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POTENTIAL DURABILITY AND SERVICE LIFE PREDICTION OF REINFORCED CONCRETE BRIDGES ON THE SWAKOPMUND-WALVIS BAY FREEWAY UPGRADE

Kevin John Walter

A dissertation submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering

Johannesburg, 2020

DECLARATION

I declare that this dissertation is my own unaided work. It is being submitted for the partial fulfilment of 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 other University.



…….

Signature of Candidate

_____13th____day of _____October_____ year _____2020_____

ABSTRACT

Deterioration of concrete structures in the marine environment is a major concern for engineers and asset managers. This dissertation presents a case study involving the testing and assessment of concrete in newly constructed reinforced concrete bridges on the Swakopmund–Walvis Bay Freeway upgrade in Namibia, designed with the expectation of a maintenance-free service life of at least 100 years. The project specifications followed a prescriptive durability design and specification approach, setting restrictions and limits on concrete-related aspects such as binder type and content, maximum water-to-cement ratio, placement, compaction, and curing.

Testing was carried out for compressive strength, durability indexes (DI) and cover depth, and compared with performance-based specifications used by SANRAL. The measured DI values were found to be satisfactory, exceeding the minimum specifications in the 'deemed-to-satisfy' approach for concrete in a marine environment. The maintenance-free service life of the as-built structural bridge elements exposed to chlorides was predicted using the South African chloride ingress model. With this approach, 28% of bridge elements show areas of early local failure with potential service life as low as 56 years, and 11% of the bridge elements have a potential wider area of failure before the 100-year design life. The lower service life is attributed to inadequate cover to concrete. It is recommended that additional waterproofing and coatings be applied to these areas in order to achieve protection equivalent to concrete cover.

To my wife, Cassandra, and children, Taelor, Dylan, and Morgen, for their patience with me during the last couple of years. To my mom and dad for their sacrifices made to give me a privileged upbringing, and most importantly, to the Lord for the continued blessings in my life.

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LIST OF ABBREVIATIONS

ASTM	American Society for Testing and Materials
BS	British Standard
CCI	Chloride Conductivity Index
CEB	Comite Euro-Internacional du Beton
COLTO	Committee Of Land Transport Officials
COV	Coefficient of Variance
CSF	Condensed Silica Fume
DI	Durability Index
DLS	Durability Limit State
FA	Fly Ash
GGBS	Ground Granulated Blast furnace Slag
ISO	International Standards Organisation
ITZ	Interfacial Transition Zone
OPC	Ordinary Portland Cement
OPI	Oxygen Permeability Index
SABS	South African Bureau of Standards
SANRAL	South African National Roads Agency Limited
SANS	South African National Standard
SF	Silica Fume
SL	Slag
SLS	Serviceability Limit State
TMH	Technical Methods for Highways
TPT	Torrent Permeability Test
ULS	Ultimate Limit State
WSI	Water Sorptivity Index

1 INTRODUCTION

1.1 BACKGROUND

Reinforced concrete construction in Southern Africa has increased substantially in the last few decades. Prominent in the design and construction of new structures is ensuring that the structure meets the required design life or service life. Many concrete structures fail to fulfil their intended service life, resulting in substantial expenses for the owners of structures for maintenance and rehabilitation (Ballim, et al., 2009).

Early deterioration of concrete structures is a major concern for engineers and asset managers. Research into concrete durability has increased the understanding of complex interactions between concrete and the environment that cause deterioration. The leading causes of durability issues with concrete are a result of corrosion of reinforcement caused by carbonation or the ingress of chlorides (Otieno, et al., 2010).

Reinforced concrete structures in the marine environment are most susceptible to chloride-induced corrosion of reinforcement due to the presence of high chloride concentrations and humid or saturated conditions (Mackechnie, 2001). Corrosion is manifested in cracking and spalling of cover concrete due to the formation of expansive corrosion products at the reinforcement, and local pitting at the anode which reduces the cross-sectional area of the bar. Investigations have indicated the durability performance of reinforced concrete structures in the marine environment to be generally unsatisfactory (Mackechnie, 2001).

There is, therefore, great value and incentive in extending the service life of new structures through a more detailed durability design approach. Many modern approaches to the design of concrete structures include durability requirements as well as strength requirements (Ballim, et al., 2009).

1.2 SWAKOPMUND-WALVIS BAY FREEWAY UPGRADE

The study was carried out on concrete from newly constructed bridges on Phase 1 of the Swakopmund-Walvis Bay freeway upgrade in Namibia. The freeway is situated in the Erongo Region on Namibia's Atlantic West coast (Figure 1-1). It forms part of the inland route between Walvis Bay and Swakopmund as well as the Trans-Kalahari, Trans-Kunene, and Tran-Zambezi Highways. Ongoing and future expansions at the Port of Walvis Bay are

expected to considerably increase export and import of goods along these corridors. With the coastal road being notorious as a high accident zone, it is critical that a safer, more convenient road be constructed for the use of heavy vehicles.

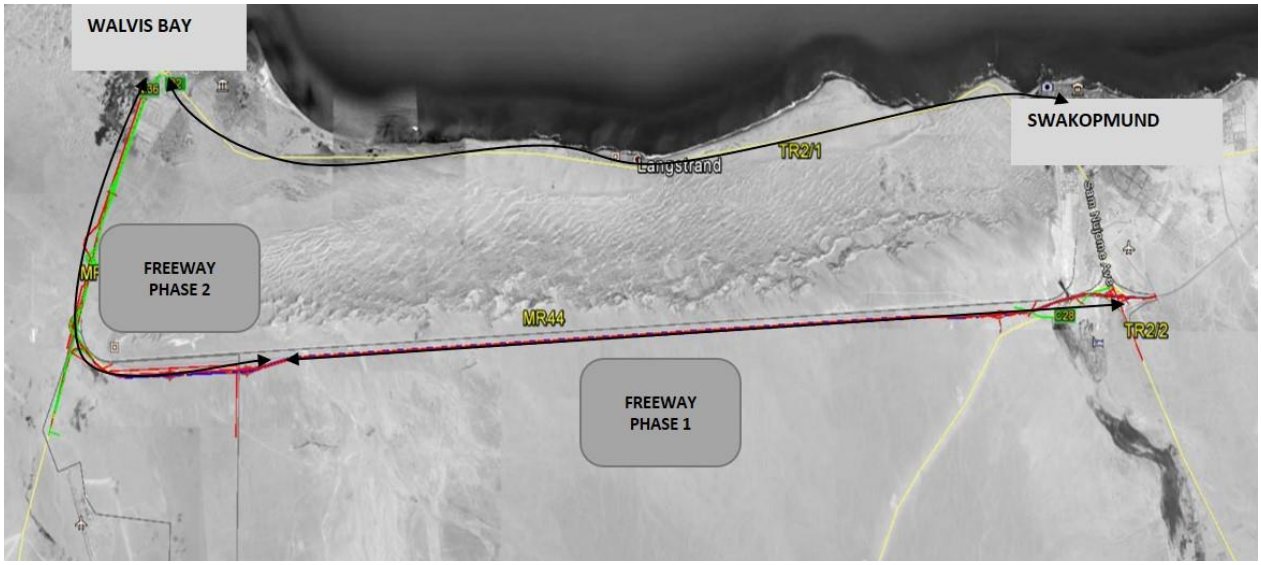


Figure 1-1 – Swakopmund-Walvis Bay Freeway Upgrade project location

The N$950 Million Namibian dollar (N$) project consists of four post-tensioned concrete bridges with overall deck lengths ranging between 56 and 109 m. The two bridges over the Swakop River were constructed with more than 2,500 m^3 of concrete each while the other two interchange bridges (MR52 Interchange and Swakopmund Interchange) exceed 1,700 m^3 of concrete each. The total cost of the four bridges was approximately N$110 Million.

The design lifespan of the concrete is 100 years (TMH7, 1981). The project specifications followed a prescriptive durability design and specification approach, setting restrictions and limits on concrete-related aspects such as binder type and content, maximum water-to-cement ratio, placement, compaction, and curing. Although it was accepted that a premium be paid for the prescribed mix design and the sourcing of blast-furnace slag, etc to improve concrete quality, this acceptance came with expectations on service life and maintenance requirements.

