability performance specification and highlights of pros and cons in South African context

Recently the performance-based specification was implemented in South Africa. This section is based on a case study by Nganga et al. (2013). The objective of the study reported in this paper was to validate the practicality of the DI-based performance approach. The South African National Roads Agency Limited (SANRAL) has recently implemented DI-based performance specifications in a major infrastructure improvement project, the so-called Gauteng Freeway Improvement Project (GFIP). SANRAL is the first state-owned entity in South Africa to use the performance-based systems of specifications.

The control of quality was carried out on reinforced concrete structures classified as ‘Class W’ which means that such structures should be tested to meet certain durability criteria (using DI tests and cover depth measurements), in addition to strength. The structures included precast median barriers, and structural elements on interchanges, such as bridge piers, decks and abutments as stated in Nganga et al. (2013).

The GFIP is located in a high altitude, summer rainfall inland region, and thus their prediction model was based on carbonation-induced corrosion. Figure 2.9 illustrates the prediction model that was used for SANRAL's development of the durability specifications for the GFIP.

The input parameters used in the model considered the following:

- Limit OPI value of 9.70,
- Inland exposure conditions with relative humidity of 60%,
- Binder type of 100% CEM I.
- For a service life of 100 years,
- Limit OPI value of 9.70, the carbonation depth was predicted to be 28 mm, and thus a limit value on cover depth of 40 mm was sufficient.

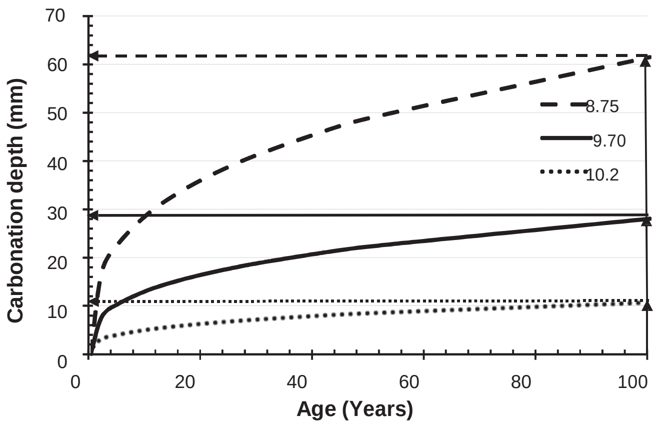


Figure 2.9: OPI prediction model for the GFIP. The values 8.75, 9.70 and 10.2 are the negative log of the measured O_2 permeability at 28-days (Nganga et al, 2013)

Considering the fact that Rand water structures are also in the same environment as the GIPF structures, the same carbonation model can be adopted for the Rand Water reservoirs, with few variations of the blender as the bulk of the structures are designed as water retaining structures therefore the use of CEM1 alone would not be beneficial due to its high rate of heat of hydration. The use of blenders will then reduce the clinker content of the cementitious binder. In addition to changing concrete materials and mix designs to reduce the portland cement clinker component of concrete, making durable concrete structures has a larger impact on improving sustainability since times to rehabilitate and replacement i.e service life can be extended**.**

According to Hooton (2013) it is the unpredictable areas of poorly compacted, poorly cured or cracked concrete with less than the design depth of cover which will severely shorten the predicted time to corrosion, regardless of what model is used. Therefore, the most effective way to obtain the model predicted service life of a structure is to address the site conditions prior to, or during, construction.

The Gauteng Freeway Improvement Project represents the first large scale implementation of the DI-based performance approach in South Africa in which site–specific samples were obtained and tested. Nganga et al. (2013) found out that during their review of the data, it was evident that the approach can be practically applied on site, and valid test results that comply with limit values in specifications can be obtained. In a previous study on a similar project in the Western Cape Province in South Africa, Beushausen et al. (2013) reported on the effectiveness of the implementation of DI-based performance specifications on the production quality of precast median barriers. The implementation of the DI-based specifications had led to improvements in construction practices, which was interpreted as a step forward and encouraging.

The two studies conducted on SANRAL projects both concluded that the durability of reinforced structures can best be guaranteed when the 'as-built' quality of the cover layer is verified. It was also concluded that the DI-based performance approach was a practical approach to control the quality of structures for durability however there were difficulties that were identified with the implementation on a large scale and these difficulties will serve as a vital lesson for improvement to ensure practical application.

The implementation of the DI-based performance approach on site was also regarded as an important first step towards the construction of more durable reinforced structures in South Africa.

2.9 The need for research in durability performance on water reservoirs

Reinforced concrete water-retaining structures are commonly used in South Africa for the purposes of storage. The bulk of Rand Water reservoir designs and construction methodologies are based on prescriptive specifications. The structures are all designed as per the prescriptions of the BS 8007 as a South African code pertaining to the design of concrete water-retaining structures does not currently exist. This performance- based specification approach has proved invaluable in determining durability requirements for tailor-made concrete, since prescriptive-based approaches alone have been found to be inadequate. The same deterioration modes of the concrete structures have been assessed over time, affecting all the different types of reservoirs within the system. This has necessitated the need for new ways in which reservoirs have to be designed and specified through the cover layer and the cementitious materials.

“The main factors controlling the surface zone of concrete are a) its deterioration rate, namely the constituent materials, b) construction quality and c) the aggressiveness of the exposure environment” (Ballim et al., 2009). “What is therefore required is the ability to quantify the quality of the cover layer in terms of durability parameters that can immediately be useful to designers; these quantifiable parameters can then form the basis for crafting performance-based specifications that should help improve overall quality of reinforced concrete structures” (Alexander et al., 2008). Since it is difficult to control the quality of the surface zone, alternative strategies for durability design should ideally focus on the constituent materials and construction practices.

Currently Rand Water has no concrete durability specifications dedicated for its water-retaining structures therefore the study addresses, and investigates, the need for a performance-based design approach and specification, for various durability properties. Performance-based design approaches are becoming more popular due to the influx of new cementitious materials. With an increased rate of deterioration in reinforced concrete structures due to durability performance, efforts are being made to quantify in-situ performance. The study will serve as a basis to promote a deeper understanding of concrete durability, performance-based specification, as well as the initiation and/or implementation of the DI Approach within the Rand Water system.

Reinforced concrete water retaining structures are commonly either rectangular or circular in plan. Approximately 80 % of Rand Water reservoirs are therefore rectangular ranging from 20 to 640 million litres (M*l*). "Depending on the geology and geographical location in South Africa, and the preference of the client, reservoirs are often buried partially or fully. Under these circumstances, the reservoir is constructed on excavated ground, with a fill embankment constructed after the completion of the reservoir structure. Sizes vary according to the required storage capacity” (McLeod, 2013).

2.10 The benefits of a performance specification to Rand Water reservoirs

Rand Water is currently building one of its largest water treatment plants, inclusive of water tanks, industrial and commercial offices and other water retaining structures, over the course of five years. The current project is built as per each contractor’s individual preferences with regard to the mix design. The very few parameters stated on the project were:

- service life of 65 years;
- compressive strength of 35 MPa;

- the use of blenders such as fly ash limited to 30%;
- no durability parameters were specified as to be monitored;
- no quality control parameters to achieve durability except what the contractors would provide;
- the prescriptive requirements as per BS 8007 requirements.

Considering the case studies by SANRAL, it is evident that the use of performance-based approach specifications is the way to go, with regard to reinforced concrete structures. One of the ways that can help ensure concrete durability is to use appropriate performance specifications, since durability design includes more than the selection of concrete materials and mix proportions. As traditional standards and specifications for concrete have largely been prescriptive and can sometimes prevent attaining the expected performance of concretes by requiring limitations such as minimum cement contents, there is a current trend away from prescriptive towards performance-based specifications around the world, as SANRAL has done.

The philosophy behind the specification of concrete is that if it is known exactly what concrete is required for a specific purpose and exactly how to make it, the tendency is to be able to reproduce it as many times as required (Angelucci, 2013).

2.11 Chapter summary

2.11.1 Key factors in durability assessment

The factors which are most likely to dominate in any given durability assessment are the water binder ratio, binder content, curing and control/monitoring of temperature variations of concrete, and the permeability and penetrability of the cover layer.

2.11.2 Deterioration mechanism and their implications

Throughout the literature review, different models have presented carbonation as a main deterioration mechanism due to Gauteng’s environment, which ultimately leads to corrosion of reinforcement. Detailed assessments conducted on Rand Water reservoirs, indicate that corrosion is the main form of deterioration mechanism experienced within the network of water reservoirs. All the defects noted lead or led to corrosion if no maintenance or remedial measures were implemented.

Carbonation can ultimately lead to the loss of passivation of the steel, and according to Ballim et al (2009), the process requires some time for the carbonation front to penetrate through the cover of the concrete and render the reinforcing steel liable to corrosion, and the rate of corrosion gets determined by the types and concentrations of the ions that reach the steel, as well as the concentration of oxygen that diffuses to the steel surface.

Corrosion of reinforcement is regarded as the most serious durability problem in construction engineering. How to eliminate the degree of steel corrosion is a problem in practical engineering and researchers all over the globe, are building models in order to better the quality of the cover layer. The deleterious ions penetrate and infiltrate the concrete through a porous and inadequate cover layer, thereby causing corrosion of the reinforcement. Improving the quality of concrete has been considered as a primary protection method for corrosion. Protection methods such as reinforcement coatings, corrosion inhibitors and adequate specifications are critical for the layer cover.

2.11.3 Importance of durability design and improvements to future design codes of practice

The basis of design is concerned with the methods that establish the requirements for safety, durability and serviceability of the structures (Barnardo et al, 2014). Durability design includes more than the selection of concrete materials and mix proportions.

As mentioned before, the cause of premature deterioration in reinforced concrete structures can be linked to inadequate specification for achieving durable concrete at the design stage, poor workmanship at construction stage and a lack of knowledge and application of maintenance management strategies by owners of the structures. The design and construction aspects can be achieved by improving the codes of practice to include durability requirements.

The current code of practice for designing WRS in South Africa is based on BS8007 however SANS 10100-3 (draft) has been developed recently, to address the South African environment. These codes of practice are prescriptive codes that can be improved to incorporate durability requirements to achieve optimal performance of the structures. The codes of practice currently stipulate the methodology for specifying durable concrete and outlines some of the performance requirement for the WRS such as water-tightness, minimum thermal and shrinkage cracking limits and acceptable crack widths, minimum strength requirements, cover depth requirements, design life of structure and individual elements and the serviceability requirements.

The future design codes of practice can further stipulate the following as recommended by (Richardson, 2002), (Hooton, 2013) and (Barnardo et al, 2014):

- Deterioration mechanism is key when considering the material specification.
- Inclusion of early - age properties of concrete and reinforcing steel. Properties such as heat of hydration, strength and stiffness evolution, bleeding, capillary pressure, plastic and settlement shrinkage may dominate early age behavior of concrete, which in turn may determine the eventual crack width patterns in the hardened concrete of water retaining structures as recommended by (Barnardo et al, 2014).
- Quality control and tests methodology and their frequency.
- Quantifiable description of the criteria that defines serviceability failure and the level of risk acceptable, permissible extent of maintenance.
- Future infrastructure should represent sustainable development; get the balance right between optimum use of materials and the costly risk of failure during a defined service life.
- The mathematical models of degradation are required which can then be used for probabilistic analysis of durability and life cycle costing.

Improvements in construction practices can include the following as stated by (Hooton, 2013):

- Construction detailing;
- Temperature control;
- Adequate compaction specifications;
- Protection of fresh concrete in the specifications, and that testing be conducted to ensure that the specifications are being followed, and therefore performance has to be stated clearly.
- Curing methods: as poor curing methods are considered detrimental to durability and increasing the risk of reinforcement corrosion.

2.11.4 The importance of performance-based specifications

"Durability aspects are purely a material performance concept for a given structure in a given environment and cannot be determined accurately through the prescription of mix parameters. Instead the relationship between mix design parameter and properties, durability properties need to be thoroughly investigated and clearly understood" (Ballim et al, 2009).

Prescriptive codes and standards are very conservative and solely based on strength requirements. Depending on the intended application, concrete composed from the code requirements may result in unnecessary expensive, and inefficient solution, asserts Angelucci (2013). They deal with structural design and use of concrete, and specify prescriptive criteria for durability and concrete structures. They also tend to pose requirements on constituents and some of the modern codes, such as Eurocode 2, deal with construction practice and structural details on the basis of exposure to environment, expected in-service life and intended service life condition of structure.

In order to achieve sustainable, economical and durable concrete, performance objectives should be identified. “The performance-based specifications address the quality of concrete evaluated through a series of concrete durability experiments before the service life model, and thus are based on durability transport properties instead of limiting particular ingredients, proportions or construction operations” (Alexander, 2008).

“*The DI tests have been developed over a long period of time and have undergone improvements through multiple stages of round robin tests, resulting in more refined test procedures. Although the tests have been in practical use for some time, there is still room for further incremental improvements, particularly regarding test variability and preconditioning of test specimens, so that they can be more confidently used in the construction industry*” (Mukadam, 2016).

To ensure durability, the following then becomes critical; all key points as stated by Richardson (2002):

- Deterioration mechanism and the mathematical models of degradation required for probabilistic analysis of durability and life cycle costing;
- Methodology for specifying durable concrete, future infrastructure should represent sustainable development, get the balance right between optimum use of materials and that can be achieved through performance specifications since durability is more than materials and their proportioning;
- Costly risk of failure during a defined service life.

CHAPTER 3 - METHODOLOGY

3.1 Introduction

Durability can be assessed by conducting various tests methods and assessments that allow for the deterioration processes likely to affect the structure to be studied, understood and possibly quantified. The development of a suitable test methodology for assessing the durability performance of concrete structures i.e. brief descriptions of each structure, test positions and detailed description of their environment related to concrete durability is outlined in this chapter. The assessment was undertaken in two parts: preliminary assessments; and detailed assessments. The preliminary assessments were used to characterise the nature of the deterioration and the possible mechanism, and gives guidance in planning for the detailed assessments, based on visual inspections (Otieno, 2010). The detailed assessments were then used to confirm the cause and quantify the severity of the deterioration mechanism (Otieno, 2010).

3.2 Selection of reservoirs

Rand Water's water reservoirs are primarily concrete structures typically consisting of: RC columns, roof and ring beams; reinforced or mass or post-tensioned concrete walls; and mass concrete floor slabs (Figure 3.1 and Figure 3.2). The life of a reservoir depends on the durability of its components. For a correctly designed concrete water reservoir and good-quality materials and workmanship, the design life of the concrete water reservoir should be 60 years (BS 8007); however, with proper maintenance the actual life can be greatly increased to 80 years and beyond.

Selected Rand Water reinforced concrete reservoirs across the Palmiet system of the bulk water distribution network were assessed in this study. The criteria for selecting reservoirs to be included in this study are discussed in the following sections.

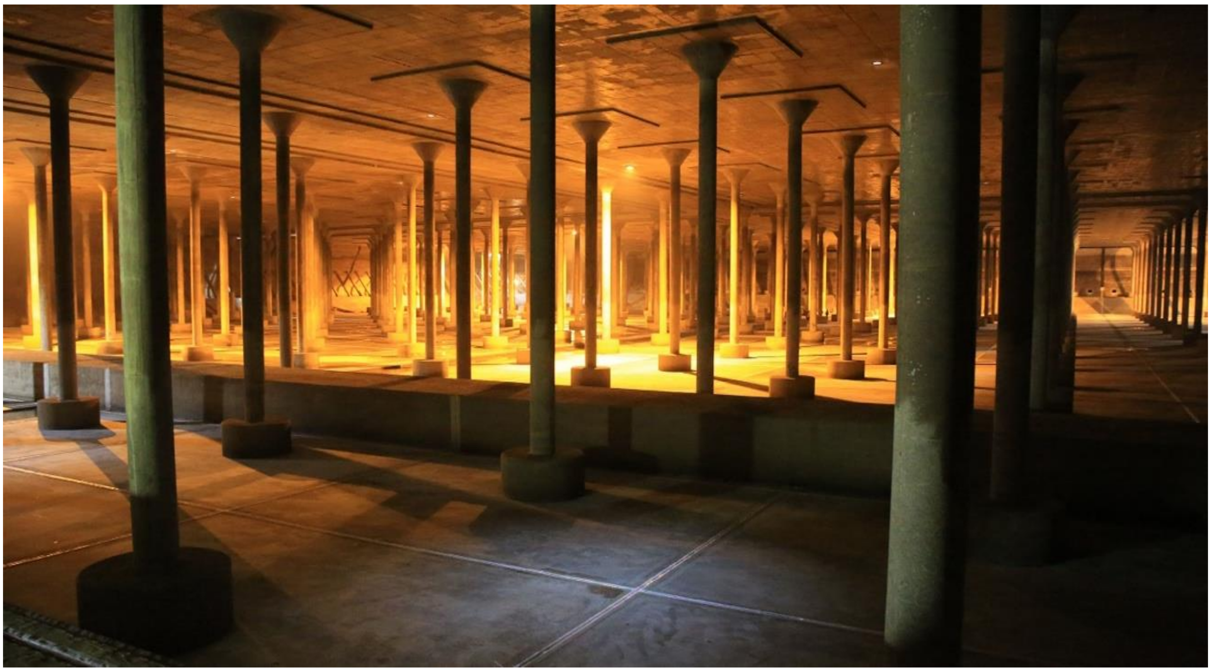


Figure 3.1: Typical interior of a Rand Water reservoir

Figure 3.2: Typical interior of a Rand water reservoir

3.3 Location

The selection of the reservoirs to be assessed was complex as most of the reservoirs were in operation. This limited the scope of the research. The only time that inspections were possible was when maintenance was being carried out on a particular reservoir. All the reservoirs were located in Gauteng, between Johannesburg, Pretoria and the Vaal region as the bulk of the operations were based closer to the Vaal River and the Vaal Dam.