The principle concern with this approach to durability design is that the durability aspects of the as-built concrete are not directly measured but assumed to be proportional to the concrete strength. Of importance in this research topic is to indicate the importance of measuring durability aspects of concrete and the possible usage of these determinations in the calculation of potential service life of structures.

1.3 PROBLEM STATEMENT

For new structures, the service life is defined as “the time which a concrete fulfils its performance requirements, without non-intended maintenance” (Nilsson, 2003). Temporary structures and replaceable structural elements will have lower service lives (i.e. 1-25 years) while buildings and bridges may have higher service lives (i.e. 50-100 years). The service life requirements are either given by the owner of the structure or taken from codes of practice for the given structure (FIB, 2006). Several factors influence the ultimate durability of concrete. These include environmental conditions, constituent materials, mix design and construction practices.

Conventional structural design codes follow a prescriptive approach to durability and service life design, whereby the requirements to meet durability are specified for the contractor (SANS 10100-1, 2000; TMH7, 1981; BS 8110-1, 1997). This approach is limiting since there is difficult to measure or control the presumed durability or service life. There are no real determinations on the concrete structure’s resistance to the penetrability of harmful substances. This approach also assumes a direct correlation between compressive strength and durability since the acceptance of the as-built structure is based solely on strength requirements. Several researchers have shown this assumption to be invalid (Al-Amoudi, et al., 2009; Burrows & Mehta, 2001).

The South African Durability Index approach is widely used in South Africa, but several Consultants still use specifications that are based on the strength of concrete to assess potential durability. For new structures, basing the acceptance solely on concrete strength would reassure the Engineer that the structure will not fail when loaded within the limits of the structural design. This reassurance, however, will only be valid for as long as the concrete or reinforcing steel does not undergo additional physical or chemical deterioration. A performance-based approach similar to the specifications used by SANRAL is considered a more suitable method for or the major concrete works in this environment.

1.4 RESEARCH SIGNIFICANCE

The climate along the Namibian Coast is very conducive to chloride-induced corrosion of steel. Research has shown the Namibian Coast to be one of the most highly corrosive environments in Southern Africa (Bulley, 1986). A relevant case study on the corrosiveness of the Namibian Coast is the Omaruru River Bridge (1982) in Henties Bay. This bridge was constructed with 45 MPa concrete in the deck, and 50 mm cover. However, investigations

only eleven years after the construction of the bridge indicated severe chloride attack on the concrete and main reinforcing bars in the deck and piers. Repair work was undertaken on the spalled concrete and the main reinforcing bars needed to be replaced (Hess, 1996).

Long-term corrosion rates at the Walvis Bay military base (20 km from the coast) were recorded to be, on average, 0.85 mm/year for mild steel (Callaghan, 1991). This is substantially higher than recorded corrosion rates in Cape Town and Durban, measured in the same study. Several probable aspects of the coastal climate contribute to the harsh environment. These include the high relative humidity, the wetting-drying action of the fog, and the abundance of chlorides in the atmosphere (Bulley, 1986).

There has been an initiative in South Africa in recent years for the incorporation of durability index (DI) testing into the South African National Standards (Kessy, et al., 2015). This follows international trends in durable concrete design (Beushausen & Alexander, 2008). The South African National Roads Agency (SANRAL) has already incorporated DI testing and cover-meter readings into their specifications for major concrete bridges (SANRAL, 2011). However, there is still concern and resistance from industry as to how the durability specifications can be practically implemented (Nganga, 2011).

The South African Chloride Ingress model is a performance-based model for reinforced concrete in the marine environment. The model uses early age chloride diffusion results from the South African chloride conductivity test. The chloride conductivity values are then converted to long-term diffusion coefficients used in Crank's solution to Fick's second law of diffusion, with additional input for cover depth and exposure environment. Marine environments in this model are classified according to EN206-1 (2000), modified for South African conditions.

1.5 AIM AND OBJECTIVES OF RESEARCH

The aim of this research topic was to assess the potential durability and expected service life of four newly constructed reinforced concrete bridges on the Swakopmund-Walvis Bay Freeway Upgrade. This was done through a combination of concrete cube strength, durability index (DI) testing and electromagnetic covermeter readings. The research objectives consisted of the following:

a) Evaluation of concrete consistency through concrete compressive strength

Evidence has shown that concrete compressive strength cannot be directly correlated with potential durability (Al-Amoudi, et al., 2009; Burrows & Mehta, 2001). The comparison of compressive strength using the same mix over several months or years will, however, give insight into the consistency of the concrete mix which will enable a fairer assessment of the variation in potential durability with time.

b) Assessment of potential durability using DI tests

Durability Index (DI) tests were carried out as per the South African Durability Index Testing Manual (DI Testing Manual, 2018). The tests give an indication of the concrete's resistance to penetration of potential corrosive liquids, gases and ions. These tests can, therefore, be used to assess the potential durability of the as-built concrete. Depending on the expected environmental exposure of the concrete, tests results can be used to determine the concrete's potential lifespan as the time to corrosion activation of the reinforcing steel.

The tests consist of the Oxygen Permeability Index (OPI), Water Sorptivity, and Chloride Conductivity and are used to determine the concrete's resistance to permeation, sorptivity and diffusion respectively. Core samples were taken from 150 mm test cubes and 600×400×150 test panels cast on-site adjacent to the structure. DI test results are compared to the 'deemed-to-satisfy' limits used by SANRAL in their performance-based approach to durability design.

c) Measurement of in-situ cover using electromagnetic covermeter

Cover to concrete is an essential element in the durability of steel-reinforced concrete elements. Reinforced concrete in the marine environment is particularly susceptible to chloride-induced corrosion of the reinforcing steel. The concrete cover acts as a protective layer against the ingress of harmful agents. A reduction of concrete cover has a direct effect on the potential lifespan of the structure.

An accurate determination of as-built concrete cover was crucial in the service life prediction of the bridge elements. All bridge decks, piers and abutments on the project were specified with a 50 mm cover to concrete. Concrete cover was checked before casting concrete for each element. After casting and curing, bridge piers and abutments were assessed for cover using an electromagnetic covermeter.

d) Determination of expected service life using the South African Chloride Ingress Model

The South African Chloride Ingress Model (Mackechnie, 2001) was used to determine the time-to-corrosion-activation and subsequent service life of the new bridges. The expected service life prediction is based on a 95% confidence band for chloride conductivity and cover-to-concrete from the test results of (b) and (c) above. The chloride conductivity values are converted to a diffusion coefficient based on empirical nomograms developed by Mackechnie (2001). Additional parameters used in this model are the surface concentration of chlorides (based on the exposure environment of the structure, and the chloride threshold, which has been researched and published by several researchers (Glass & Buenfield, 1997; Oh, et al., 2003; Castellote, et al., 2002; Breit, 1998; Mackechnie, 2001). Expected service life is determined for the various elements in the structure.