Gauteng is the most populous province in South Africa, with a population of over 15 Million (StatsSA, 2019). The bulk of the Rand Water infrastructure is responsible for the distribution of water to all the citizens within this populous province. It is also the economic hub of South Africa, characterised by widespread use of concrete as a construction material as Lampacher (2000) noted.

3.4 Age, condition and performance

The lifespan of the test structures was considered critical to highlight the deterioration mechanism and the specifications prescribed for the different structures within the Rand Water system. The selected test structures were constructed during the period from the 1930s to the 1990s. This was to assess the different specification methodologies and material evolution of each time.

3.5 Environmental influences

Rand Water reservoirs are located in a high altitude, summer rainfall inland region with over 60% relative humidity, prone to carbonation-induced corrosion. Figure 3.3 indicates the typical environmental exposure for Rand Water reservoirs. The entire roof and a metre (1 m) of the wall are completely exposed to environmental influences and the remainder of the wall height is buried onto the ground. The roof and part of the wall are exposed to the following environmental influences:

- Temperature
- Precipitation
- Relative humidity
- Pollution

Lamphacher (2000) reported that the internal humidity of thick concrete elements that are exposed to moisture effects under service is likely to remain very high, and eventually promote other forms of deterioration such as Alkali Silica Reaction (ASR), regardless of the size of the element. The rate of deterioration is then a function of the degree of moisture exposure, and concrete elements directly exposed to water can deteriorate more rapidly than those elements exposed to atmospheric conditions.

Scott (1997) stated that durability is significantly affected by the environment to which a structure is exposed and its environmental classification allows for the determination of the dominant deterioration mechanisms, and thus allows for the design of the structure to resist that mechanism of deterioration.

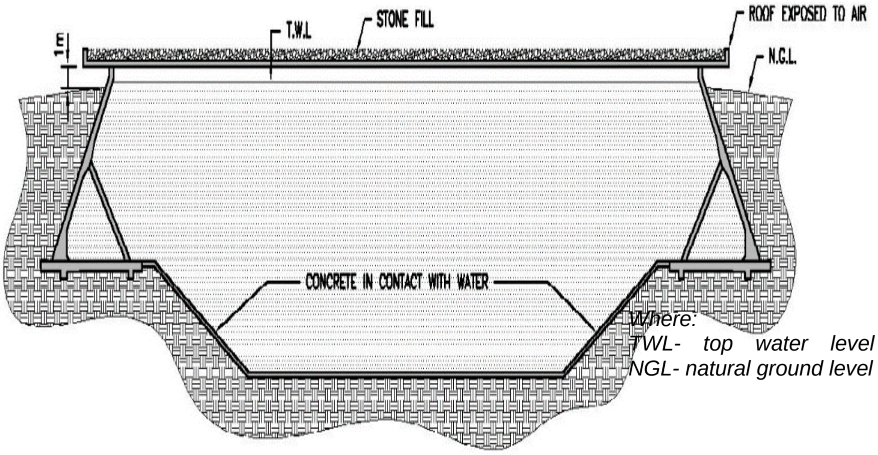


Figure 3.3: Typical representation of the structure type and exposure conditions for the reservoirs

3.6 Source of raw water

The selected reservoirs receive raw natural water for treatment from the Vaal Dam. Forest Hill reservoir is supplied by both Zuikerbosch water treatment plant and Vereeniging Water treatment plant. Brakfontein reservoir is supplied by Zuikerbosch water treatment plant, while Sasolburg reservoir is supplied by Vereeniging Water treatment plan.

3.7 Identified reservoirs

The following three reinforced concrete reservoirs were selected for assessment in this study.

3.7.1 Forest Hill reservoir

The reservoir is situated in Forest Hill in Johannesburg in the Gauteng Province (Figure 3.4). Forest Hill No. 2 reservoir forms part of cluster of three reservoirs situated in Forest Hill. The site is accessible via the access controlled gate in Towerby Street.

Figure 3.4: Forest Hill reservoir

3.7.2 Brakfontein reservoir

The reservoir is situated on Portion 12 of the farm Brakfontein 390 JR, in the Gauteng Province some eight km south of Centurion (Figure 3.5). It is situated in an industrial area with access from Olievenhoutbosch Avenue, north of Johannesburg in the Gauteng Province.

Figure 3.5: Brakfontein reservoir

3.7.3 Sasolburg reservoir

The reservoir is situated on Erf 8019 Sasolburg Ext 28, Dorp, in the Vaal, South of Johannesburg in the Gauteng Province (Figure 3.6).

Figure 3.6: Sasolburg reservoir

3.8 Detailed description of reservoir test structures

Table 3.1: Selected reservoirs and salient facts

Description	Reservoir		
	Brakfontein	**Forest Hill**	**Sasolburg**
Dimensions (m)	192 long x 117m wide, 8.84m wall height	Diameter of 115m , 9.45m wall height	105 long x 105m wide; 9.1m wall height
Capacity (Ml)	423	90	103
Age (Years)	24 (Commissioned in March 1996)	84 (Commissioned in July 1936)	49 (Commissioned in September 1971)
Condition	Poor: This implies that major deterioration mechanisms noticeable, loss of cross sectional area of steel i.e. spalling and corrosion propagated and significant erosion noticeable on earth structures. Structural integrity and design functionality is still maintained, serviceability requirements are compromised. Refurbishment and remedial work is required.	Poor: This implies that major deterioration mechanisms noticeable, loss of cross sectional area of steel i.e. spalling and corrosion propagated, and significant erosion noticeable on earth structures. Structural integrity and design functionality is still maintained, serviceability requirements are compromised. Refurbishment and remedial work is required.	Poor: This implies that major deterioration mechanisms noticeable, loss of cross sectional area of steel i.e. spalling and corrosion propagated, and significant erosion noticeable on earth structures. Structural integrity and design functionality is still maintained, serviceability requirements are compromised. Refurbishment and remedial work is required
Structural System	Reinforced concrete floors, walls, flat roof supported by circular RC columns. The reservoir walls consist of 305 mm (RC) retaining walls inclined at 60 degrees supported by 305mm x 305 mm RC struts at 2,265m spacing i.e. (Folded plate RW design) and a gravity wall.	Circular gravity concrete wall with supporting earth embankment. Reinforced concrete floors, walls, flat roof supported by circular RC columns. The reservoir roof comprises of 190 mm thick reinforced concrete slabs, supported by circular columns. All columns have a conical head and base. At the column heads the roof slabs have been thickened to approximately 280 mm.	Reinforced concrete floors, walls, flat roof supported by circular RC columns. The reservoir walls consist of 305 mm (RC) retaining walls inclined at 60 degrees supported by 305mm x 305 mm RC struts at 2,265m spacing i.e. (Folded plate RW design).
Buried/Above ground	Below ground, 1m wall height exposed to air and 4m embankment at toe.	Below ground, 1m wall height exposed to air and 4m embankment at toe.	Below ground, 1m wall height exposed to air and 12m embankment at toe.
Design and Construction Standards/Specification	BS8007 & SANS 1200	Unknown	Unknown

3.9 Preliminary assessments: Site investigations and visual assessments

The objective of this section is to record the state of deterioration of concrete throughout the chosen and identified reservoirs. Visual inspections are usually the first step in any investigation. The aim of the investigation is to give a first indication of the nature and extent of deterioration damage. Otieno (2010) states that visual inspections have to be carried out in a systematic manner whereby proof of deterioration is gathered in an objective manner, following clear guidelines that define damage in terms of appearance, vicinity and cause.

3.10 Detailed assessments: Selected test methods

The site performance of the existing structures looks into the different test methods available. The laboratory experiments included the assessment of compressive strength, durability performance indicators (Durability indices), hardness testing, corrosion assessment, carbonation depth, cover depth assessments and aggressivity analysis (Basson indices).

Diagnostic testing of structures plays a critical part in any investigation, therefore the need to plan such activities and why they are needed is very important. The aim of the tests is to identify the core causes of the defects in the structure. The information gathered from the testing should then provide the extent of the deterioration. Lampacher (2000) classified test methods into the following categories, which were found to be applicable to this study as well.

"*Non-destructive and partially destructive methods: are usually defined as those methods not impairing the intended performance of the structural element under test, and are taken to include those methods which cause localized surface zone damage. All non-destructive and partially destructive tests can be carried out directly on the in-situ concrete without the removal of a sample*" (Lampacher, 2000).

"*Methods requiring sample extraction: These commonly take the form of tests on drilled cores, which may be used in the laboratory for a range of different tests. In this case, as with partially-destructive methods, making sure that the cored-out area is repaired is necessary*" (Lampacher, 2000).

"*The nature of currently available testing equipment ranges from economical, simple devices to complex, costly apparatus, which sometimes requires extensive preparation or safety precautions. Some test methods provide a direct quantitative measure of the desired durability property and correlations are often necessary. Practical limitations, accuracy, and reliability vary widely and must be considered carefully before implementing a test programme*" (Lampacher, 2000).

Table 3.2 categorises the principal test methods used on this study, with the reasons for their choice provided in the subsequent section. The parameters listed were considered relevant for this study based on environmental exposure of the selected test structures, cost and time limitations.

Table 3.2: Selected reservoirs and properties to be measured

Property to investigate	Test	Type of testing
Assessment of surface hardness	Schmidt hammer test	In situ testing Laboratory testing
Assessment of embedded reinforcement corrosion	Visual assessment	Laboratory testing
Extent of deterioration reactions i.e. assessment of cover depth, carbonation depth	Carbonation depth measurement Cover depth measurement	In-situ testing and Laboratory testing
Penetrability of concrete Durability : DI Approach	Gas Permeability Water Sorptivity Chloride Conductivity	Laboratory testing
Strength	Compressive Strength test	Laboratory testing
Water Aggressivity	Basson indices	Laboratory testing

3.11 Core extraction on reservoirs

Concrete cores can provide useful information in durability assessment of mature structures.

Concrete cores were taken from the roof and wall sections of each reservoir (Figure 3.7). 68 ± 2 mm diameter cores were taken for durability tests while 100 mm diameter cores were taken for compressive strength measurement in line with SANS 5865 standard. The location of cores was marked on existing drawings of the reservoirs and final positions confirmed on site. The positions were determined such that the least impairment is caused to the structure.

The following was considered for core extraction:

- Cores were taken where signs of defects were present,
- Design considerations:
  - position, size and location of reinforcing bars, care was also taken to try and ensure that the cores were taken in areas where there was no reinforcement. Only in areas where there was excessive spalling were cores extracted through the reinforcement.
  - Cover depth considerations
- Exposure to environmental influences: all reservoirs are buried onto the ground therefore only one metre (1m) of the wall and the roof are exposed to the environment.

Figure 3.7: Typical 100mm diameter core out of test reservoir

3.12 Implementation of test methodology

3.12.1 Assessment of compressive strength

The compressive strengths were obtained by performing crushing tests on cores extracted from the test structures (100 mm diameter x 100 mm height). The specimens' compressive strengths were tested in accordance to SANS 5865 (1994). All the cores were soaked in water, at 23 ± 2 °C, for at least 48 hours prior to testing. All the tests were performed using an automatic hydraulic compression machine at a loading rate of 0.30 MPa/s as per SANS 5865 guidelines. The following were measured and calculated as a result of the test:

- Measured strength: represented the applied load per unit surface area (of load application) of concrete;
- Equivalent cube strength: represented the strength that takes the reinforcement factor and the diameter/length ratio of the core into account;
- Potential cube strength: represented the expected in-situ cube strength that can be attained if adequate compaction is carried out during concrete placement. It took into account the excess voids factor. The determination of excess voids is based on visual inspection of the core specimen and comparing this to Figure 1 of SANS 5865.

3.12.2 Assessment of steel corrosion

A visual assessment was conducted on the corroded specimens. Additional cores were extracted from Forest Hill reservoir, in areas where corrosion of reinforcement was evident during the visual assessment of the reservoir.

The design diameter of the reinforcing bar was compared to the measured diameter of the corroded specimen and the difference was calculated as the percentage loss of area of steel.

3.12.3 Assessment of cover depth to steel reinforcement

Cover to reinforcement is the shortest distance between the surface of a concrete member and the nearest surface of the reinforcing steel (Neville, 1999). The durability performance is generally the degree of success of the concrete’s response to the aggressiveness of the exposure environment and the response is governed by the properties of the concrete in the cover layer and the depth of concrete cover (Ronne, 2005). Cover to reinforcement is a critical parameter to ensure adequate durability of reinforced concrete structures.

The most common method of measurement of cover thickness is the use of non-destructive cover meters, Figure 3.8. There are no generally accepted methods to define either a representative sample for cover measurements or how a cover survey should be carried out (Ronne, 2005). Cover surveys were conducted on specified locations on the roof and wall sections, to check for compliance with the specification. A cover meter was used to measure the depth of cover to reinforcement and to identify the location of reinforcing steel. A minimum of 15 - 30 cover measurements were taken along the roof and wall sections of the reservoirs.

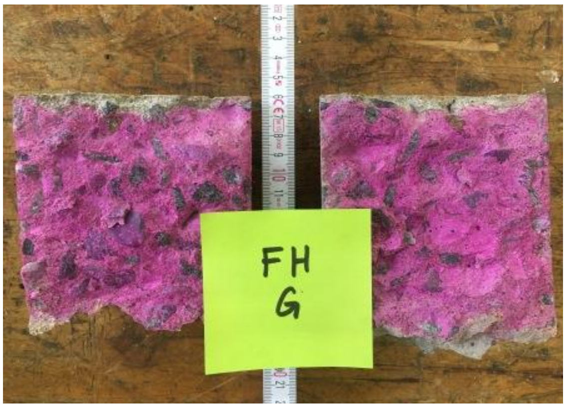
Figure 3.8: In-situ cover depth survey using a cover meter

3.12.4 Assessment of surface hardness

Surface hardness testing of concrete is a long established NDT method used to test the uniformity of concrete, measure the strength of the existing surface concrete and detect weak spots (Chandramouli et al, 2009). Surface hardness testing of concrete applies the dynamic rebound surface hardness testing devices like the Schmidt rebound hammer. During dynamic hardness measurements, the inelastic properties of concrete may be as important as the elastic properties of concrete due to the softening fashion of material response (Szilagyi et al, 2011). The rebound hammers also provide hardness information connected to both the elastic and inelastic properties of the surface layer of concrete that cannot always be related directly to the compressive strength (Szilagyi et al, 2011). A surface hardness test was performed using the Schmidt hammer. A total of three readings, per specimen, of the rebound number were taken. The assessment was not conducted in-situ but on the cores extracted from the reservoirs.

3.12.5 Assessment of carbonation depth

The most common method used to assess carbonation involves spraying freshly broken concrete surfaces with 1% or 2% phenolphthalein solution. The surface where the pH >9 turns purple and gradually lightening shades of pink appear for pH8 to 9 and represents the depth to which full or partial carbonation has taken place. A pH<8 represents a colourless surface which denotes the absence of carbonation. All cores taken from the test structures were tested with phenolphthalein solution, and carbonation depth was measured as shown in Figure 3.9. The cores were oven-dried before being split into two and sprayed with phenolphthalein indicator.



(a)

(b)

(c)

Figure 3.9: Carbonation depth measurement: a) Forest Hill reservoir b) Sasolburg reservoir c) Brakfontein reservoir

3.12.6 Assessment of concrete penetrability using durability index approach

There are three Durability Index (DI) test methods in South Africa. They comprise of oxygen permeability index (OPI), chloride conductivity index (CCI) and water sorptivity index (WSI).

“The tests were created to characterize near surface properties of concrete and to measure its resistance to fluid and ionic transport mechanisms, such as permeation, diffusion, absorption and migration” (Alexander, 2008). In spite of the fact that the values obtained from these tests may not totally represent the fundamental material characteristics, the results can be utilised to provide indices that are likely to predict material performance (Otieno, 2008). The acceptance criterion is shown in Table 3.3. These were developed to be used in fresh concrete as well as on mature structures.

Details of testing procedure are further outlined in the Durability testing manual (Alexander, 2017).

Table 3.3: Acceptance limits for durability indices (Alexander et al., 2017)

Acceptance criterion		**OPI (log scale)**	**Water Sorptivity (mm/ √ hr))**	**Conductivity (mS/cm)**
Laboratory concrete		10	< 6	< 0.75
As-built structures	Full acceptance	9.4	< 9	< 1.00
	Conditional acceptance	9.0 to 9.4	9 to 12	1.00 to 1.50
	Remedial measures	8.75 to 9.0	12 to 15	1.50 to 2.50
	Rejection	< 8.75	15	2.50

The tests were performed on 68 ± 2 mm diameter cores extracted from the test structures, at designated locations. The core was cut into 30 ± 2 mm thick discs that were then used for the tests. The tests are further outlined in the Durability Index Testing Manual (Alexander et al., 2017). A typical core, and discs are shown in Figure 3.10.

Figure 3.10: Typical 68 mm diameter core with 30 mm thick disc taken from a test structure, for DI testing at Wits concrete lab

3.12.6.1 Oxygen permeability index test

The Oxygen Permeability Index test method measures the pressure decay of oxygen passed through a 30 ± 2 mm thick slice of 68 ± 2 mm diameter core of concrete. The rate of pressure decay of a falling head permeameter is governed by the Darcy equation, allowing the coefficient of permeability to be determined (Ballim, 1991). The falling head permeameter (Figure 3.11), developed at the University of Witwatersrand, applies an initial pressure to one side of a concrete specimen with the other side at normal atmospheric pressure (Otieno, 2008).

The concrete disc specimens were placed in an oven at 50^{o}C for seven days. The oven chamber temperature and duration were selected to result in a minimum degree of microstructural alteration of the concrete disc specimens, while still maintaining the internal moisture equilibrium (Ikotun, 2017). The specimens were then cooled in a desiccator for three hours before being loaded onto the compressible collars and eventually to the permeability loading rigs as shown in Figure 3.11 and Figure 3.12.

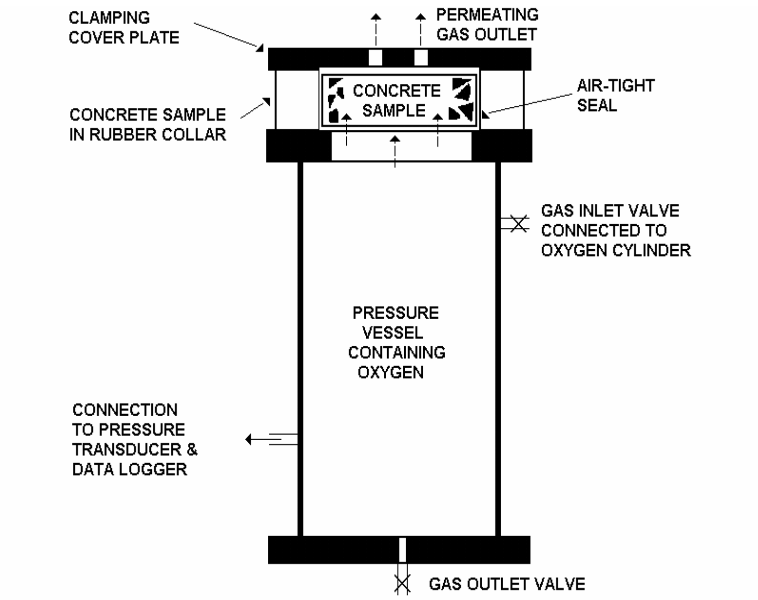


Figure 3.11: Schematic diagram of OPI apparatus (Alexander et al., 2017)

As permeation occurs, the decrease in pressure with time was measured. From the slope of the log of pressure head versus time, the oxygen permeability coefficient (**k**) was determined as shown in (equation 3.1) (Alexander et al.., 2017):

$$\boldsymbol{k} = \frac{\omega V g d}{R A \theta t} \ln \frac{P_o}{P} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

where:

k = coefficient of permeability (m/s)
ω = molecular mass of permeating gas (kg/mol)
V = volume of the pressure cylinder (m^3)
g = acceleration due to gravity (m/s^2)
d = sample thickness (m)
R = universal gas constant (Nm/Kmol)
A = cross sectional area of specimen (m^2)
θ = absolute temperature (K)
P_o, P = pressure at start of test and at time t (sec) respectively (kPa)

The oxygen permeability index is then the negative log of k: $OPI = -\log_{10} k$. A higher value of the OPI is indicative of a greater impermeability, indicating a concrete of high quality. The values range between 8.5 and 10.5.

Figure 3.12: OPI apparatus and test underway at Wits University Concrete Lab

3.12.6.2 Water sorptivity index test

The Water Sorptivity Index measures the rate of movement of water through the concrete under capillary action. It is a measure of the porosity of the concrete. The concrete disc samples were placed in an oven at 50°C for seven days (to ensure uniform moisture content). The sides of the specimens are sealed using epoxy (to ensure unidirectional movement, and placed in a few millimetres (mm) depth of water, and weighed at regular intervals to determine the mass of water absorbed (Otieno, 2008). The initial weight (M_o), i.e. the oven dried concrete was weighed to the nearest 0.01g, and partially immersed in a solution of distilled water saturated with $Ca(OH)_2$ with one face exposed to the solution as shown in Figure 3.13.

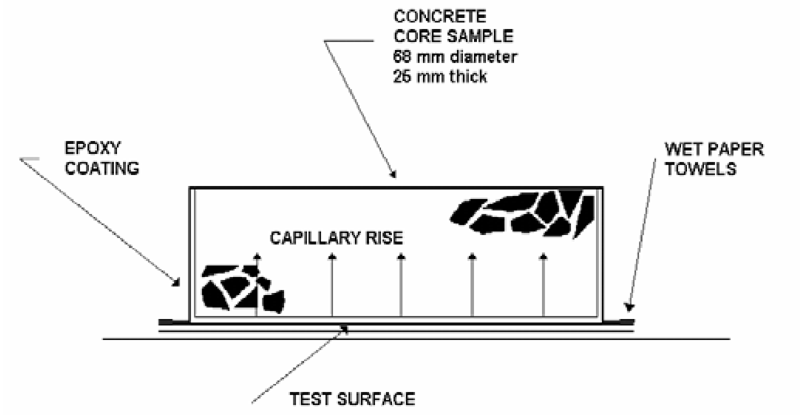


Figure 3.13: Schematic diagram of water sorptivity apparatus (Alexander et al, 2017)

The weight of the partially immersed concrete disc (M_{sat}) was measured at 3, 5, 7, 9, 12, 16, 20 and 25 minutes time intervals. The samples were then vacuum-saturated at pressures between -75kPa and -80 kPa for three hours. The chamber was then filled with the $Ca(OH)_2$ solution to a level of 40 mm above the top of the specimens. The vacuuming was repeated again at the same pressures for a further one hour ± 15 minutes. The specimens were left to soak for 18 more hours before weighing them again. Figure 3.14 indicates the apparatus used during testing of the specimens.

The water sorptivity **S**, was obtained from the slope of the straight line of mass of water absorbed versus the square root of time (as shown in equation 3.2). The lower the index, the better is the potential durability of the concrete. The values range from 5 mm/h for grade 30-50 concretes and 15-20 mm/h for grade 20 concretes.

$$\boldsymbol{S} = \frac{\Delta M_t}{t^{1/2}}\left(\frac{d}{M_{sat}-M_o}\right)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.2)$$

where:

ΔM_t = change in mass with respect to the oven dried mass (g)
M_o = mass of oven-dried concrete (g)
M_{sat} = mass of saturated surface dry concrete (g)
t = period of absorption (hr)
d = average thickness of sample (mm)


Figure 3.14: Water sorptivity testing underway at Wits University concrete lab

3.12.6.3 Chloride conductivity index test

The chloride conductivity test characterises the intrinsic potential of the concrete to resist chloride penetration (Streicher and Alexander, 1999). Streicher et al (1999) developed the test that involves measuring the ionic flux (current) across a sample due to a 10V potential difference shown in Figure 3.15.

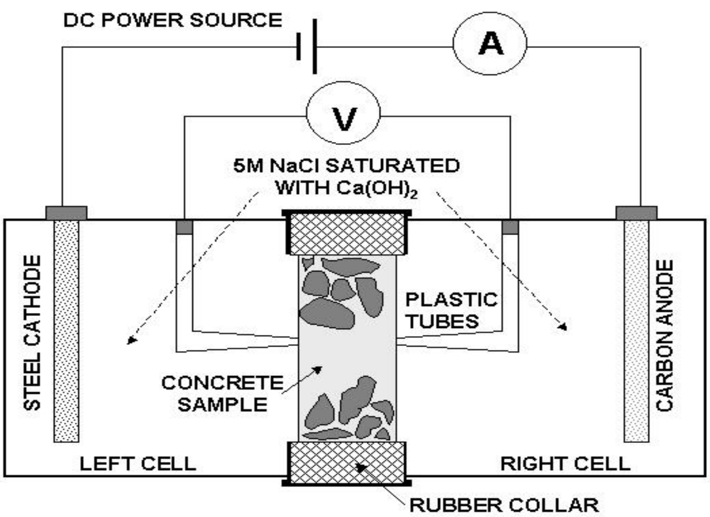


Figure 3.15: Schematic diagram of chloride conductivity apparatus (Alexander et al, 2017)

Pre-conditioning of the concrete disc samples (68 ± 2 mm diameter and 30 ± 2 mm thickness) involved oven drying at 50°C for seven days, followed by vacuum saturation in a 5M NaCl solution for 18 hours. The specimens were dried off, measured and then placed into the test rig. Once the specimens were placed inside the testing rig, the anode and cathode compartments were filled with NaCl solution through the holes, and tightened to make sure that there were no signs of leakage.

The test was then conducted by connecting and adjusting the ammeter and voltmeter with readings of the current and voltage recorded simultaneously as shown in Figure 3.16. The determination of the chloride conductivity (**σ**) index value which was calculated using the following, equation 3.3:

$$\sigma = \frac{it}{VA} \ldots\ldots\ldots\ldots\ldots\ldots (3.3)$$

where:

σ= conductivity of the specimen (mS/cm)
i = electric current (mA)
t = average thickness of specimen (cm)
V = voltage difference (V)
A = cross-sectional area of the specimen (cm^2)

The lower the index, the better is the potential durability of the concrete. The values range between 0.75 and 2.5.

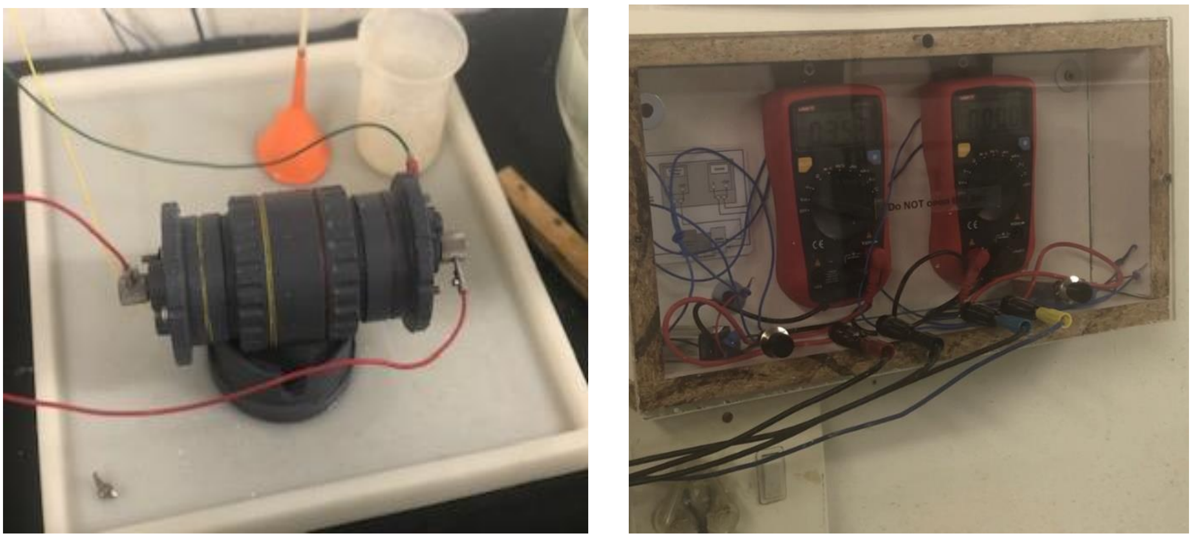


Figure 3.16: Chloride conductivity equipment at Wits University Concrete Lab

3.13 Assessment of water aggressiveness using the corrosion indices

In order to quantify the aggressiveness of the contained liquid, the Basson indices were utilised to quantify the corrosivity of the water. This approach is applicable to concrete structures exposed to aggressive liquids. Water samples taken from all reservoirs, are tested at Rand Water, water quality laboratories, on a monthly basis. Table 3.4 indicates the guidelines for assessing overall aggressiveness.

Table 3.4: Guidelines for assessing overall aggressiveness (N_C) (Ballim & Basson,2001)

N_C	Aggressiveness
Not greater than 300	None to mild
400-700	Mild to moderate
800-1000	High
= or > 1100	Very high

Table 3.5 indicates the parameters monitored, to allow the aggressiveness of the water to be quantified, using the aggressiviness index (**N_c**). To establish the mode of attack, the values of the LCSI and SCSI were compared and the greater of the two, indicated the dominant mode of attack.

Table 3.5: Calculation of corrosion indices (Ballim & Basson, 2001)

Property	Value	Formula	Index
pH	V1	N1 = 200 x (9.5 -V1)	N1
Calcium carbonate (saturated pH)	V2	N2 = 2000 x (V2-V1)	N2
Calcium hardness (as $CaCO_3$) *(mg/l)*	V3	N3 = 2,2 x (500 – V3)	N3
Total ammonium ion (NH_4)	V4	N4 = 10 x V4	N4
Magnesium ion (as Mg) *(mg/l)*	V5	N5 = 0.6 x V5	N5
Total sulphate ion (SO_4) *(mg/l)*	V6	N6 = 0.3 x V6	N6
Chloride ion (as Cl) *(mg/l)*	V7	N7 = 0.2 x V7	N7
Total dissolved solids (calculated) *(mg/l)*	V8		

Corrosion Indices (***LCSI & SCSI***) are determined as follows:

- **Leaching corrosion sub index:**

$$\boldsymbol{LCSI = \frac{N1 + N2 + N3}{3}}$$

- **Spalling corrosion sub index:**

$$\boldsymbol{SCSI = \frac{N4 + N5 + N6}{3}}$$

- **Aggressiveness index:**

N_c = (*LCSI* + *SCSI*)

**The greater of the two indices determines, the dominant mode of attack.*
** Corrections for turbulence, stagnance and temperature to compute the overall AI considered if applicable. Only temperature was considered for this study.*

CHAPTER 4 - RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents the results, analysis and discussion of the various assessments outlined in Chapter 3. The observations that were made on site during visual assessments of the actual reservoirs and visual assessment of cores at the laboratory are compared and conclusions drawn. An attempt to quantify the impact and effect of durability performance using durability indices and corrosion indices is also presented and discussed.

4.2 Preliminary assessment

4.2.1 On-site visual assessment

4.2.1.1 Summary of typical defects (structural and non-structural)

Structural defects refer to the deterioration of elements of a water reservoir's structural system, which threatens the structural integrity of the reservoir. These defects, as indicated in Table 4.1, may have major implications on the continued use of the reservoir. Non-structural defects are usually surface defects which do not have an immediate effect on the structural integrity of the reservoir. They may, however, have an effect on the long-term service life performance of reservoir elements eventually affecting the structural integrity.

Table 4.1: Typical defects observed in the Rand water reservoirs

Typical defects observed in concrete	• Cracks • Spalling • Delamination • Disintegration
Typical defects observed in reinforcement	• Uniform corrosion • Pitting corrosion
Cause of defects	• Carbonation • Corrosive contaminants • Stray currents • Insufficient cover

Visual assessment was conducted on three selected reservoirs i.e. Forest Hill, Brakfontein and Sasolburg.

4.2.2 Reservoir-specific on-site visual assessment results

4.2.2.1 Forest Hill reservoir

Forest Hill reservoir is one of the oldest (84 years) reservoirs to be assessed in this study. Signs of distress were observed in the roof and wall areas. The reinforcement on ring beam, wall sections and approximately 30% of the roof panels showed signs of corrosion, as shown in Figure 4.1 and Figure 4.2. The reinforcing bars on the ring beams were completely corroded, although it should be noted that, due to the gravel on the top of the roof, it was not possible to observe additional signs of distress, such as cracks.

Cracks traversing diagonally towards the construction joints were observed in approximately 40% of the roof panels. Additional observations were made of what appeared to be leaching of calcium hydroxide ($Ca(OH)_2$) on the wall and approximately 30% of the roof panels, as shown in Figure 4.1. Wet patches of concrete were observed throughout the reservoir, particularly above the water level and roof, as shown in Figure 4.2.

The surfaces of the walls had black stains and were generally chipped with aggregate exposed, and some areas of the concrete had spalled off. The roof and wall interface indicated signs of moist concrete.

(a) (b)

Figure 4.1: a) Roof cracked and leaching for Forest Hill reservoir
b) Internal wall leaching for Forest Hill reservoir

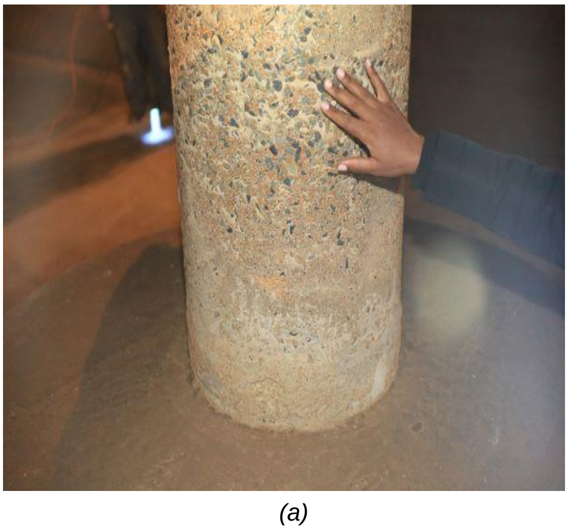
(a)

(b)

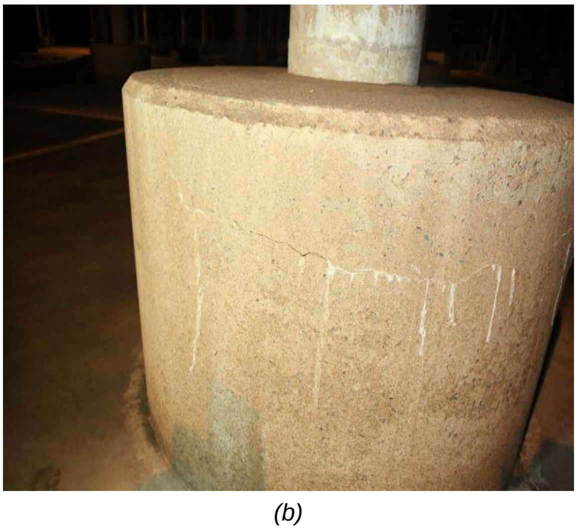
Figure 4.2: Forest Hill reservoir: a) moisture effects on wall b) corrosion on roof slab

4.2.2.2 Brakfontein reservoir

The following observations were made during the visual inspection of the internal and external reservoir areas. Most of the columns (Figure 4.3 and Figure 4.4) showed sections of exposed aggregate, softening of the surface, and cracks with leaching calcium hydroxide ($Ca(OH)_2$) from the inside of the concrete. When the $Ca(OH)_2$ reacts with carbon dioxide (CO_2) in the water it then deposits an insoluble $CaCO_3$, on the surface of the column (Ballim, 1993). Some of the columns had cracks around the bases, indicating possible additional loading strain on the columns, (Figure 4.5). Crack widths approximately over 0.1 mm were observed but not measured, on approximately 20% of the floor panels (Figure 4.5) and the cracks showed associated dark staining (Figure 4.6).