1.6 SCOPE AND LIMITATIONS

The investigative approach was to determine concrete quality and potential durability of the structures based on Durability Index (DI) testing, and cover readings using an electromagnetic covermeter. Thereafter, service life predictions were made using the South African Chloride Ingress Model developed by Mackechnie (2001). Although the assessment covers several durability aspects, there were some limitations as listed below:

a) Representative Panels

Specimens for DI testing were cored from sample panels cast alongside the structure. These panels were intended to represent the structural elements and cast with the same formwork type, degree of vibration, and curing conditions as the as the structures. However, the actual structure may have defects due to poor vibration, bleeding, or insufficient curing which would not be considered on the representative panels. Ronny (2011), carried out several tests comparing durability results of in-situ concrete with representative test panels and test cubes. Results from these tests indicated that DI results from test cubes cast on site and cured in the laboratory do not correlate well with DI results from the in-situ concrete, with test panels being more representative of in-situ construction (Ronny, 2011)

b) Limitations of DI Tests

Durability Index tests assess the transport properties of the cover concrete. The in-situ structural elements are, however, susceptible to cracking which is not accounted for in the testing. Although the design has been carried out to limit crack widths, there will inevitably

still be cracks visible on the structure due to the restraint of the members, structural loading, and thermal movements. The transport properties through cracked concrete are not covered in the service life predictions of the structural elements.

c) Cover depth test areas

Cover depth testing was carried out on randomly selected 1-m^2 test area representing a minimum of 30-m^2 of in-situ concrete structure on each structural face. It would be impractical to scan every bar on the entire structure for cover. There may be, however, individual locations with particularly low cover which were not scanned and would be highly susceptible to corrosion.

d) Assumptions in service life prediction model

The South African Chloride Ingress Model uses several variables for the prediction of time to corrosion activation in structures exposed to chlorides. These variables consist of the exposure environment, surface concentration of chlorides, chloride threshold level, concrete cover and diffusion coefficient. There are guidelines for the selection of some of these variables. There can be no certainty, however, that the actual exposure conditions match the assumed conditions. The Model's sensitivity is assessed in the discussion on results, but the final prediction model requires that some of these parameters are kept constant.

e) Effect of surface protection

The project specifications required protective coating to be applied to all structural elements to prevent the ingress of liquids into the pore structure of the concrete. All concrete faces in contact with soil were coated with a bitumen-based protective coating. All above-ground concrete elements, exposed to chlorides, were coated with a silane-based water-repellent impregnation cream, and the top surfaces of bridge decks were coated with a waterproof spray-on polyurea. Research shows that coating and waterproofing can extend the time to corrosion activation to anything between 15 and 25 years (Christodoulou, et al., 2012; EOTA, 2010). Service life predictions in this study consider the concrete properties only and do not include this additional protection from the surface coatings.

1.7 LAYOUT OF THESIS

The thesis consists of five chapters:

The first chapter is an introduction to the topic. The chapter consists of details on durability of concrete and important aspects related to durability design and service life prediction.

The second chapter consists of a literature review considering aspects of durability of concrete and factors affecting concrete durability and penetrability. An important aspect is the corrosion of steel in concrete and how this is induced. Details are given on available test methods on cover concrete with a specific focus on the South African Durability Index (DI) test methods. There are further discussions on the durability design of concrete structures and the modern approaches to ensuring durable concrete structures with a focus on service life models and the prediction of concrete service life of structures.

The third chapter is on the methodology used in determining potential durability and service life of the concrete bridges considered in this study. The first part of chapter three consists of a detailed background on the project which includes design considerations, the chosen approach to durability design, concrete mix design aspects and construction practices.

The fourth chapter is a statistical analysis on the durability parameters obtained on the project, consisting of concrete strength, cover readings and durability indexes. Also included in this chapter is a prediction on the potential service life of the concrete structures using the South African Chloride Ingress Model.

The fifth chapter presents the conclusions made regarding the potential durability and predicted service life of the concrete structures. Recommendations are given for the improvement of the DI test methods as well as suggestions on approaches to service life prediction.

2 LITERATURE REVIEW

2.1 INTRODUCTION

The literature review covers essential aspects of concrete durability. The first section includes the environmental, extrinsic and intrinsic factors that affect the durability of concrete. This section is followed by details on transport properties of the concrete pore structure. Details are given on typical durability index tests and the use of the parameters obtained from the tests in assessing the potential durability of concrete followed by the corrosion of steel in concrete with an emphasis on concrete in the marine environment exposed to chlorides. The chapter concludes with a discussion on durability design and service life predictions of concrete structures.

2.2 DURABILITY OF CONCRETE

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" (ASTM E632-82, 1988). In Figure 2-1, factors listed under concrete system deal with the concrete's ability to resist mechanisms of deterioration. Factors listed under environment are the factors responsible for the degree of aggressiveness that the concrete will be able to withstand (Ballim, et al., 2009). As shown, several factors influence the ultimate durability of concrete. These include environmental conditions, constituent materials, mix design and construction practices.

2.2.1 Aggressiveness of Environment

Ultimately, the exposure environment will significantly affect the concrete's durability and potential lifespan. Different types of concrete deterioration may be described by the nature of the attack, whether it is external or internal, and in what environments the attack will occur. The fundamental nature of deterioration is mainly of three types: chemical, physical, or electrochemical, the latter concerning reinforcement corrosion which is addressed in Section 2.3 (Ballim, et al., 2009).

Chemical attack involves the dissolution of chemicals within the concrete pore structure or the reaction between substances and components within the concrete. The reaction may cause problems due to the concrete breaking down or the reaction products causing expansion cracks. There are numerous sources of chemical attack which will not be addressed in detail

in this report. These include acid attack, sulphate attack, alkali-aggregate reactions, carbonation, and soft water attack. (Choo & Newman, 2003). Physical attack involves the concrete's exposure to non-reactive liquids or heat, which cause stresses and cracking within the concrete. These include fire, frost attack, weathering and erosion, and abrasion.

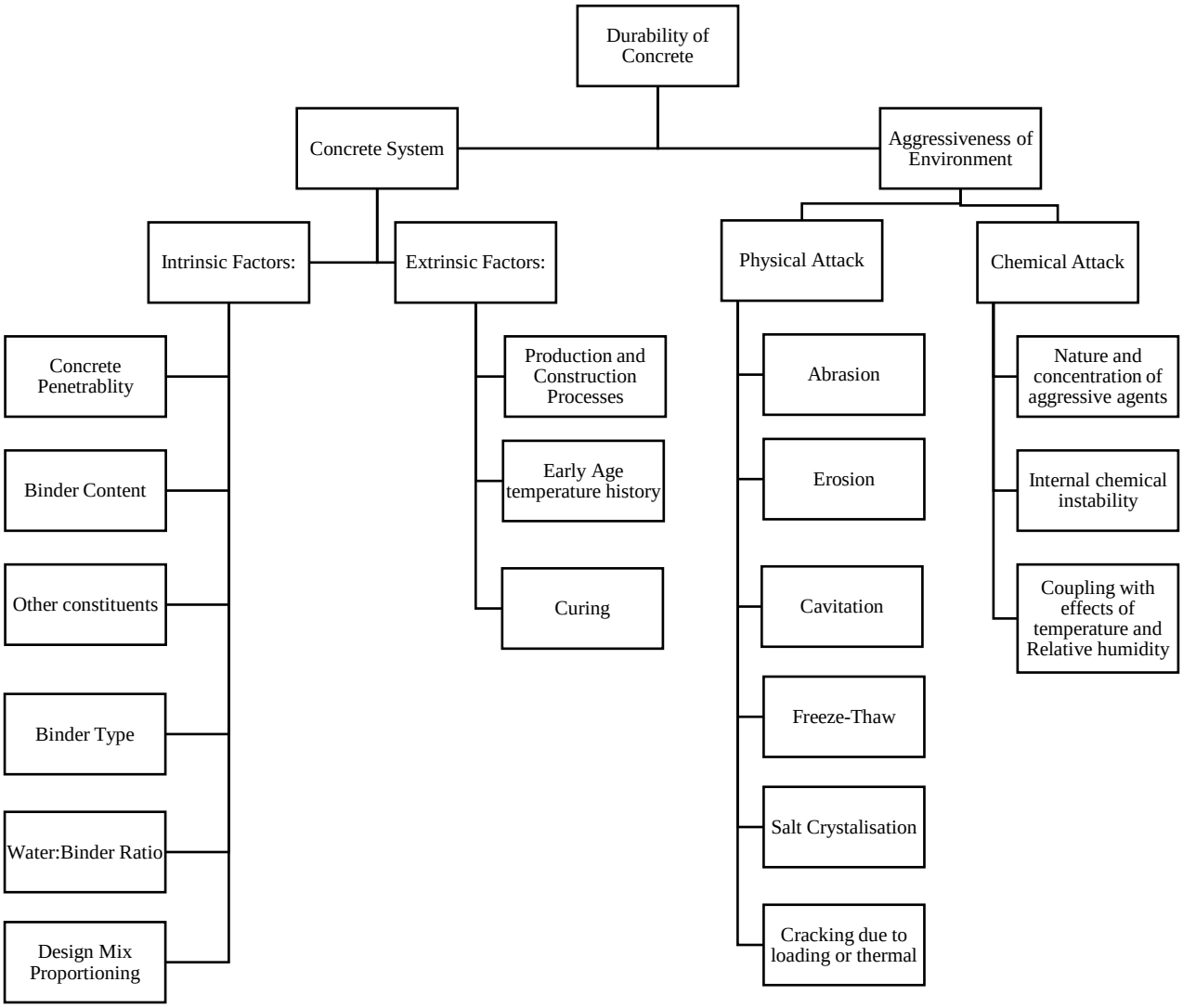


Figure 2-1 - Concrete and environment: Factors influencing the durability of concrete (Ballim, et al., 2009)

2.2.2 The Concrete System

2.2.2.1 Concrete Microstructure

Concrete is a composite material consisting of a paste phase and an aggregate phase. This results in an additional phase referred to as the interfacial transition zone (ITZ). This zone has a higher porosity and penetrability than the bulk paste and, therefore, provides a path for the movement of fluids through the concrete (Ballim, et al., 2009).

Pores within the concrete structure can significantly affect the concrete's durability depending on the type of pore. Pores are generally defined as macropores, capillary pores, and gel pores (CEB, 1992):

Macropores:	Visible to the naked eye and caused by entrapped air within the concrete
Capillary pores:	Residue from original water-filled spaces within the fresh paste
Gel pores:	Form within the cement paste and are on the nano-scale size

Capillary pores are critical to the transport properties of concrete since the water is free to move within the pore structure, and the pores allow pathways for movement. The size and interconnectivity of the pores within the concrete microstructure have a direct effect on the transport properties of the concrete (i.e. the ease at which fluids move through the concrete). The pores are connected by varying amounts through the concrete microstructure, with the highest level of interconnectivity being at the ITZ where the concrete is most porous. There are numerous reasons for the porosity of the ITZ, but most common theories are related to the accumulation of bleed water around the coarse aggregate particles (Mukadam, 2014).

There are several factors within the concrete system that affect the concrete microstructure and subsequent potential durability of the concrete structure. These factors can be broken down into intrinsic and extrinsic factors. The intrinsic factors relate to the aspects of the concrete mix design, especially the materials used and their proportions. The extrinsic factors correspond to the construction practice with regards to how the concrete is mixed, placed, compacted, and cured.

2.2.2.2 Intrinsic Factors Affecting Concrete Durability

Intrinsic factors are predetermined by the mix design of the concrete. Binder content, binder type, water-binder ratio, and aggregate type are the most influential intrinsic factors in a concrete mix.

a) Binder Content

The influence of binder content on the microstructure of concrete is understood in terms of paste volume. Durability performance is affected by the paste volume, which has the most significant effect on the porosity of the concrete, with a higher paste content resulting in a more porous concrete (Kolias & Georgiu, 2005).

At a constant w:b ratio, an increase in binder content results in an increase in water content. The increased water results in an increase in capillary absorption as well as an increase in chloride penetration (Wasserman, et al., 2009; Kolias & Georgiu, 2005), Figure 2-2.

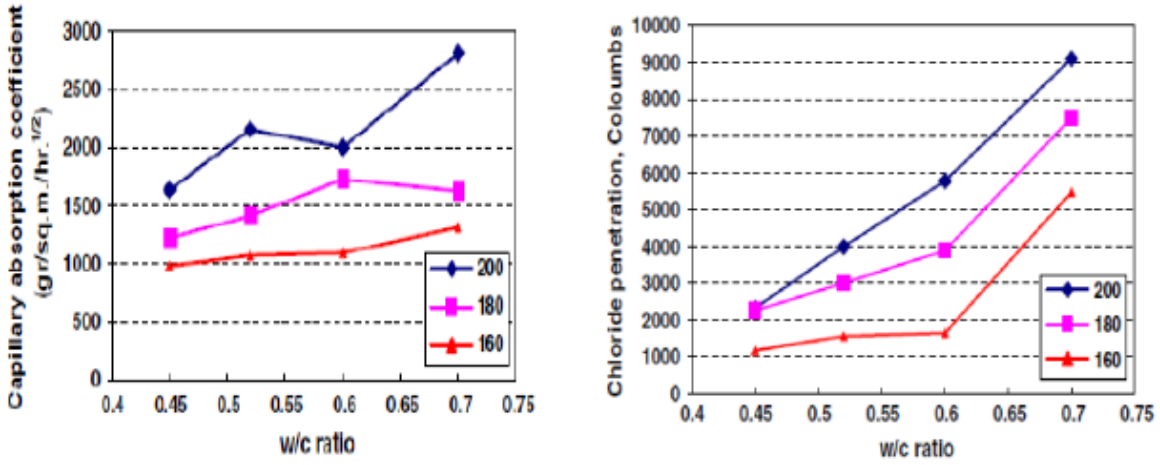


Figure 2-2 - Effect of cement content on absorption and chloride penetration (Wasserman, et al., 2009)

It can be reasonably concluded that the use of a minimum binder content specification may not ensure durability. It has been shown that improved durability can be achieved with different binder types rather than with different binder contents (Dhir, et al., 1996).

b) Binder Type

All concrete mixes will have a certain percentage of Ordinary Portland Cement (OPC), but most modern concrete mixes contain an added percentage of cement extenders. These extenders generally have positive effects on the fresh and hardened properties of concrete as well as generally beneficial effects on concrete durability.

Table 2-1 summarises the advantages and disadvantages of the most used cement extenders in South Africa (Grieve, 2009).

Table 2-1 - Effects of typical cement extenders on the properties of concrete (Grieve, 2009)

Extender	Advantages	Disadvantages
Fly Ash (FA)	Improved workability Increased strength Reduction in water demand Reduction in permeability Reduced water penetration Refined pore structure Improved resistance to chemical attack Reduction in shrinkage Reduced heat of hydration Reduction in cracks Increase in chloride binding capacity Improved sulphate resistance	Longer curing requirements Slower strength development
Ground granulated blast furnace slag (GGBS)	Increased strength Reduced permeability Refined pore structure Improved sulphate resistance Reduced heat of hydration	Retards setting Slower strength development Increased rate of carbonation Longer curing requirements
Condensed Silica Fume (CSF)	Increased cohesiveness Reduction on bleeding Increased strength Reduced permeability	Reduced workability

There has been considerable research into the effect of cement extenders on transport mechanisms of concrete. Ground granulated blast-furnace slag (GGBS), fly ash (FA) and condensed silica fume (CSF) have been shown to delay the onset of corrosion in a chloride environment and improve the durability of the concrete. The improved durability is through a combination of mechanisms such as improved microstructure, lower degree of interconnected voids, improvements to the interfacial transition zone (ITZ) between the matrix and the aggregate particles and chloride binding (Scott & Alexander, 2007). A summary of the effects of cement extenders on the transport mechanisms of concrete is shown in Figure 2-3.