(a)



(b)

Figure 4.3: Brakfontein reservoir: a) exposed aggregate on column inside, b) cracked base leaching

(a) (b)

Figure 4.4: Brakfontein reservoir: a) fines disintegrated in one column b) cracked leaching column

(a) (b)

Figure 4.5: Brakfontein reservoir: a) & b) cracks on floor panels

(a) *(b)*

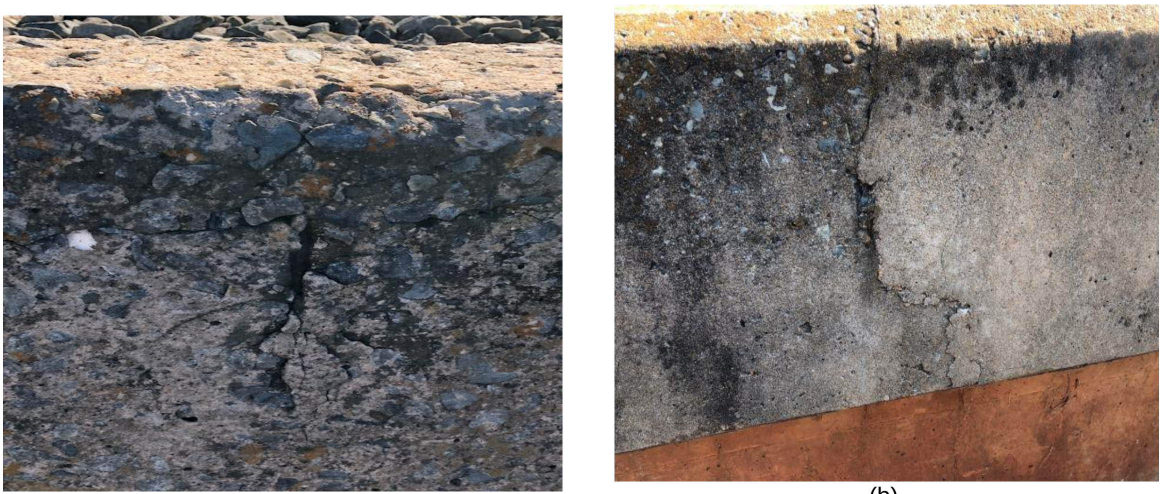
Figure 4.6: Brakfontein reservoir: a) cracks with associated dark staining, b) cracks on column bases

4.2.2.3 Sasolburg reservoir

The deterioration effects observed in this structure, considering the relatively younger age of the reservoir, are more advanced than other reservoirs. The corrosion appears substantial when compared to reservoirs of the same age in the Rand Water reservoir network.

Sasolburg reservoir is the reservoir where internal inspection was not possible within the timeframe of this study. Signs of distress were observed in the external parts of the roof and wall as shown in Figure 4.7 and Figure 4.8. The reinforcement on the ring beam and wall sections showed signs of complete corrosion (Figure 4.8).

The surfaces of the walls were stained black and generally showed scaling of the cement paste, with aggregate exposed. Some areas of the concrete had also spalled off. The roof and wall interface indicated signs of moist concrete. The photos (Figure 4.7 and Figure 4.8) show the condition of the reservoir roof and wall.



(a) (b)

Figure 4.7: Sasolburg reservoir a) cracked wall section and exposed aggregate b) stained wall along the length of the reservoir

(a) (b)

Figure 4.8: Sasolburg reservoir a) cracked wall section with exposed corroded reinforcing bar and exposed aggregate b) completely corroded reinforcing bar

4.2.3 General observations on the three reservoirs from on-site visual assessment

Corrosion products are evident on the surface of some columns observed during the internal visual inspection of Forest Hill and Brakfontein reservoirs, following the configuration of the spiral reinforcement. Ballim (1993) noted similar corrosion patterns on the columns he observed, where the corrosion is seen only on one side of the column. This may be due to eccentric placement of the circular reinforcement cage, with the cover depth being compromised on one side of the column. The leaching of calcium hydroxide associated with cracking is evident in all the reservoirs observed in this study and it affects all structural members of the reservoirs.

The cause of the leaching that was observed in the reservoirs where internal visual inspection was possible, could be due to the fact that structures that are submerged are vulnerable, as the solubility of CO_2 increases with pressure, resulting in higher concentrations of H_2CO_3 (Ballim & Basson, 2001). They further explain that the initial reaction with the $Ca(OH)_2$ of the HCP results in the formation of $CaCO_3$; however, the carbonate is rapidly converted to the more soluble bicarbonate which then results in more rapid leaching.

4.2.4 Laboratory visual assessment of concrete cores

Visual assessments were also conducted on extracted concrete cores as shown in Figure 4.9 and focussed on the following:

- size and type of aggregate;
- aggregate distribution;
- voids content;
- presence of defects and/or any evidence of deterioration.

A summary of the assessment is presented in Table 4.2 and these were interpreted with the results from the on-site visual assessment of the reservoirs. A detailed assessment is shown in Appendix A.

Table 4.2: Visual assessment on cores

Reservoir	**Visual assessment findings**
Brakfontein	• All the concrete cores were generally of good quality with respect to aggregate distribution and voids content (ca. 0.5%). • No signs of concrete deterioration were observed on the concrete cores. • Witwatersrand quartzite aggregate.
Sasolburg	• The majority of concrete cores were of good quality with respect to aggregate distribution and voids content (ca. 0.5%). • Some cores showed drying shrinkage cracking, while some had excess fines. These could have resulted from of a number of factors including poor on-site curing, poor compaction and/or poor concrete mix proportions. • Witwatersrand quartzite aggregate.
Forest Hill	• Some cores had single-sized coarse aggregates. • Signs of reinforcing steel corrosion were evident on the concrete cores – concrete cover to steel had spalled, and the exposed reinforcing steel was severely corroded. • Witwatersrand quartzite aggregate.

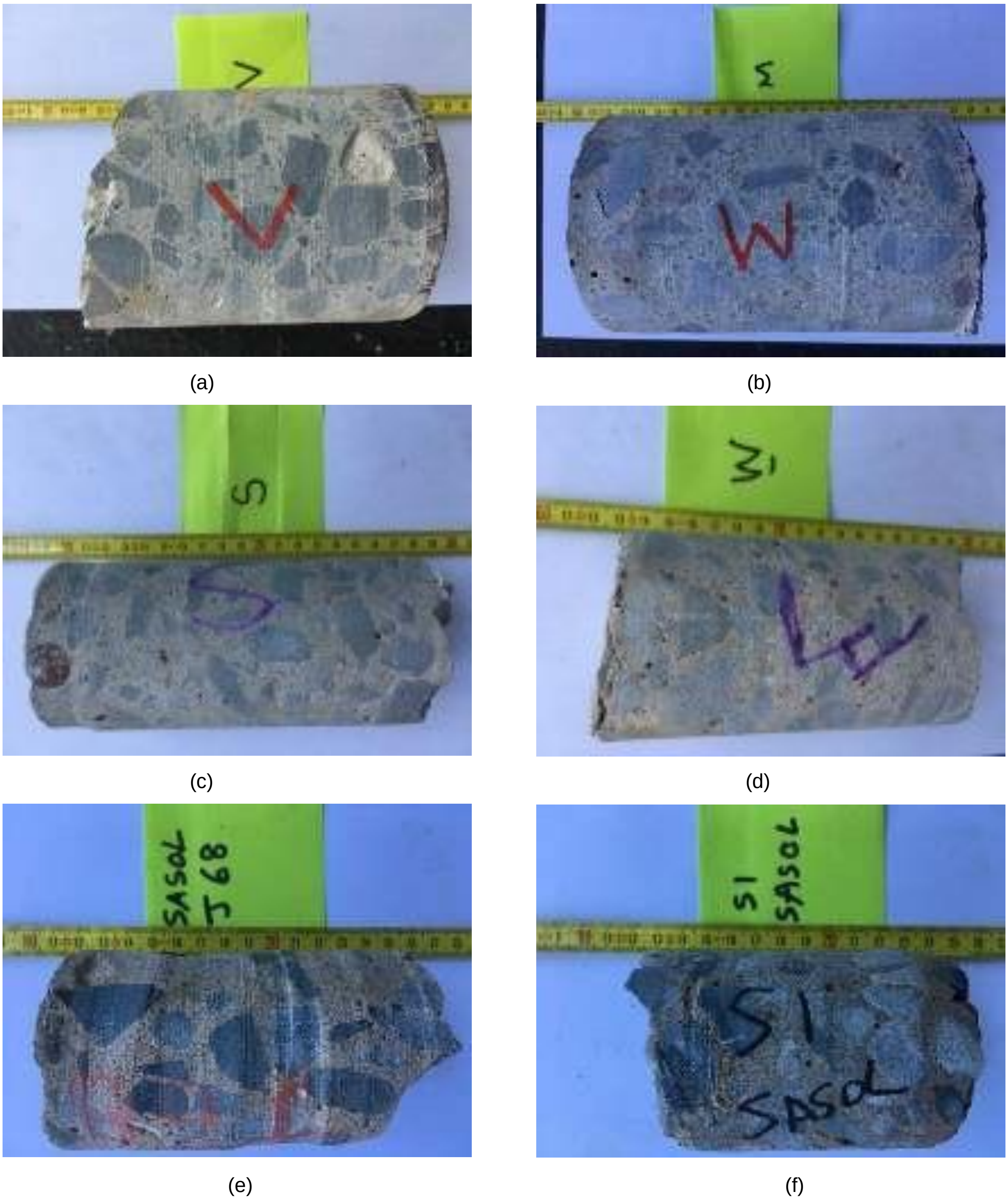


Figure 4.9: Cores observed during visual assessment: a) and b) Brakfontein reservoir; c) and d) Forest Hill reservoir: e) and f) Sasolburg reservoir

4.2.5 Surface hardness (Schmidt hammer testing)

Surface hardness (rebound index) results are presented in Appendix A. These rebound index values are based on Schmidt hammer test and correlated with the compressive strength results (Figure 4.10). These results indicate a trend which is contrary to expected results (i.e. increasing compressive strength with increasing rebound index) and therefore deemed unreliable.

The unreliability of the test as a surface measure on deteriorated or softening of the surface zone was compared with the compressive strength of internal concrete and possibly high local variation in concrete quality.

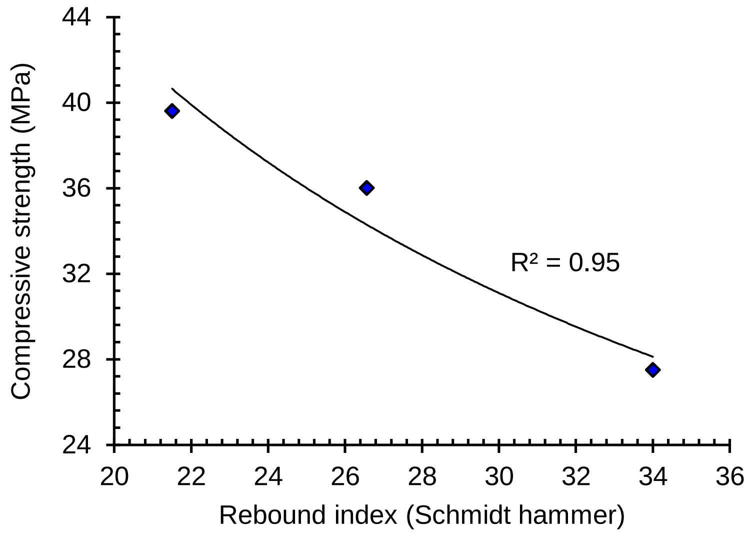


Figure 4.10: Plot of cumulative % of frequency of cover depth measurements of the reservoirs

4.2.6 Cover depth surveys

Cover depth surveys were conducted on site, using a cover meter. A summary of the results is presented in Figure 4.11, Figure 4.12 and Table 4.3. The results indicate that the cover depth ranges between 20 – 60 mm for the Brakfontein and Sasolburg reservoirs and a range of 10 – 120 mm for Foresthill reservoir. These values are rounded off to the nearest 10 mm for ease of interpretation.

Approximately 50% of the results have a minimum of 40 mm of cover. The low values of the cover (,40 mm) are not in compliance with the design code for WRS (BS 8007), which stipulates a minimum cover depth of 40 mm. This is also indicative of durability problems regarding cover reinforcement and to protect against ingress of deleterious materials including corrosion of the reinforcement. The results also show a wide scatter of the cover depth measurements for all reservoirs. This is evident from the high values of the coefficient of variation (CoV) in Table 4.3.

Table 4.3: Minimum and maximum measured cover depths

Reservoir	Minimum cover depth (mm)	Maximum cover depth (mm)	CoV (%)
Brakfontein	20	60	36
Sasolburg	20	60	40
Forest hill	10	120	69

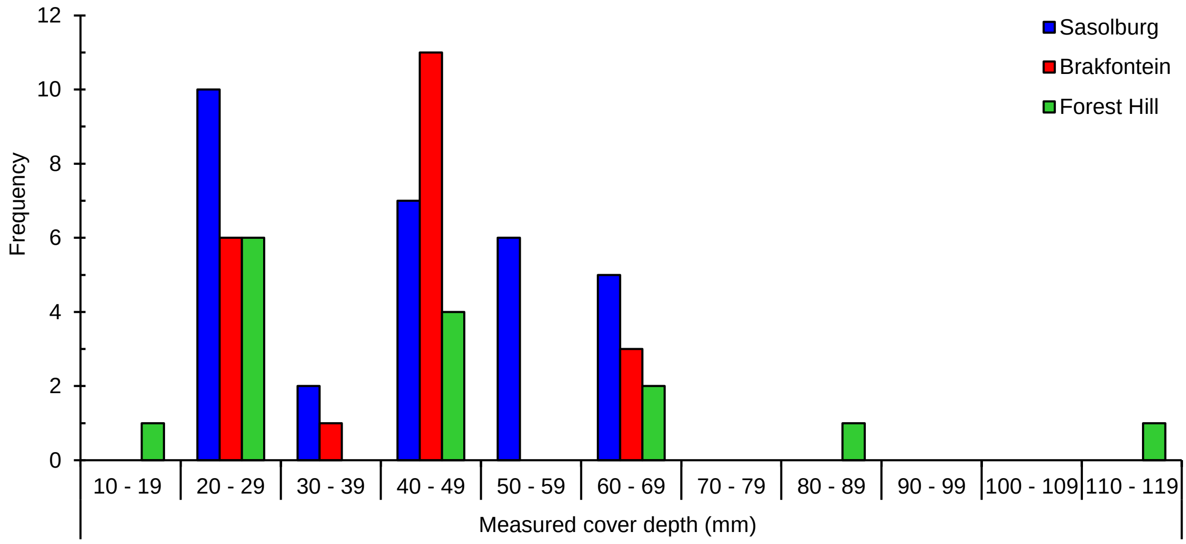


Figure 4.11: Distribution of measured cover depths for the reservoirs

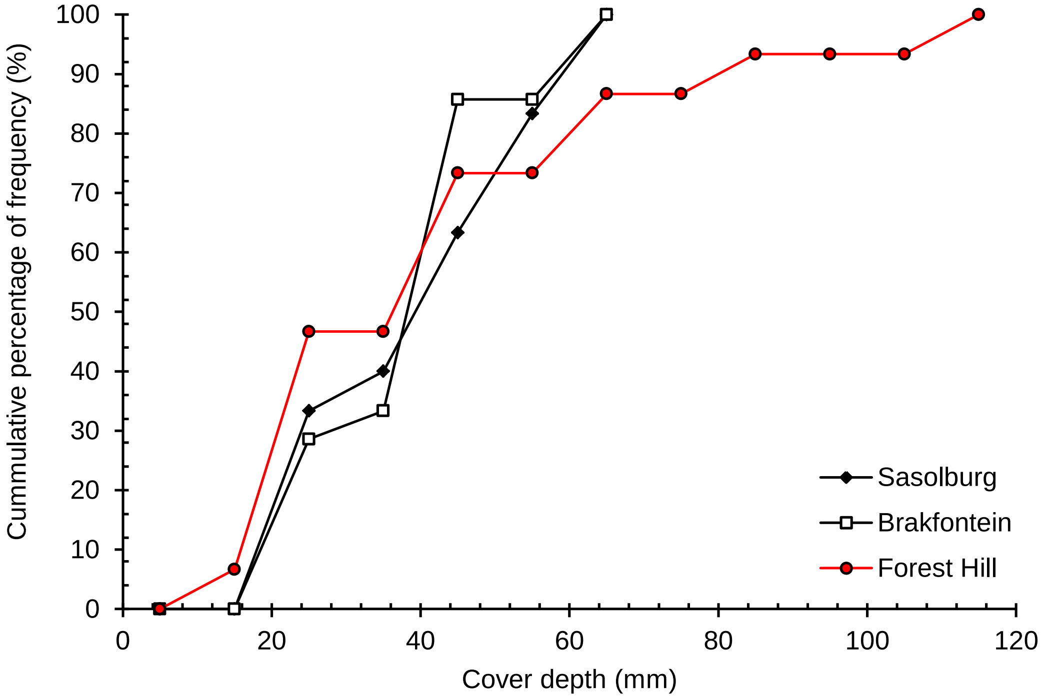


Figure 4.12: Plot of cumulative % of frequency of cover depth measurements of the reservoirs

It is evident from the results of the cover depth surveys and visual assessment that the reinforcement is prone to corrosion in some areas, which will eventually affect the structural integrity of the reservoirs. It is also evident that the older reservoirs were not designed in compliance with the design code for water reservoirs, which stipulates that the cover to reinforcement should be a minimum of 40 mm. Urgent action is required to restore the cover layer in areas where the damage is evident, and to protect the reinforcement from further corrosion. The loss of reinforcement cross sectional area due to corrosion, as observed in all reservoirs, is further evidence of the damage to the cover layer.