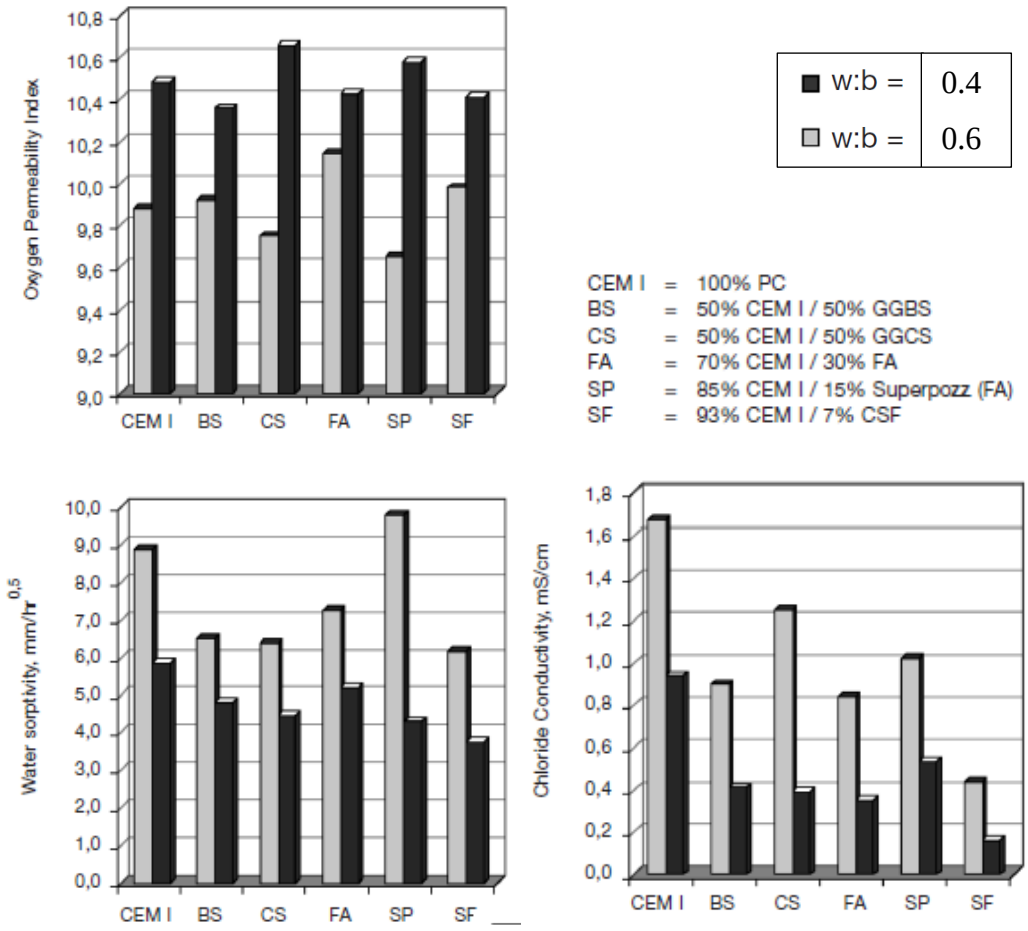


Figure 2-3 - Typical tests results for Durability Index values, measured on water-cured samples at 28 days of age, in relation to w:b ratio and binder type (Ballim, et al., 2009)

c) Water Binder Ratio

Traditionally, improved durability and increased compressive strength have been associated with decreased w:b ratios. Permeability increases considerably with an increase in w:b ratio due to the increased porosity and interconnectivity of the pores (Ballim, 1994). Figure 2-4 indicates how the chloride conductivity is increased with an increase in w:b ratio for all binder types.

d) Aggregate Type and Size

Aggregates have different levels of porosity and absorption. In previous versions of SANS1083, aggregate water absorption of less than 1% was considered safe. Naturally, a high level of absorption in an aggregate can reduce the durability of the concrete. Porosity of aggregates within concrete is particularly important for concrete exposed to chemical attack, or in freeze-thaw conditions (Grieve, 2009).

Mehta and Monteiro (2006) note that larger aggregate particles attract more bleed water around concrete aggregates. This increases the water/ binder ratio at the aggregate interface resulting in a higher porosity of the ITZ. Loseby (2014), tested the effects of aggregate properties on concrete potential durability and found that the influence of particle size and grading were not statistically significant and less than effects of variations in water/binder ratio and binder type and content (Loseby, 2014)

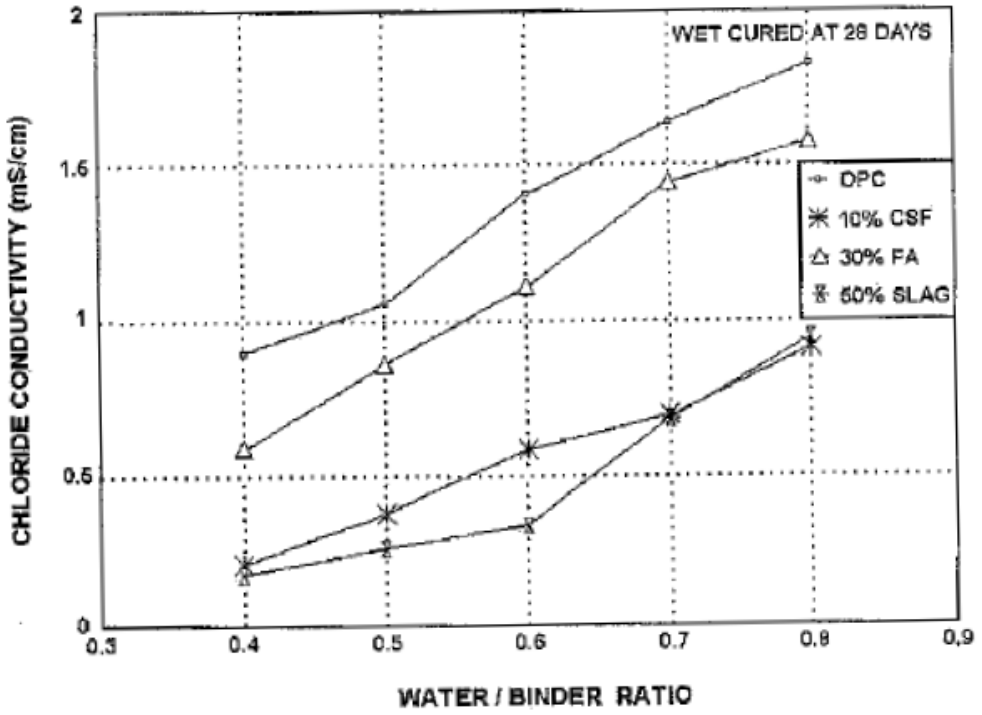


Figure 2-4 - Chloride conductivity versus water-binder ratio (Alexander, et al., 1999)

2.2.2.3 Extrinsic Factors Affecting Concrete Durability

Extrinsic factors relate to the production, placing, compaction and curing of the concrete. The desired outcome from placing and compacting of concrete is to obtain a uniform mix, with limited voids and no honeycombing, segregation or excessive bleeding.

a) Concrete Mixing

Thorough mixing is essential to produce concrete of uniform quality and, as such, affects the ultimate penetrability of the concrete. Mixing must be such that the various sizes of aggregates are uniformly distributed throughout the mix and each aggregate particle is coated with a cement paste of uniform consistency (Kellerman, 2009). The slump tests, as described in SANS5862-1, is commonly used to measure the concrete's consistency. A batch of concrete with variable consistency may result in inconsistent consolidation of the concrete

resulting in areas of poorly compacted concrete having excessive voids and reduced penetrability.

b) Concrete Placing and Compacting

Improper placing of concrete may result in segregation and honeycombing of the placed concrete resulting in a significant reduction in concrete durability. To produce on site concrete with minimal voids (Choo & Newman, 2003):

- The concrete should be discharged as close as possible to its final position, preferably straight into the formwork;
- The free-fall distance should be kept to a minimum to avoid segregation.
- Cold joints need to be avoided by ensuring that placed concrete does not set before fresh concrete is placed over it. This is achieved by good planning of the concrete casting sequence and monitoring of concrete properties during casting.
- Adequate vibration of concrete is critical for the removal of air bubbles within the concrete. Vibration is achieved by internal or external vibrators.
- Vibration should not be used to move the concrete into place as this may cause segregation of the concrete.
- High-workability mixes should not be over-vibrated – this may cause segregation.

Most concrete mixes are not self-compacting and will require a certain amount of assistance in compacting the in-situ concrete. Mechanical vibration is the most common and effective way of compacting concrete. Insufficient vibration will result in excessive voids in concrete, increasing the penetrability (Kellerman, 2009).

c) Curing

Curing is the process of limiting the early loss of water from concrete to facilitate the hydration of cementing reactions of the paste in the near-surface zone of the concrete member. Adequate curing will limit the permeability or accessibility to aggressive external agents into the surface zone of the concrete (Ballim, et al., 2009).

Poor curing is detrimental to the durability of concrete, particularly the outer concrete which is required to protect the embedded reinforcement (Pillai, et al., 2017). In general, the longer the period of curing, the better will be the quality of the concrete. Concrete, therefore, needs to be kept moist, both by the prevention of loss of moisture due to evaporation and by the provision of extra water from outside the concrete (Hoppe, 1995).

The most common and effective curing methods on site are listed below (Marsh, 2015):

Formwork Retention:	Moisture loss is prevented by keeping the formwork in place
Impermeable Coverings:	Sheets of polythene or plastics used to cover horizontal surfaces or wrap around vertical elements for the prevention of moisture loss
Absorptive Coatings:	Hessian or sand covering horizontal elements and kept moist
Addition of Water:	Ponding on horizontal surfaces
Curing Membranes:	Curing compounds hand or spray applied

2.2.3 Transport Properties of Concrete

For a concrete structure to be durable, it needs to limit the movement of liquids, ions, or gases through the pore structure. Since the most common durability problems associated with concrete is the corrosion of steel reinforcing within the concrete, it is imperative that this steel is protected against harmful substances. This protection is provided by the cover concrete between the concrete surface and the location of the reinforcing bars (Figure 2-5). The movement of aggressive agents through the cover concrete is synonymous with the transport properties of the concrete.

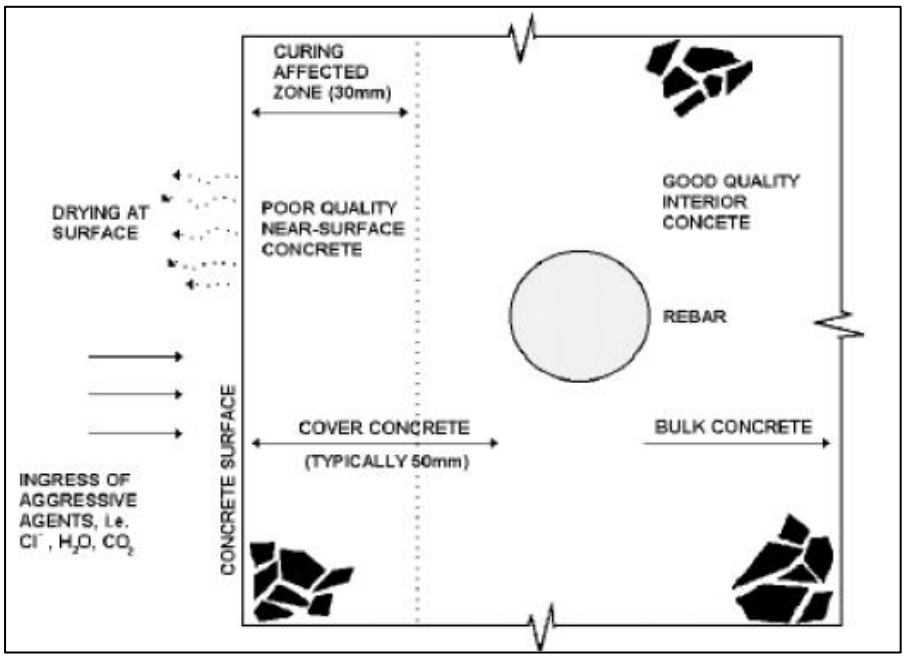


Figure 2-5 - Schematic of concrete cover layer (Alexander, 2004)

Transport properties of cementitious materials are key to predicting their durability since deterioration mechanisms such as chemical attack, leaching, chloride ingress or carbonation all relate to the ease with which a fluid or ion can move through the concrete microstructure (Ballim, et al., 2009). Concrete transport properties refer to the mechanisms of permeation, absorption, diffusion, migration, convection, and wick action. Transport mechanisms concerning cracked concrete are beyond the scope of this study.

2.2.3.1 Permeation

Permeation is the movement of fluids through the saturated pore structure under an externally applied pressure. The permeability of concrete is dependent on the degree of interconnection of the concrete's pore structure and moisture content of the material (Basheer, et al., 2001). For permeation, the average velocity of flow ($\bar{v}\ -\ measured\ in \frac{m}{s}$) for the fluid through the pore structure can be calculated using D'Arcy's law (Nganga, 2011):

$$\bar{v} = K\left(-\frac{dh}{dx}\right) \tag{2.1}$$

Where K = permeability coefficient (m/s)

$\frac{dh}{dx}$ = rate of change in pressure head in x-direction

x = distance (m)

2.2.3.2 Absorption

Absorption refers to the uptake of fluids into the concrete through capillary action. The rate of movement of a wetting front through a porous material under the action of capillary forces is defined as sorptivity (S) (Kropp & Alexander, 2007).

$$S = \frac{\Delta M_t}{t^{0.5}}\left(\frac{d}{M_{sat} - M_0}\right) \tag{2.2}$$

$\frac{\Delta M_t}{t^{0.5}}$ = slope of the straight line produced when the mass (g) of water absorbed is plotted against the square root of time (s)

d = Sample thickness (mm)

M_{sat} & M_0 = Saturated mass and dry mass of concrete specimen respectively

2.2.3.3 Diffusion

Diffusion refers to the movement of liquid, gas, or ions through the concrete due to a concentration gradient (Jackson & Dhir, 1996). The diffusion of chloride ions in hardened concrete is widely recognised as an important factor in the corrosion of reinforcing steel, particularly in the marine environment where concretes are exposed to chlorides from seawater (Page, et al., 1981).

The chloride diffusivity of concrete, characterised by the diffusion coefficient (D), is often used as a material parameter to quantify the concrete's resistance to chloride ingress (Streicher, 1997). The modelling of diffusion through concrete is done using Fick's second law of diffusion, which concerns the rate of change of concentration with respect to time (t) and position (x). This law is stated in Equation 2.3:

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} \tag{2.3}$$

Where D = Diffusion Coefficient (m^2/s)

C = Chloride Concentration (g/m^3)

With boundary conditions:

C_x = 0 at t = 0 and $0 < x < \infty$

C_x = C_s at x = 0 and $0 < t < \infty$

Equation 2.3 can be solved using Crank's error function (Alexander, et al., 2012):

$$C_{x,t} = C_s\left[1 - \text{erf}\left(\frac{x}{2\sqrt{D_a t}}\right)\right] \tag{2.4}$$

Where $C_{x,t}$ = Chloride concentration at depth x (m) at a given time t (s)

C_s = surface concentration (g / m3)

D_a = Apparent chloride diffusion coefficient (m^2/s)

t = time of exposure (s)

erf = mathematical error function

The equation assumes that the chloride surface concentration and the diffusion coefficient are constant with time, although this is not entirely true. Firstly, the surface concentration of

chlorides will vary due to wetting and drying. Secondly, the diffusion coefficient varies with time due to chloride binding and continuous hydration of the cementitious products, and for this reason, an apparent diffusion coefficient (D_a) is often used which is time-dependant (Alexander, et al., 2012).

2.2.3.4 Migration

Migration is the movement of ions in a solution under an electrical potential. Migration, also referred to as accelerated diffusion, is directly related to diffusion (Bockris, et al., 1981). Migration is the key transport mechanism in laboratory-accelerated chloride tests, and is governed by the Nernst-Plank equation (Otieno, et al., 2010):

$$v = \left(D\frac{zF}{RT}\right)\left(\frac{dU}{dx}\right) \tag{2.5}$$

Where v = velocity of ionic species (m/s)
D = Diffusion coefficient of ionic species (m^2/s)
z = electrical charge of diffusing ions
F = Faraday's constant (9.6548 x 10^4 J/Vmol)
T = Absolute Temperature (K)
U = Potential Difference across the sample (V)
x = distance variable (m)
R = Universal gas constant (8.314 J/molK)

As seen in Equation 2.4 and Equation 2.5, both migration and diffusion are related to the Diffusion Coefficient (D) of the concrete. For this reason, tests for concrete migration are often used to predict a concrete's resistance to diffusion.

2.2.3.5 Convection and Wick Action

Convection describes the transport of a solute resulting from the bulk movement of water. This process is described by the following equation (Otieno, et al., 2010):

$$\frac{\partial C}{\partial t} = -\bar{v}\frac{\partial^2 C}{\partial x^2} \tag{2.6}$$

Where C = Concentration of solute at depth x (m) after time t (s)
$\bar{v}$ = average velocity vector of fluid flow (m/s)

Wick action is the transport of water and ions from a face in contact with water/salt solution to a drying face which is exposed to air with less than 100% relative humidity. Wick action is often dominated by convection (Puyate & Lawrence, 1999). Chlorides can pass through concrete through wick action.