The variations and inconsistencies with achieving the cover depth could possibly be due to inadequate construction practices at the time. In practice there will be some variation in distribution of reinforcement depths. Thus, the reinforcement with less cover, where the passive layer has been destroyed, will have a greater probability of corrosion than other areas of the reinforcement.

4.2.7 General observations from the preliminary assessment

The visual assessment of the reservoirs on-site indicates concrete that has been subjected to multiple deterioration mechanisms. However, the visual assessment performed on cores at the laboratory indicates that concrete is generally in good condition for all reservoirs. This indicates that surface damage is more significant, while internal concrete is acceptable. The findings show that the three reservoirs have suffered significant deterioration due to leaching, spalling, cracking and reinforcement corrosion. The significant deterioration at this stage, was still restricted to the surface zone. The internal concrete is in good state and is not in need of attention. Therefore, the following two observations can be made:

- The most important concern is corrosion of the reinforcement in the near surface zone, which is a surface phenomenon with serious structural consequences;
- This may also explain the counter-intuitive results of the Schmidt hammer in relation to compressive strength. Schmidt hammer on damaged/deteriorated surface vs compressive strength on good, internal concrete. It is also indicative of high surface values with low internal strength.

Approximately 50% of the cover depth results indicate that the cover is over 40 mm, which is the design minimum and this should give some comfort over the corrosion exposure of the surface zone.

Upon observation of the older reservoirs, i.e. Forest Hill (84 years) and Sasolburg (49 years) concerns of possible alkali silica reaction (ASR) came to light due to the cracking, use of Witwatersrand quartzite aggregate and of Portland cement in the concrete mix. Oberholster (2001) stated that there are three factors that influence ASR a) high alkalinity in the pore solution b) reactive aggregates and c) environmental conditions (i.e. moisture, temperature and climate) to promote the reaction. The Witwatersrand quartzite is an aggregate identified as potentially alkali reactive, both through service record and laboratory testing (Oberholster, 2001). Johannesburg is one of the cities with the highest incidence of ASR. ASR has affected structures such as reservoirs and bridges, and in all cases the aggregates that were utilised contained the Witwatersrand supergroup quartzite (Oberholster, 2001). Petrographic analysis was not conducted as part of this study to ascertain the presence of ASR; this will require further studies.

4.3 Detailed assessment

4.3.1 Carbonation depth

The results of the carbonation depth measurements are presented in Appendix A. Figure 4.13 shows a plot of the measured carbonation depths in the reservoirs. The measured carbonation depths are relatively high, but these should be assessed considering the cover depth to reinforcing steel in these reservoirs.

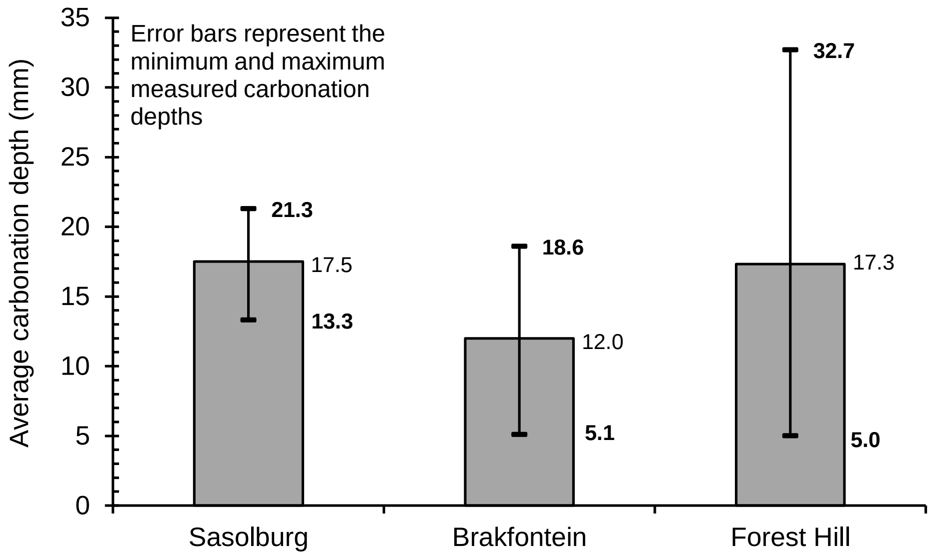


Figure 4.13: Scatter of the measured carbonation depths in the various reservoirs

“Carbonation rate is defined as the progress of the carbonation front in concrete with time. It is used to assess the carbonation resistance of different concrete and it is influenced by w/b ratio, binder type, duration of curing and climatic conditions at the exposure site and duration of exposure” (Ikotun, 2017). Carbonation of concrete is a very slow process and the rate is governed by a number of factors including the penetrability of concrete, type of binder and the concentration of carbon dioxide, relative humidity and temperature of the environment.

It is generally accepted that the progress of carbonation is governed by the following equation 4.1:

$$\boldsymbol{x} = K_c\sqrt{t} \text{ ....equation 4.1}$$

Where x = carbonation depth
t = carbonation exposure duration
K_c = carbonation coefficient

In order to assess the rates of carbonation in the concretes, the estimated carbonation coefficients (**K_c**) based on equation 4.1, are presented in Table 4.4. These values are calculated assuming a conservative square-root of time carbonation service life model. Using these values, and the measured maximum carbonation depths, cumulative percentage of frequency (%), conservative estimates of the residual service life values of the reservoirs were made – these are presented in Table 4.4.

Table 4.4: Carbonation coefficient (k_c) values and estimated conservative residual service life

Reservoir	K_c (mm/√yr)	Residual service life (years)
Brakfontein	3.88	30.4
Sasolburg	3.11	N/A
Forest Hill	3.57	4.2

*****N/A for Sasolburg carbonation front has already exceeded the minimum cover depth to steel

Figure 4.14 shows a comparison of the measured maximum carbonation depths with the corresponding minimum measured cover depths for the panels where the concrete cores used for carbonation depth measurements were taken.

It is evident from this graph that the carbonation depths in the Sasolburg reservoir are beyond the level of the reinforcing steel. These results mean that the reinforcing steel in this reservoir is already subject to steel corrosion initiation and possible propagation. This is further confirmed by the visual assessment conducted, and evidenced in Figure 4.8.

The Sasolburg reservoir is situated close to Sasol Industries, a petrochemical company. An assumption can be made that there is a likelihood that CO_2 levels are higher than average in this area, which could result in higher carbonation rate in the concrete. This is a further reason for concern with this particular reservoir.

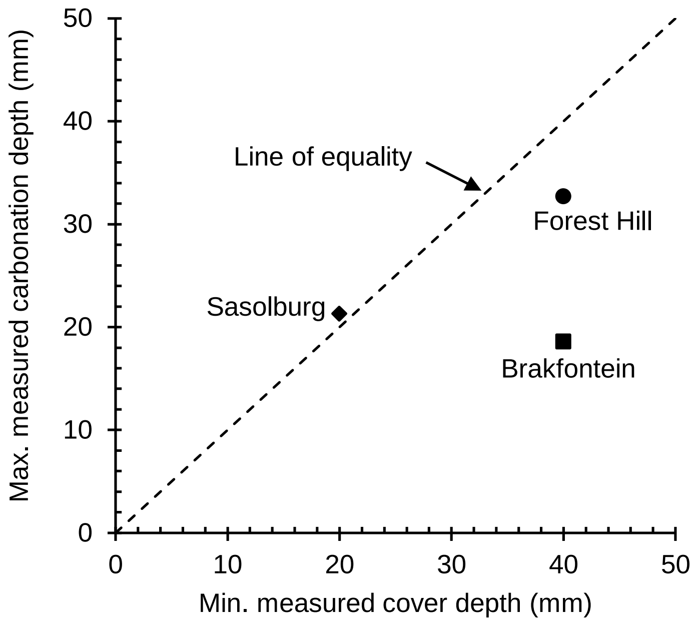


Figure 4.14: Comparisons of carbonation depth and concrete cover depth results of all the reservoirs

Carbonation is the main deterioration mechanism that leads to corrosion in the Gauteng region (Lampacher, 2000). The environmental conditions which a structure is exposed to over its service life has a direct influence on the carbonation process. Carbonation is more severe in areas exposed to relative humidity of 50% - 70%, and this is the range typically found in the Gauteng region where the assessed reservoirs are located. Therefore, it is no surprise that the chosen reservoirs are all experiencing significant carbonation fronts in the cover layer, some reaching the reinforcing level in the concrete. Also, the Forest Hill reservoir, which is the older of the reservoirs, showed high depths of carbonation therefore confirming that the age of the structure has an impact on the extent of carbonation it will experience over its service life.

4.3.2 Compressive strength

The compressive strength results of the tests on the cores obtained from the reservoirs are presented in Figure 4.15 and Appendix A.

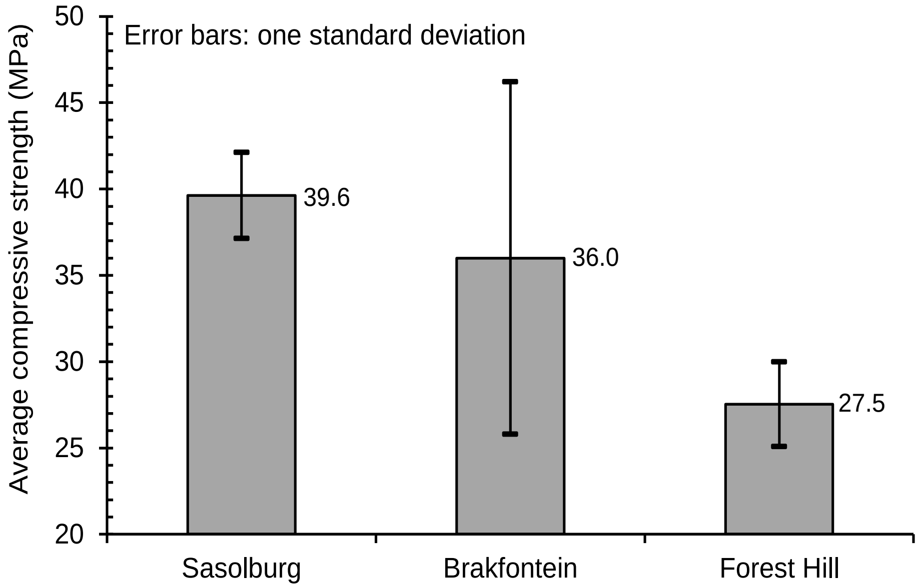


Figure 4.15: Variability of compressive strength results for samples taken from cores extracted from all reservoirs

These results show that some of the concretes meet the structural grade requirement of at least 35 MPa, the design strength for water retaining structures as per the design code for water retaining structures, BS 8007. This code has been the basis for Rand Water Reservoir Design.

The compressive strength value of the Forest Hill reservoir is relatively low, taking into consideration the advanced age of the reservoir (84 years). The range in the compressive strength results for the Brakfontein results is relatively high and may point to a number of possible causes, including inherent poor-quality concrete mix, poor on-site concrete quality control, and use of different concrete mixtures in the construction process.

A comparison of the actual and potential compressive strengths is presented in Figure 4.16 and shows that the actual and potential compressive strengths are similar. This is a clear indication of the volume of voids in the concrete, hence compaction was adequate. The low strength concrete must therefore be attributed to the use of a poor-quality concrete mix.

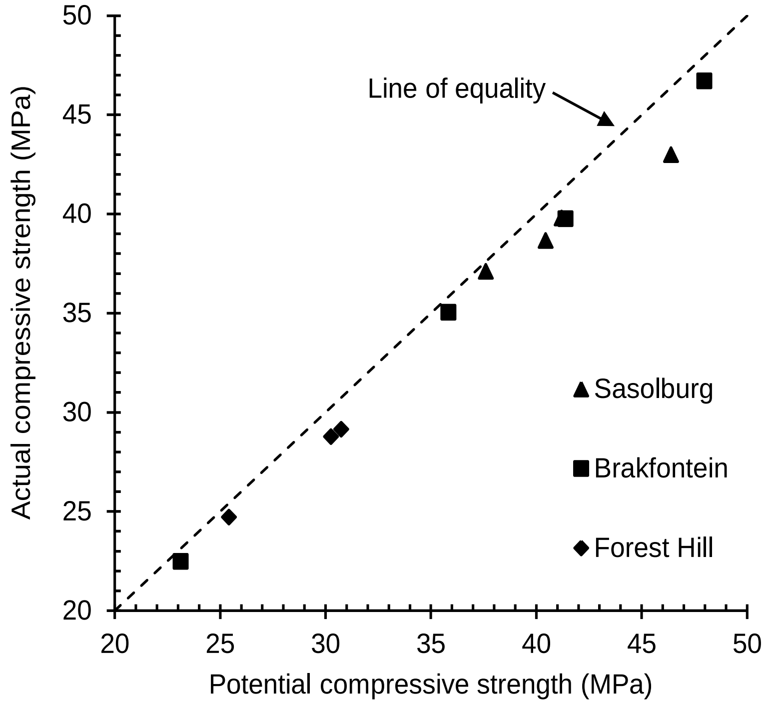


Figure 4.16: Comparison of the actual and potential compressive strengths for samples taken from cores extracted from all reservoirs

4.3.3 Durability testing (concrete penetrability)

The Durability Index tests measure different transport mechanisms in concrete to predict its potential durability performance. Results of the DI are presented in Appendix A.

4.3.3.1 Chloride conductivity index

Chloride conductivity index is a direct measure of the resistance of the material to the movement of a charge under an electrical potential gradient. It also measures the susceptibility of concrete to ingress of chlorides by diffusion.

"*As corrosion is dependent upon ionic movement through concrete to complete the electrochemical circuit, a decrease in the conductivity can reduce or limit the corrosion rate. Based on these, chloride conductivity can be correlated to corrosion rate*" (Scott, 2004). The CCI values of all the reservoirs are relatively high, signifying susceptibility to chloride penetration leading to corrosion, as shown in Figure 4.17.

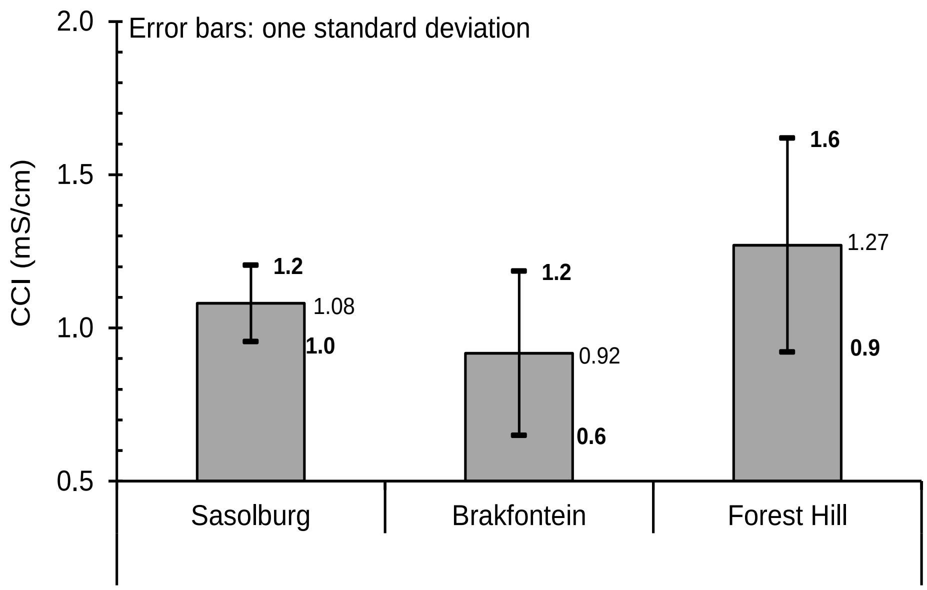


Figure 4.17: Chloride conductivity results for samples taken from cores extracted from all reservoirs

4.3.3.2 Oxygen permeability index

Oxygen permeability index (OPI) represents the negative log of the coefficient of oxygen permeability, k, (m/s) through a concrete sample (i.e. –log10 k). A poor concrete with good connectivity of the pores will therefore have a high coefficient of permeability (low OPI value) and would allow easy access of oxygen (and carbon dioxide) through the concrete.

Concretes with very low coefficients of permeability (high OPI value) would impede the access of oxygen to the steel and limit the corrosion progress, given sufficient (thickness and/or quality) concrete cover. The results of the OPI measured on the core samples taken from the test reservoirs are presented in Figure 4.18. The OPI values of all the reservoirs are ~ 9 signifying poor quality concrete, considering the age of the reservoirs.