2.3 CORROSION OF STEEL IN CONCRETE

Concrete is a durable material which offers protection to steel in reinforced concrete. Theoretically, a reinforced concrete structure should be highly durable since the concrete offers physical and chemical protection to the embedded steel. However, corrosion of steel in concrete has become a significant durability problem in mild as well as in severe climatic conditions (Bentur, et al., 2005). Corrosion damage is usually observed as rust stains on the surface of the concrete, followed by expansive cracking running parallel to the reinforcing steel (Yeomans, 2017).

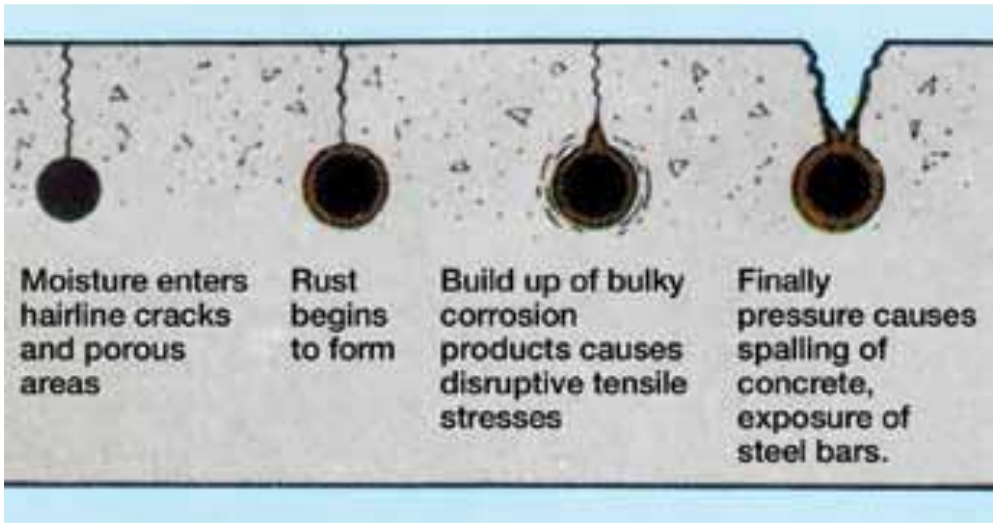


Figure 2-6 - The progress of corrosion-induced damage in concrete (Yeomans, 2017)

The long-term durability of structures in the marine environment is of serious concern due to the structures' exposure to chloride ions and the effect of these chloride ions on the rate of corrosion of reinforcement. Although there has been much research on the transport mechanisms of aggressive ions through concrete as well as the corrosion of reinforcing steel (Yu, et al., 1993; Streicher, 1997), corrosion of reinforcement is still the greatest cause of concrete durability failures (Cement and Concrete Institute, 2012). Investigations have indicated the durability performance of reinforced concrete structures in the marine environment to be generally unsatisfactory. (Mackechnie, 2001)

2.3.1 The Corrosion Process

Corrosion may be defined as "the deterioration of a metal by reaction with species in the environment to form chemical compounds" (Choo & Newman, 2003). Corrosion is an electrochemical process involving the flow of ions and electrons. The fundamental cause of corrosion of steel is the inherent instability of steel in its metallic form. The tendency is for metals is to revert to more stable forms such as their oxides, hydroxides or sulphides (Heckroodt, 2002).

For corrosion to occur, four essential elements are required (Otieno, et al., 2010):

a) Anode: Site where corrosion occurs and from which current flows
b) Cathode: Site where corrosion does not occur but to which the electric current flows
c) Metallic Path: Connection between anode and cathode through which the electrons flow
d) Electrolyte: A medium capable of conducting current by ionic current flow

The corrosion of steel embedded in concrete is illustrated in Figure 2-7. Electrons flow along the steel from the anode to the cathode (the steel being the metallic path), while hydroxyl ions (OH^-) flow through the concrete from the cathode to the anode (the concrete pore fluid being the electrolyte) (Alexander, et al., 2012).

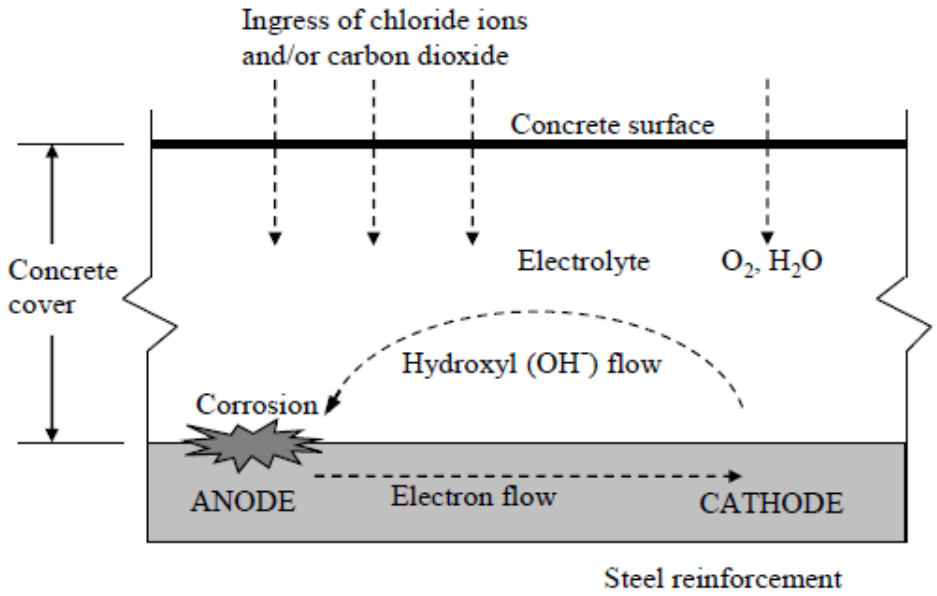


Figure 2-7 - A schematic illustration of the corrosion process in concrete (Alexander, et al., 2012)

For steel corrosion, the ionic flow is governed by the progressive removal of Iron (Fe^{2+}) from the steel. The corrosion reactions resulting in the flow of electrons and hydroxyl ions are given by Equation 2.7 and Equation 2.8 below (Bockris, et al., 1981):

$$2Fe \rightarrow 2Fe^{2+} + 4e^{-} \tag{2.7}$$

$$2H_2O + O_2 + 4e^{-} \rightarrow 4OH^{-} \tag{2.8}$$

Iron (Fe) that has been converted to Fe^{2+} can then form corrosion products of hydroxides, oxides, and oxide-hydroxides (Alexander, et al., 2012). These corrosion products occupy larger volumes than the original iron (Figure 2-8), and therefore result in expansive cracking and spalling of concrete.

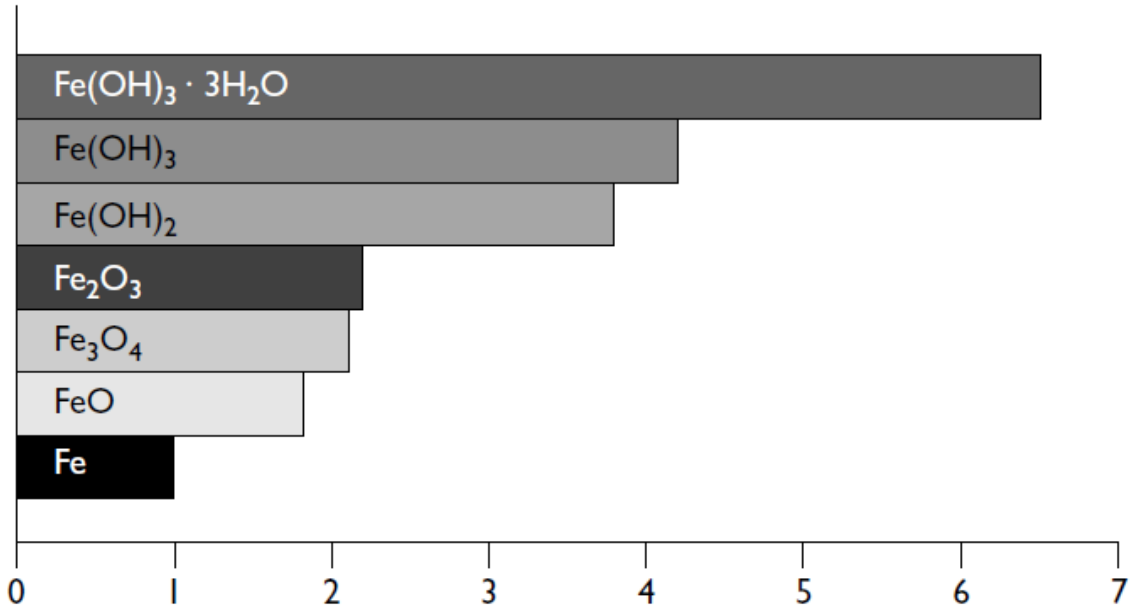


Figure 2-8- Relative volume of iron and its oxides (Corrosion1981mansfield) – (See Broomfield corrosion of steel in concrete page 9)