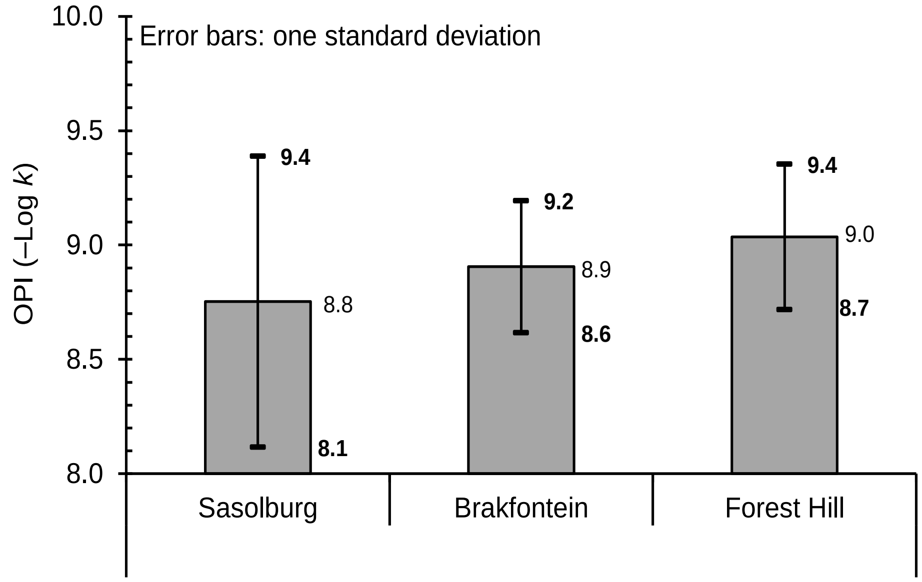


Figure 4.18: Oxygen permeability results for samples taken from cores extracted from all reservoirs

4.3.3.3 Water sorptivity index

Sorptivity is a measure of the movement of a wetting front through a porous medium due to capillary action (Alexander et al., 2017). The rate at which the wetting front moves through the concrete, as measured by the gain of mass with time, provides information on the general pore structure of the material. The water sorptivity index (WSI) test measures the rate of movement of a water front through the concrete under capillary suction, normalised by porosity. It is particularly sensitive to the near-surface properties of concrete and therefore reflects the effectiveness of curing (Ballim, 1994).

The lower the sorptivity, the more resistant is the concrete to penetration of moisture. The results of the WSI measured on the core samples taken from the test reservoirs are presented in Figure 4.19. The WSI values of Brakfontein and Sasolburg are relatively high, signifying high susceptibility to moisture ingress.

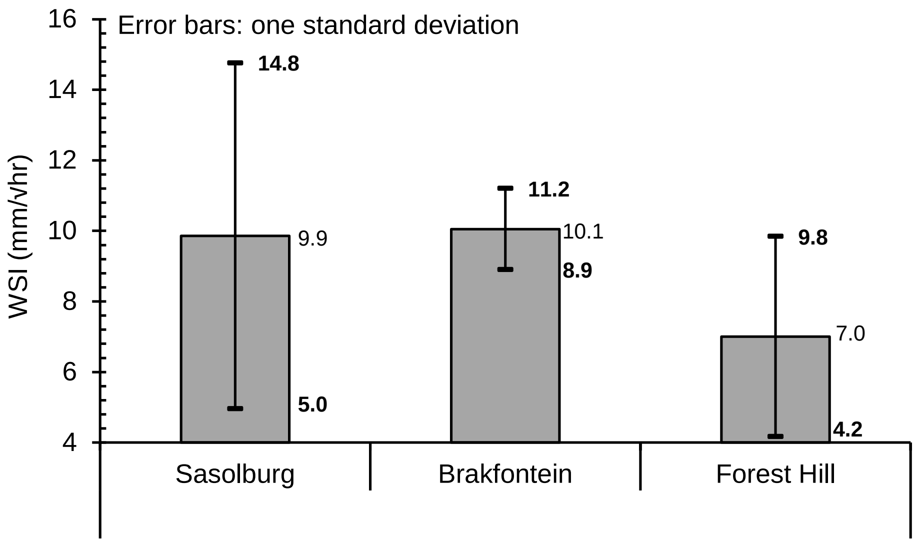


Figure 4.19: Water sorptivity results for samples taken from cores extracted from all reservoirs

Blight et al. (1998) reported that there is an increased permeability of the surface of mature structures, but also noted a decreased sorptivity on the structures they had tested. This was not evident in case of the Rand Water reservoirs. Blight et al. (1998) also indicated that there is no correlation between the two parameters and therefore they cannot be used as measures of quality for mature structures. However, more research has since been conducted on mature structures, to provide more information and understanding of the indices and parameters.

The results for the durability index tests indicate that the concrete for all reservoirs has poor durability potential. The WSI results for Brakfontein and Sasolburg were relatively high, signifying concrete susceptible to moisture ingress by absorption, whereas the Forest Hill (the oldest of the reservoirs) results were within acceptable limits. The OPI values were all ~ 9, signifying poor quality concrete with regard to permeability. This is also possibly due to the advanced age of the concrete. The CCI values of all the reservoirs are relatively high, signifying susceptibility to chloride penetration leading to corrosion.

Blight et al. (1998) studied mature structures with the intention of finding out if the durability tests used for quality control in early-age concrete can be used in mature structures. Their study revealed a conflicting relationship of the indices between early-age concrete and mature concretes in service. For the mature concretes, increased permeability values were associated with decreased WSI values. This is in contrast with the Rand Water reservoirs where permeability values were low and WSI values were high.

Dias (2000) reported in a study of mature structures, that water sorptivity decreases with age due to surface carbonation, and that surfaces with poor initial quality benefit from more carbonation as they experience greater carbonation depths. This is in contrast to the results obtained in this study, particularly Sasolburg and Brakfontein reservoirs which presented with high sorptivity values.

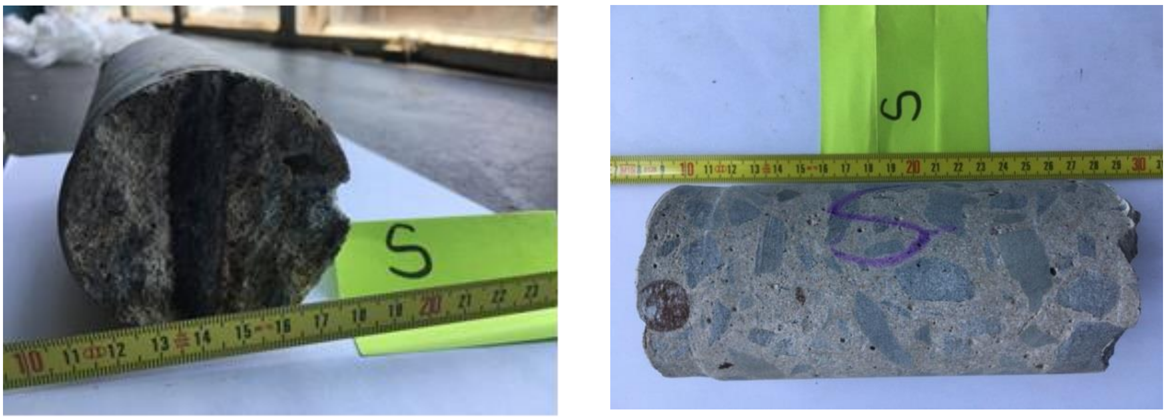
The overall results confirm the concern about the reduced quality of the surface concrete and its ability to protect the reinforcement against corrosion.

4.3.4 Steel corrosion rate assessment

Corrosion of steel reinforcement is one of the major causes of deterioration in concrete structures. Corrosion of steel reinforcement has become a significant downside for the durability of reinforced concrete structures, considering the economic consequences linked to assessment, maintenance and repair for the increasing amount of ageing infrastructure (Otieno, 2008).

When corrosion develops significantly in a structure, it impacts on the structural integrity and the load carrying capacity of the structural members. The loss of steel cross section is one of the structural consequences due to corrosion. Corrosion not only impacts the structural integrity but has dire consequences on the serviceability of the structure. The service life of corroding reinforced concrete structures is reduced by the presence of corrosion-induced cracks and/or spalling of the concrete cover due to products of corrosion. Additional cores were extracted from Forest Hill reservoir, in areas where corrosion of reinforcement was evident during the visual assessment of the reservoir.

The nature and extent of the corrosion in this reservoir are discussed below. Figures 4.20 to Figure 4.22 show core samples extracted from areas where corrosion was evident in Forest Hill reservoir.



(a) (b)

Figure 4.20: Photograph of a concrete core sample with a corroded steel bar from Forest Hill reservoir

(a) (b)

Figure 4.21: Photograph of a concrete core sample with a corroded steel bar from Forest Hill reservoir

(a) (b)

Figure 4.22: Photographs of a corroded steel bar extracted from the core sample a) U and b) S from Forest Hill reservoir are shown here

Table 4.5 indicates the percentage steel loss due to corrosion for the reinforcement samples extracted from the cores shown in Figure 4.22. There is a 15% loss in the cross sectional area of the reinforcing bars. The measurement of the cross-sectional area confirmed the presence of corrosion initiated by carbonation. This was expected, considering the age and the depth of carbonation measured in the concrete.

Table 4.5: Steel corrosion mass loss results

Core label	Measured bar diameter (mm) In the uncorroded sections	Minimum measured bar diameter (mm) In the corroded sections	Percentage mass loss (%)	Percentage cross section loss (%)
S	15.94	14.70	6.3	15.0
U	16.00	14.76	4.4	14.9

The high degree of steel corrosion (up to 15% loss in cross-section) shows that some of the reservoirs are structurally at risk – potentially due to the low cover depth. Corrosion protection measures, such as application of suitable coatings to protect the embedded reinforcing steel from corrosion, are urgently required now if the functional life of the structures is to be extended.

4.3.5 Aggressivity of water and its effect on reservoirs

The Basson indices provide a measure of the aggressivity of water when in contact with concrete. The reservoir water is sampled on a monthly basis from the tertiary network i.e. reservoir sites. These results were used to determine the corrosion indices i.e. Leaching corrosion sub-index (LCSI) and the Spalling sub-index (SCSI). These indices were then used to calculate the overall Aggressivity index (AI) and results are presented in Figure 4.23.

Values of the calculated aggressiveness index, over a period of time, are presented in Appendix A. The parameters used to compute the AI, are not monitored monthly consistently, and thus presented a limitation to this analysis. The water quality monitoring programme has key properties that it monitors on a monthly basis, such as pH. The other parameters are not monitored every month, and average values were utilised to compute the AI.

Based on the results in Figure 4.23, it was clear that the structures are exposed to water with an average of AI (~ 800), which is a relatively high aggressiveness index. This is to be expected considering the 3-stage dosing methodology (ammonia, monochloramine and chlorine disinfectants) that Rand Water utilises.

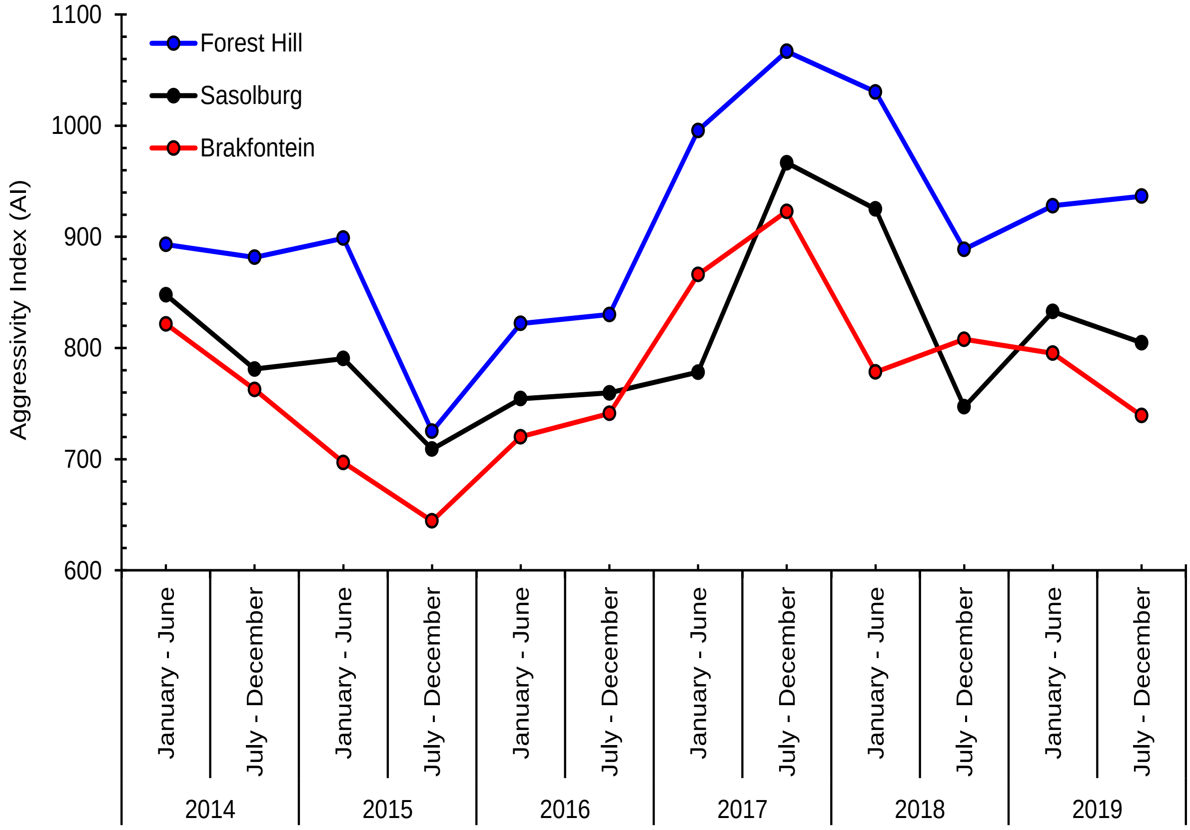


Figure 4.23: Typical average measured values of selected properties of aggressive water over indicated periods

Due to the observations of leaching of $Ca(OH)_2$ from the visual assessment for Brakfontein reservoir, an extended water aggressivity analysis was conducted. The rationale behind this was to understand the evolution of the treatment regime over 10 years in that particular reservoir. Figure 4.24 below indicates lower values of the Basson index in the period between 2009 and 2014, which indicates that the water was mildly aggressive for that period.

An increase in the pH, resulted in the high increase of AI between 2016 and 2017 (Figure 4.23) and between 2009 and 2014 (Figure 4.24). This increase can be attributed to a change in pH and softness, resulting in operational changes in the dosing methodology. To achieve the required alkalinity, lime is introduced in the dosing to achieve the required pH in compliance with water quality standards.

The high levels of Basson indices have remained consistent around an average of (AI ~ 800) throughout the five-year period. However, the AI ranged between 500 and 600 in the year 2009 (Figure 4.24). This is still indicative of mild to moderate aggressiveness in the water.

Ballim et al. (1993) conducted an analysis on the aggressiveness of the water from Rand Water reservoirs between 1977 and 1988 and concluded that Rand Water reservoir water was, at the time, found to be on occasion soft and considered to be aggressive to concrete.

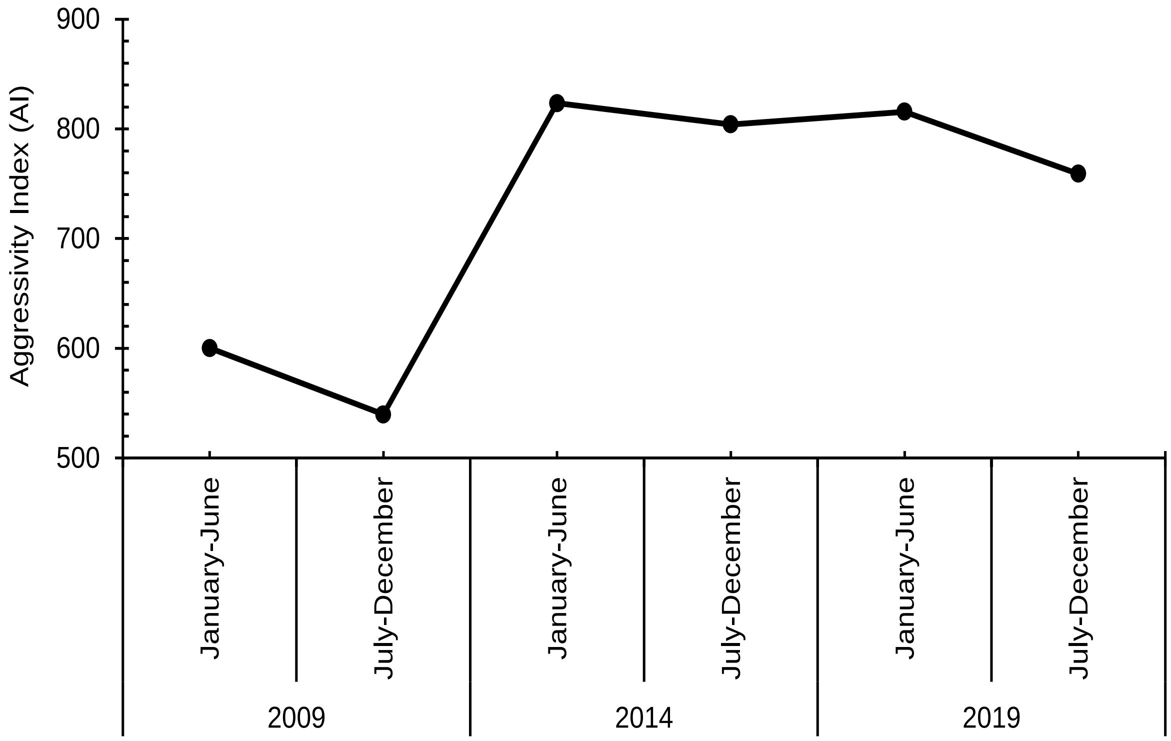


Figure 4.24: Brakfontein reservoir average measured values of selected properties of aggressive water over indicated periods to compute AI.

The overall values of AI for all three reservoirs were then corrected for temperature as shown in Figure 4.25. It was noted from literature that temperature has an influence on the aggressivity of water and hence the rate of corrosion worsens in the warmer seasons.

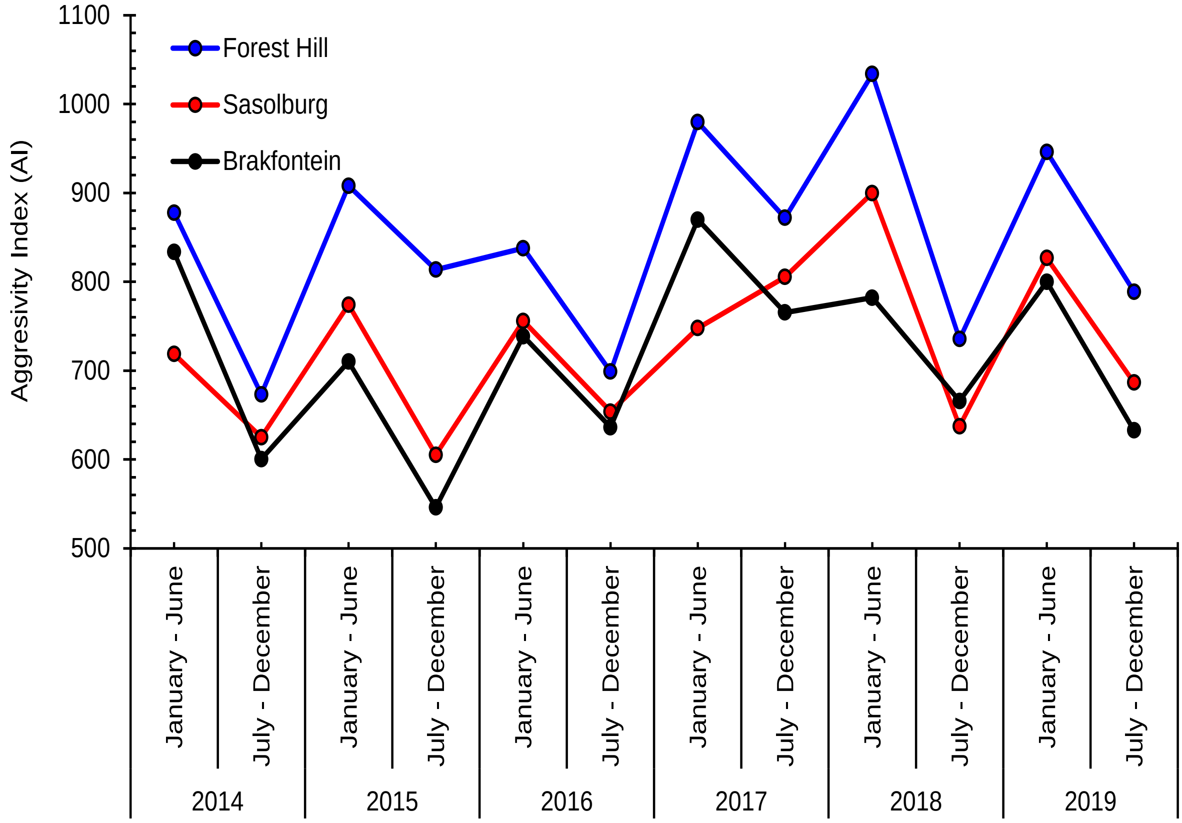


Figure 4.25: Temperature-corrected average aggressivity indices of the reservoirs

4.3.5.1 Implications of leaching and spalling corrosion

To determine the dominant deterioration mechanism, a comparison between the LCSI and the SCSI was conducted. This analysis indicated that the major deterioration mechanism internally results from the LCSI. This is also evident from the visual assessment conducted internally for all the structures. All reservoirs inspected internally had indications of leaching on all structural members of the reservoir. The chemicals utilised in the water treatment process have an impact on the deterioration of the reservoir structures irrespective of the source of the raw water. Therefore, the provision of an internal coating system is recommended to line and protect the concrete.

The overall results from this study show aggressive water throughout the period of analysis, even when corrected for temperature to achieve the overall AI. The possibility of ASR and the impacts due to high alkalinity need to considered and studied further.

CHAPTER 5 - CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The aim of this study was to investigate the durability performance of aged reinforced concrete water reservoirs. The results indicated that all the concrete reservoirs tested for durability performance experienced poor concrete quality and were all experiencing significant deterioration mechanisms. In general, the study showed that all reservoirs tested for durability performance experienced poor concrete quality, and were prone to corrosion due to carbonation, ASR, insufficient cover layer and aggressive waters.

5.1.1 Durability performance of the reservoirs

5.1.1.1 Modes of deterioration (aggressiveness of the exposure environments)

- **Carbonation and reinforcement corrosion**

Carbonation is the main deterioration mechanism that leads to corrosion in the Gauteng region (Lampacher,2000). The Rand Water network of reservoirs lies in this area, therefore the expectation is that this mode of deterioration will eventually impact all the reservoirs.

The results indicate that all the reservoirs are experiencing relatively high values of carbonation depths and therefore are highly susceptible to corrosion. The carbon dioxide diffusion through concrete is mainly governed by characteristics of its porous nature. The results for the durability index tests indicate that the concrete for all reservoirs is of poor quality. The WSI results ranged between 3.16 and 12.79 signifying concrete susceptible to moisture ingress by absorption for some reservoirs. The OPI values ranged between 8.02 and 9.43, signifying poor quality concrete with regards to permeability for all reservoirs. The CCI values ranged between 0.58 and 1.62 signifying susceptibility to chloride penetration leading to corrosion for some reservoirs. The indices characterise the near surface properties of concrete and measure its resistance to fluid and ionic transport mechanisms. A reduced quality in the surface concrete results in its inability to protect the reinforcement against corrosion (Alexander,2008).

Lampacher (2000) noted in a study that the rate of carbonation in Gauteng is considered to be a fairly rapid 3.16 mm/√yr. This value is slightly lower than the values found in this study, except for the case in the Sasolburg region, where the value is 3.11 mm/√yr. This fairly rapid rate implies the comparatively early depassivation of the steel and the inherent risk of steel corrosion. Sasolburg reservoir carbonation font has exceeded the minimum cover depth to steel, already experiencing extensive corrosion, as observed during the visual assessment on-site.

The assumption that there is a likelihood that CO_2 levels are higher than average in this area which could result in higher carbonation rate in the concrete, needs to be investigated further.

Counter measures, in a form of maintenance strategies explained below, need to be put in place where the carbonation rate has not exceeded cover depth to steel. The decision to build future reservoirs in the Sasolburg area needs to be evaluated from a financial point of view, as the initial capital investment might be slightly higher in this region due to the exposure environment.

- **Chemical aggressivity**

The general conclusion from the analysis of water aggressivity is that the water in all reservoirs is aggressive in its leaching potential, irrespective of the source of raw water. It is also evident that the softening of the inner surface concrete of the reservoirs, resulting from the leaching of $Ca(OH)_2$ from the concrete, is the cause of the deterioration of the binding capacity of the cement in the surface region.

To counter the effects of leaching, Basson et al. (2001) proposed a coating system to line and protect the concrete. Modifications to the mix design for future reservoirs are therefore required to counter the impacts of the aggressive waters. This is an aspect to be investigated further, where civil engineering and water chemistry intersect. Regular assessments should be conducted on the water being stored in the reservoirs to ensure early detection of aggressiveness and recommend that appropriate action be implemented.

The monitoring of the chemical parameters required to compute the aggressivity index needs to be done on a monthly basis for accuracy in computing the aggressivity index.

- **Improvement and restoration of cover depth**

Approximately 50% of the results indicate that the cover depth of the structures was not compliant with design requirements and therefore not sufficient to protect reinforcement from ingress of deleterious material.

Urgent action therefore is required to restore the cover layer in areas where the damage is evident, and to protect the reinforcement from further corrosion. The cover layer provides the initial resistance to the degradation agents. Cover depth, quality and condition, all have significant effects on the processes that lead to the corrosion of steel in concrete. Therefore, minimum cover depth to resist different degrees of aggressiveness of the environment must be incorporated in all new designs for reinforced concrete structures to ensure durability performance. Inconsistencies to achieve cover depth in practice can be avoided by utilising good quality, dense concrete cover blocks, and constant supervision. Improvement of the quality of concrete therefore is required to ensure that durability performance is achieved. It is mandatory that Rand Water specifies and incorporates the durability indices in all specifications for Rand Water reservoirs and other reinforced concrete structures.

5.1.1.2 Degree of deterioration and residual service life

Rand Water reservoirs are designed to have a lifespan of 60 years as prescribed by design codes, and their functional life is usually extended to 80 years with a rigorous maintenance intervention programme. Carbonation and corrosion service life prediction models need to be investigated and developed for all reservoirs in the network.

Rand Water currently utilises a condition grading system rate for its reservoirs. Figure 5.1 represents a service life model that can be integrated into the management of the reservoirs. Verma et al. (2013) developed the model in Figure 5.1 that predicts the residual service life of reinforced concrete structures, by evaluating the present condition of structures from a proposed condition rating system. Further investigation will be required to merge the Rand Water grading system and the one utilised on this service life model, so that a more applicable model can be developed for the Rand Water network of reservoirs.

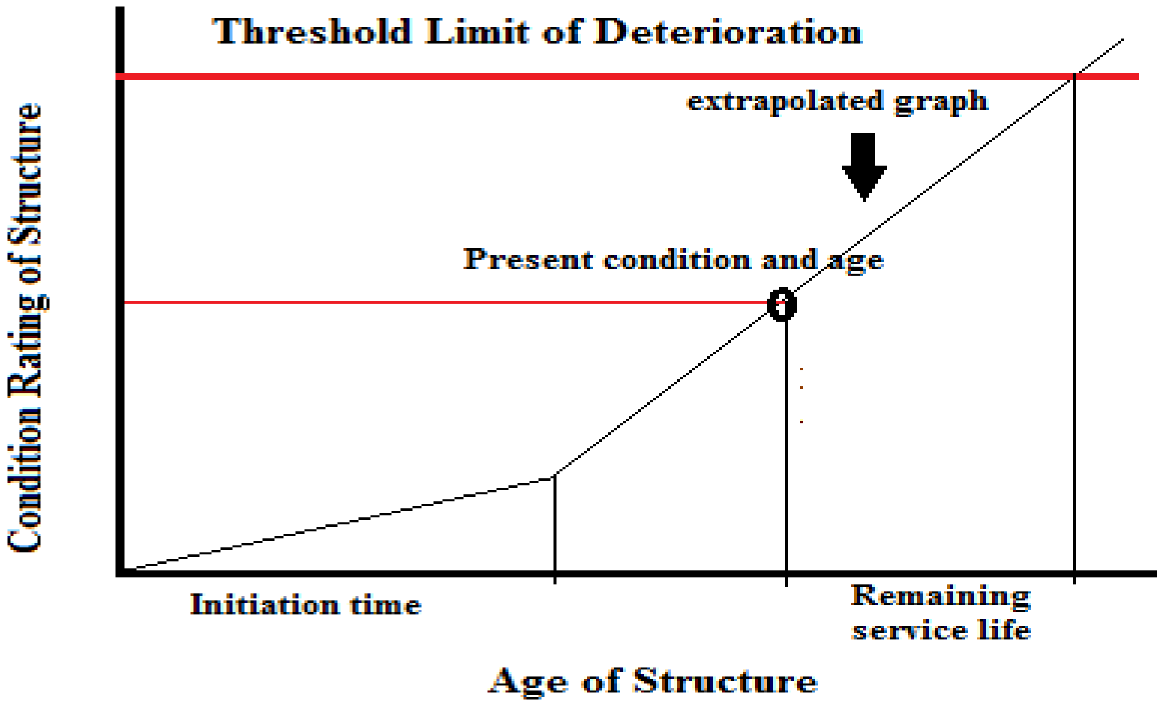


Figure 5.1: Model for predicting remaining service life (Verma et al., 2013)

5.1.2 Practical implications of the findings

5.1.2.1 Maintenance and repair of the reservoirs

There are five maintenance and repair strategies that must be considered in order to address the deterioration of reinforced concrete structures and restore structural integrity where required. These are: a) Do-nothing approach; b) Apply surface treatments; c) Reconstruction of damaged areas; d) Cathodic protection and prevent corrosion; and e) Complete replacement of damaged section. These strategies can be implemented concurrently, where possible. The following strategies need to be implemented on a case by case basis within the Rand Water network of reservoirs.

- **Do nothing**

Reservoirs have to be inspected in compliance with the Health and Safety Regulations, and those registered as dams also have to comply with the Dam Safety Regulations. The inspections schedule will then have to be implemented on an annual basis, as opposed to the regular five-year inspection schedule as required by the Dam Safety Regulations. However, additional requirements may be necessary, to ensure the safety of people and the environment. This is applicable to all reservoirs in the network that are between 60 and 80, and over 80 years old.

- **Application of surface treatments to the concrete**

Surface coatings and treatments can be used to help reduce the penetrability of the concrete and thereby limit, or at least slow, the rate of penetration of aggressive substances (including moisture and oxygen (Scott, 1997). This is the recommended remedial measure to be considered for most non-structural defects observed. This can be used in conjunction with the reconstruction of damage areas methodology for structural defects.

Surface treatments have been utilised in the rehabilitation of reinforced concrete structures in the Rand Water network. Care must be taken, however, to always comply with the water quality standards and potable water limits.

- **Reconstruction of damaged areas**

This method of repair is recommended only for structural members like beams and columns that have observed defects like cracking and spalling, where repair methods of concrete jacketing can be implemented.

- **Cathodic protection and corrosion prevention**

In most instances, cathodic protection is applied as a preventative strategy. Considering the size of the reservoirs within the Rand Water network, the sacrificial anodes system will not be feasible. The impressed voltages system will be more appropriate; however, for maximum benefit it needs to be implemented together with surface treatment. The implementation needs to be considered depending on the severity of the exposure environment.

The method of repair involves the removal of all contaminated concrete, cleaning the reinforcement, and subsequently patching of the damaged area. This, however, may not be feasible for all Rand Water reservoirs, as cathodic protection requires electricity. The electrical infrastructure can be solar panels and/or batteries, although these come with a high risk of vandalism. The impressed voltages system is also a maintenance intensive system. Most of the reservoirs are scattered all over Gauteng, in areas not accessible for electrical infrastructure, but they also have no security in place.

5.1.2.2 Need for a performance-based concrete durability design approach

Current specifications for concrete durability in both South African Standards and European Standards follow the prescriptive type, but the behaviour of structures, including Rand Water reservoirs, suggests that these are not effective. In this case there is no observed strategy in place to ensure longevity and sustainability of structures. New specifications should then be inclusive of relationships between mix design parameters, compressive strength and other durability properties.

Development of a new methodology for design, specification and constructing approaches for all Rand Water reservoirs is therefore required. This approach will obviously increase the initial capital cost of these structures but the long-term savings on maintenance and repair will be exponential.

In order to develop useful and meaningful tools for mix design using Durability Indices, it is crucial to be aware of numerous parameters and their effects on mechanical and durability properties of the concrete. Durability Index test results are usually affected by aspects such as exposure conditions, site practices and material properties of the sample.

The new design approach and methodology should be based on the premises governing durability and incorporating Durability Indices; these indices are a powerful tool for performance-based RC durability design. They can be improved by finding correlations between the predicted uncracked concrete performance and actual long-term performance of the cracked reinforced concrete structure.

5.2 Recommendations for further research

- This study was limited to the exterior in terms of conducting detailed assessments. The study was limited to surface concrete only, and therefore to understand the behaviour and the durability performance of the water reservoirs, a comparison between the interior and exterior parameters would have been advantageous.
- Research into the different service life models for carbonation and corrosion more suitable to Rand Water reservoirs.
- Further studies into the appropriate coating system to be applied inside water reservoirs that comply with both chemical requirements and potable water standards, while providing protection to the concrete to counter aggressivity.
- Further studies into the effects temperature loading may have on the deterioration mechanisms i.e. spalling
- Further investigation to ascertain if physical deterioration like wetting and drying is experienced in most of the reservoirs in the network.

REFERENCES

ASM International. (2000). Corrosion: Understanding the basics.

Alexander, M.G. and Otieno M. (2017). Revised Durability Index Testing Procedure Manual – ver. 4.2. Department of Civil Engineering University of Cape Town and University of the Witwatersrand, South Africa.

Alexander, M.G., Streicher, P. and Mackechnie, J. (1999). Rapid Conductivity Testing of Concrete. Department of Civil Engineering University of Cape Town.

Alexander, M.G. and Beushausen, H. (2008). The South African Durability Index tests in an international comparison. *Journal of the South African Institution of Civil Engineering*.

Alexander, M.G. (2018). Durability and service life prediction for concrete structures-developments and challenges. CMSS-Matec Web of conferences 149, 01006.

Alexander, M.G. and Mackechnie, J.R. (2000). Concrete options for water retaining structures. *Concrete Beton, Volume 96, pp 9-11*.

Alexander, M.G. and Stromeier, J.H. (1996). Deterioration, repair and maintenance of reinforced structures in the Cape Peninsula-Part 1. *Concrete Beton, Volume 81, pp 8-12; 14-15; 18-21*.

Angelucci, M. (2013). The influence of mix design parameters and compressive strength on durability indices. Department of Civil Engineering University of Cape Town Masters Thesis.

Ballim, Y. (1991). A low cost falling head permeameter for measuring concrete gas permeability, *Southern Africa 61*, pp.13–18.

Ballim, Y. (1994). Curing and the Durability of Concrete. PhD Thesis submitted to the University of Witwatersrand. Johannesburg.

Ballim, Y. and Blight G.E. (1993). Report on an investigation into the extent of deterioration of concrete at the Aeroton reservoir. Department of Civil Engineering, University of the Witwatersrand. Johannesburg.

Ballim, Y. and Basson, J. (2001). Durability of concrete. Fulton concrete technology 8th edition, Chapter 9, pp 135- 163.

Ballim, Y., Alexander, M.G. and Beushausen, H. (2009). Durability of concrete. Fulton concrete technology 9th edition, Chapter 9, pp 155- 188.

Barnardo-Viljoen, C., Mensah, K.K., Wium, J.A. and Van Zijl, G.P.A.G. (2014). Background to the draft SA National Standard for the design of water retaining structures. *Concrete Beton, Volume 138, pp 10-19.* Basson, J.J. (1989). Deterioration of concrete in aggressive waters. Midrand: Portland Cement Institute.

Beushausen, H. (2016). Repair and rehabilitation of concrete structures. University of Cape Town Lecture Notes. Cape Town.

Bhaskar, S., Srinivasan, P. and Chellappan, A. (2006). Condition assessment of 30-year old overhead RCC reservoir. National Seminar on Non-destructive evaluation. Hyderabad, India.

BSI. BS8007:1987. (1987). Code of practice for design of concrete structures for aqueous liquids.

Blight, G.E. and Lampacher, B.J. (1998). Permeability and sorption properties of mature near - surface concrete, *ASCE Journal of Materials Civil Engineering 10*(1).

Chandramouli, S.V., Sarawathi, S. and Pothnis, S.R. (2009). Assessment of relative surface hardness of existing concrete member by Schmidt hammer*. NBM & CW: Infra Construction & equipment magazine*. India.

Dias, W.P.S. (2000). Reduction of concrete sorptivity with age through carbonation. University of Moratuwa: Sri Lanka.

Dhir, R.K., Jones, M.R. and McCarthy, M.J. (1992). Pulverised-fuel ash concrete: Carbonatiom induced corrosion rates. *Proceedings of the Institution of Civil Engineers, 94(3).* ISSN 0965-0911.

EN 1992-1-1. (2004) *Eurocode 2:* Design of concrete structures.

Grieve, G. (2001). Aggregates for concrete. Fulton concrete technology, Chapter 3, pp 25- 60.

Hooton, R.D. and Brickley, J.A. (2013). Design for durability: Key to improving concrete sustainability. University of Toronto.

Kramer, W. and Martin, W. and Viljoen, W. (2018). Demolition of old oak bridge B4113: Condition of a 54 year-old prestressed concrete bridge. SMEC Consulting Engineers, *ICCRRR2018 Conference paper*. Cape Town.

Lampacher, B.J. (2000). Assessing the durability of concrete structures. University of The Witwatersrand: PHD Thesis. Johannesburg.

Mackechnie, J.R. (2001). Research monogram No 1: Predictions of reinforced concrete durability in the marine environment. Department of Civil Engineering University of Cape Town.

Matthews, S. and Mancini, G. (2018). Forensic engineering-fib MC 2020 and existing structures. Building Research Establishment. Watford.

McLeod, C.H. (2013). Investigation into cracking of reinforced concrete water retaining structures. University of Stellenbosch: Masters Thesis. Stellenbosch.

Mukadam, Z., Alexander, M.G. and Beushausen, H.D. (2016). The effect of drying preconditioning on the South African durability index tests. *Cement and Concrete Composites Publication.*

National Water Act. (1998). Regulations regarding the safety of dams in terms of section 123(1) of the national water act. Department of Water Affairs No R.139

Nemati, K.M. (2015). Durability of Concrete. University of Washington Concrete Technology Lecture Notes. Washington.

Neville, A. (1999). Concrete cover to reinforcement – or cover up. *Concrete Beton. Volume 95 pp 13-18.*

Nogueira, G. and Leonel, E.D. (2013). Probabilistic models applied to safety assessment of reinforced concrete structures subjected to chloride ingress, Engineering Failure Analysis, 31, pp. 76–89 http://dx.doi.org/10.1016/j.engfailanal.2013.01.023

NTIS. (1997). Carbonation in Concrete and Effect on Steel Corrosion. U.S. Department of Commerce publication PB 98-106735.

Nganga, G., Alexander, M.G. and Beushausen, H.D. (2013). Practical implementation of the durability index performance-based design approach, *Construction and Building Materials Journal*. University of Cape Town. Cape Town

Nganga, G., Alexander, M.G. and Beushausen, H.D. (2017). Practical implications of the durability index performance-based specifications. *Concrete Beton. Volume 150 pp 18-22.*

Oberholster, B. (2001). Alkali-silica reaction. Fulton Concrete technology, chapter 10 pp. 163-189.

Occupational Health and Safety Act. (2014). Construction Regulations. Department of Labour No R.647.

Otieno, M. (2008). Corrosion propagation in cracked and uncracked concrete. University of Cape Town MSc Thesis. Cape Town.

Otieno, M. (2010). Assessment techniques for corrosion in reinforced concrete structures. University of Cape Town. Cape Town.

Otieno, M., Alexander M.G. and Du Plessis, J. (2017). Soft water attack on concrete tunnel linings in the Ingula pumped storage hydro-power scheme: Assessment of concrete resistance and protection. *Journal of the South African Institution of Civil Engineering*. *Vol 59 No 3 pp 57-67, Paper 1723*.

Owens, G. and Addis, B. (2001). Fulton's Concrete Technology, 8th Edition. Midrand: Cement and Concrete Institute, Midrand.

Owens, G. (2009). Fulton's Concrete Technology, 9th Edition. Midrand: Cement and Concrete Institute, Midrand.

Page, C.L. and Page, M.M. Durability of concrete and cement composites. Technical Books Pdf. https://www.technicalbookspdf.com/durability-of-concrete-and-cement-composites-by-c-l-page-and-m-m-page/

Papadakis, V.G. (1999). Effect of supplementary cementing materials on concrete resistance against carbonation and chloride ingress. *Cement and Concrete Research*, *30* pp. 291-299.

Pakkala, A.T., Kolio, A. Lahdensivu, M. and Lah, K. (2014). Durability demands related to carbonation induced corrosion for Finnish concrete buildings in changing climate. Tampare University of Technology, Finland. *Construction and Building Materials Journal*.

Richardson, M. (2002). Fundamentals of Reinforced Durable Concrete. London*: British Library Catalogue.*

RILEM Technical Committee. Report on Durability Design of concrete structures.

Rodriguez, J., Izquierdo, D., Ortega, L.M. and Andrade, C. (2006). Calculation of structural degradation due to corrosion of reinforcements. Measuring, monitoring and modelling concrete properties. Conference paper pp. 527-536. DOI: 10.1007/978-1-4020-5104-3_64.

Ronne, P.D. (2005). Variation in cover to reinforcement: local and international trends. *Concrete Beton. Volume 111 pp 7-13.*

SANS 5865: 1994, The drilling, preparation and testing for compressive strength of cores taken from hardened concrete, South African Bureau of Standards, Pretoria.

SANS 10100 Part 2: 2014, Structural use of concrete, materials and execution of work, South African Bureau of Standard, Pretoria.

Scott, A.N. (1997). Designing for durability in reinforced concrete structures, and associated economics. University of Cape Town MSc Thesis. Cape Town.

Scott, A.N. (2004). Influence of binder type and cracking on reinforcing steel corrosion in concrete. University of Cape Town PHD Thesis. Cape Town.

StatsSA. (2019). *Statistical release P0302-midyear population estimates*. Pretoria.

Song, H. and Saraswathy, V. (2007). Corrosion monitoring of reinforced structures-A review. *International Journal of Electrochemical Science*.

Szilagyi, K., Borosnyoi, A. and Zsigovics, I. (2011). Surface hardness and related properties of concrete. Budapest University of Technology. Budapest.

Taffese, W.Z. and Sistonen, E. (2017). Machine learning for durability and service-life assessment of reinforced concrete structures: Recent advances and future directions, vol. 77, pp.1–14. http://doi.org/10.1016/j.autcon.2017.01.016.

TRID the TRIS and ITRD database (2010). The impact of tropical urban environment on the durability of RC in Iberoamerican countries. https://trid.trb.org/view/1141145

Tragardh, J. (1999). Microstructural features and related properties of self-compacting concrete. Swedish Cement and Concrete Institute. Sweden. Rilem Publications.

Verma, S.K. and Bhadauria, S.S. and Akthar, S. (2013). Estimating residual service life of deteriorated reinforced concrete structures. *American Journal of Civil Engineering and Architecture, 2013, 1(*5) pp. 92-96.

APPENDIX A -EXPERIMENTAL RESULTS

A1 Visual assessment laboratory summary

Table A 1: Summary of laboratory visual assessment for all reservoirs

Reservoir	Core label	Diameter (mm)			Core length (mm)	Voids (%)	Aggregate			Assessment for corrosion, delamination and other observations
		d_1	d_2	Ave			Size	Shape	distribution	
Brakfontein	H	104.3	104.9	104.6	145	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion
	M	103.2	103.3	103.3	155	0.4	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion
	R	104.6	104.2	104.4	150	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion
	J	104.3	104.4	104.4	160	0.3	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion
	O	104.5	104.6	104.6	155	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion
	V	103.4	104.5	104.0	150	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion
Forest Hill	FH D	105.0	104.2	104.6	165	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Surface painted
	FH E	104.5	104.4	104.5	160	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Surface painted.
	FH N	104.3	104.4	104.4	145	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	FH L	104.1	105.3	104.7	145	0.6	40	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Bituminous surface coating.
	FH G	104.6	104.6	104.6	130	0.3	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Bituminous surface coating

Reservoir	Core label	Diameter (mm)			Core length (mm)	Voids (%)	Aggregate			Assessment for corrosion, delamination and other observations
		d_1	d_2	Ave			Size	Shape	distribution	
Sasolburg	Q100	104.3	104.6	104.4	150	1.0	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	GH100	103.2	103.3	104.6	140	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	E100	104.6	104.2	103.1	145	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	S100	104.3	104.4	104.4	150	0.3	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	G100	104.5	104.6	104.4	185	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	K100	103.4	104.5	104.2	145	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Presence of drying shrinkage cracks observed.
	D68	65.0	64.4	64.7	125	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	V68	65.7	66.1	65.9	160	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	U68	65.7	65.7	65.7	155	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	UU68	64.6	64.6	64.6	130	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	D 68 8-9	65.1	66.1	65.6	140	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Excess fines observed.
	D 168	64.9	64.6	64.8	133	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion
	IJ 68	64.5	64.6	64.6	140	0.6	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Presence of drying shrinkage cracks observed.
	I68	65.2	65.0	65.1	155	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.

	CD 68	65.5	65.5	65.5	120	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Specimen was broken 30mm from the top.
	J68	64.7	65.0	64.9	120	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Excess fines observed
	S2	64.5	64.6	64.6	115	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	S1	64.5	64.6	64.6	115	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion.
	P 68	65.2	65.5	65.4	90	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Excess fines observed.
	C68	65.3	65.2	65.3	145	0.5	26.5	Angular	Uniform	No signs of delamination, no reinforcement present for assessment of corrosion. Specimen was broken 70-80mm from the top.

A2 Surface hardness

Table A 2: Surface hardness results (Schmidt hammer)

Reservoir	Specimen No.	Rebound index			
		Actual readings	Min	Max	Ave (std.dev)
Brakfontein	1	25, 28, 32	21	32	27 (3.9)
	2	21, 30, 24			
	3	22, 31, 26			
Sasolburg	1	15, 19, 17	15	36	22 (4,6)
	2	27, 25, 19			
	3	27, 36*, 23			
Forest Hill	1	35, 30, 38	20	38	34 (3.4)
	2	20*, 22*, 33			

***** *considered as outliers and omitted in the analysis*

A3 Carbonation depth

Table A 3: Carbonation depth measurement results

Reservoir	Core label	Carbonation depth, x (mm)			
		x1	**x2**	**x3**	**x4**
Brakfontein	O	5.1	6.0	8.9	7.6
	V	17.2	18.6	17.4	15.1
Sasolburg	G100	17.5	14.0	13.3	17.7
	K100	21.3	15.4	20.0	20.8
Forest Hill	FH L	32.7	31.4	23.6	26.2
	FH G	7.0	5.0	7.0	5.7

A4 Compressive strength

Table A 4: Compressive strength results

Reservoir	Core label	Diameter, d (mm)	Length, l (mm)	d/l ratio	Saturated mass (g)	Density (kg/m^3)	Failure load (KN)	Failure mode	Measured strength (MPa)	Corrected factors		Equivalent cube strength (MPa)	Correction factors for excess voids factor, f_v	Potential cube strength (MPa)
										For d/l ratio	For steel			
Brakfontein	R	104.4	104.6	1.00	2.221	2480	340.4	Satisfactory	**39.8**	1.00	-	**39.8**	1.04	**41.4**
	M	103.3	102.6	1.01	2.150	2500	188.4	Satisfactory	**22.5**	1.00	-	**22.5**	1.03	**23.1**
	J	104.4	104.3	1.00	2.200	2464	300.0	Satisfactory	**35.0**	1.00	-	**35.0**	1.02	**35.8**
	H	104.6	101.4	1.03	2.218	2545	401.4	Satisfactory	**46.7**	0.99	-	**46.1**	1.04	**48.0**
Sasolburg	E100	103.1	104.6	0.99	2.221	2543	322.8	Satisfactory	**38.7**	1.01	-	**38.9**	1.04	**40.5**
	Q100	104.4	103.3	1.01	2.101	2376	368.0	Satisfactory	**43.0**	1.00	-	**42.8**	1.08	**46.4**
	S100	104.4	102.1	1.02	2.120	2425	317.6	Satisfactory	**37.1**	0.99	-	**36.8**	1.02	**37.6**
	GH100	104.6	103.4	1.01	2.136	2404	342.0	Satisfactory	**39.8**	1.00	-	**39.8**	1.04	**41.2**
Forest Hill	N	104.4	101.5	1.03	2.038	2345	211.6	Satisfactory	**24.7**	0.99	-	**24.4**	1.04	**25.4**
	E	104.5	107.5	0.97	2.227	2415	246.8	Satisfactory	**28.8**	1.01	-	**29.1**	1.04	**30.3**
	D	104.6	108.5	0.96	2.232	2394	250.4	Satisfactory	**29.1**	1.01	-	**29.6**	1.04	**30.8**

A5 Durability Indices

Table A 5: Durability index tests results

Reservoir	Specimen No	CCI (mS/cm)	OPI (-Log k)	WSI (mm/ √ hr)	Porosity (%)	
					n_a	n_b
Brakfontein	1	0.58	8.56	8.94	10.54	4.15
	2	0.89	9.21	10.24	12.25	2.85
	3	1.23	8.79	11.58	9.58	3.58
	4	0.97	9.06	9.45	11.43	2.54
Sasolburg	1	1.02	8.45	12.79	10.35	3.15
	2	1.24	9.43	12.72	9.10	4.01
	3	0.95	8.02	12.57	12.18	3.45
	4	1.11	9.11	11.35	9.12	2.45
Forest Hill	1	0.95	8.67	9.92	13.72	3.86
	2	0.99	9.26	7.90	11.00	3.81
	3	1.52	9.34	7.05	11.53	4.82
	4	1.62	8.87	3.16	10.70	4.62

* *k*: Coefficient of gas (oxygen) permeability (m/s); n_a: from WSI test; n_b: from CCI test

A6 Aggressivity Analyses

Table A 6: Aggressivity index tests results (Basson indices) for all reservoirs

Years		Reservoirs (AI)		
		Forest Hill	**Sasolburg**	**Brakfontein**
2014	January - June	893.16	847.83	821.72
	July - December	881.63	781.10	762.66
2015	January - June	898.74	790.51	697.04
	July - December	725.12	709.03	644.48
2016	January - June	822.13	754.35	720.17
	July - December	830.16	759.61	741.20
2017	January - June	995.63	778.18	866.01
	July - December	1066.79	966.67	922.90
2018	January - June	1030.36	924.98	778.34
	July - December	888.84	747.11	807.85
2019	January - June	928.01	832.85	795.31
	July - December	936.55	804.74	739.21

Table A 7: Aggressivity index tests results (Basson indices) for Brakfontein reservoir over a period of 10 years

Years		Aggressivity index (AI)
2009	January - June	600.18
	July -December	539.66
2014	January -June	823.47
	July - December	804.11
2019	January -June	815.70
	July - December	759.02

Table A 8: Temperature-corrected average aggressivity index results for all reservoirs

Years		Reservoirs (AI)		
		Forest Hill	**Sasolburg**	**Brakfontein**
2014	January - June	877.86	718.81	833.45
	July - December	673.38	625.04	600.39
2015	January - June	908.05	774.27	710.07
	July - December	813.62	605.31	546.08
2016	January - June	837.87	756.04	738.77
	July - December	698.94	653.91	636.35
2017	January - June	979.79	747.90	869.99
	July - December	872.12	805.47	765.40
2018	January - June	1033.92	899.98	782.21
	July - December	735.57	637.40	665.88
2019	January - June	946.15	826.8	800.06
	July - December	788.91	686,57	632.74