There are two primary protective measure of steel in concrete. Firstly, the cover concrete which is dealt with extensively in Section 2.2. Secondly, the passivation of the steel. Passivation is the forming of a natural a thin passive ferric-oxide (γ-Fe_2O_3) film on the surface of the steel surface resulting from the high alkalinity of the cement matrix. This high alkalinity is due to the presence of sodium hydroxide *(NaOH)*, Calcium hydroxide *($Ca(OH)_2$)* and potassium hydroxide (*KOH)*. The ferric-oxide film is dense and impermeable and therefore suppresses corrosion by limiting the movement of cations and anions (Otieno, et al., 2010).

Porous concrete will allow aggressive ions such as chlorides or carbon dioxide to permeate through to the steel bars and break down this protective film. Prevention of steel corrosion is therefore highly dependent on the quality and depth of concrete cover. The breakdown of the protective film is referred to as depassivation and is typically initiated by the presence of carbon-dioxide or chlorides (Heckroodt, 2002).

2.3.2 Three-stage Corrosion Model

The deterioration of steel in concrete involves three stages, as illustrated in Figure 2-9. (Heckroodt, 2002)

Stage1: The initiation period where there is no evidence of damage, but harmful substances such as chlorides and carbon dioxide penetrate the cover concrete until such time that they reach the reinforcing steel surface and cause de-passivation of the reinforcement, and subsequent corrosion of steel. The time to corrosion activation is controlled by the resistance of the cover concrete to the ingress of harmful substances.

Stage 2: The propagation phase occurs after the initiation of corrosion and results in the production of expansive products, as discussed in 2.3.1.

Stage 3: In the accelerated corrosion period, the expansive products cause increased stresses in the concrete resulting in widespread cracking and spalling of concrete. This in turn creates additional paths for water and oxygen to reach the reinforcing steel and significantly increase corrosion of the steel reinforcing.

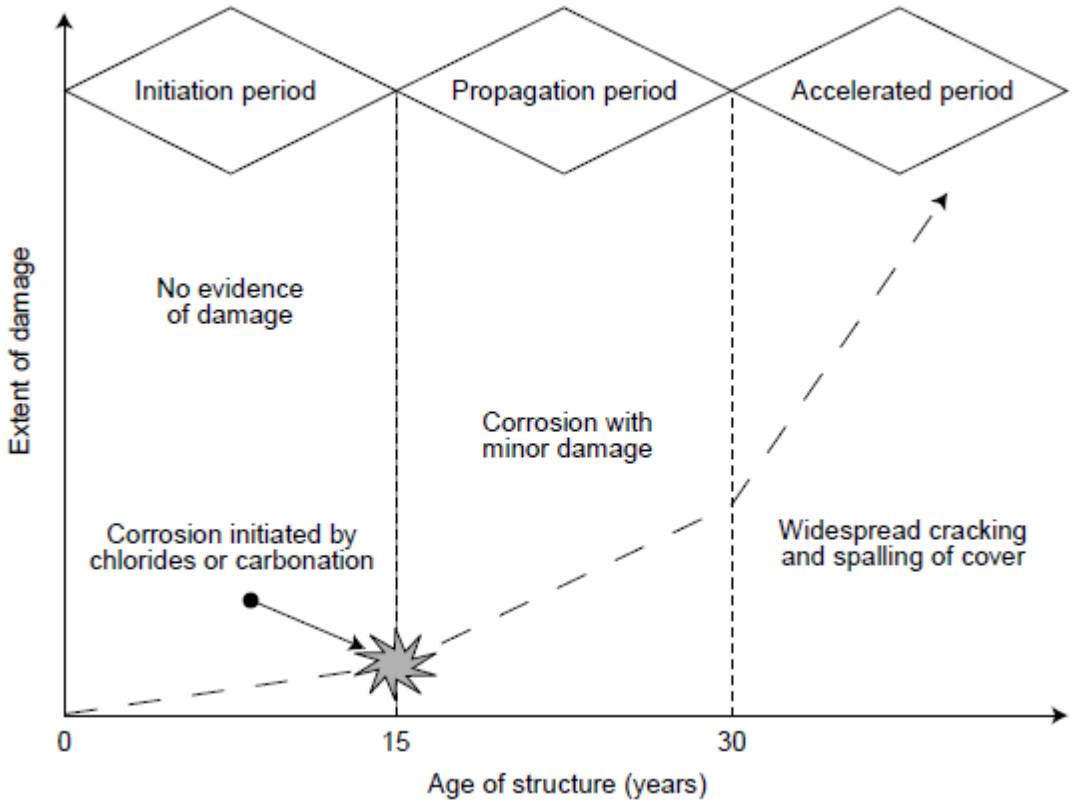


Figure 2-9 - Three stage corrosion damage model (Heckroodt, 2002)

The onset of corrosion at the end of the initiation stage, into the propagation stage, is induced primarily by carbonation or the presence of chloride ions.

2.3.3 Carbonation-Induced Corrosion

Carbonation occurs in concrete when atmospheric carbon dioxide, CO_2, penetrates the concrete and reacts with the water and calcium hydroxide in the concrete to form carbonic acid, H_2CO_3. The carbonic acid neutralises the alkalis in the pore water to form calcium carbonate. This results in the lowering of the pore solution pH from above 12 to between 8.5 and 9.0. The carbonation reactions are detailed in Equation 2.9 and Equation 2.10 (Broomfield, 2007):

$$\underset{carbon\ dioxide}{CO_2} \quad \underset{water}{+H_2O} \quad \underset{carbonic\ acid}{\rightarrow H_2CO_3} \tag{2.9}$$

$$\underset{carbonic\ acid}{H_2CO_3} \quad + \underset{\substack{Calcium\ Hydroxide \\ in\ concrete\ pore\ solution}}{Ca(OH)_2} \rightarrow \quad \underset{Calcium\ Carbonate}{CaCO_3} \quad + \underset{Water}{2H_2O} \tag{2.10}$$

This process leads to depassivation of the steel, which is in contact with the carbonated concrete. The carbonation lowers the pH of the concrete's pore solution. Carbonation moves as a "front" into the concrete as shown in Figure 2-10. This front does not advance beyond a point until all the calcium hydroxide at that point has been converted to calcium carbonate. Hence, the amount of calcium hydroxide in the pore structure of the concrete also has an influence on the rate of carbonation (Ballim, et al., 2009). Once the carbonation front reaches the steel, the corrosion process will begin in the presence of water and oxygen.

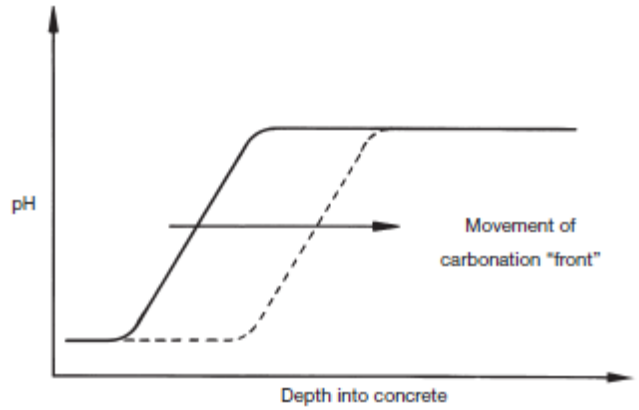


Figure 2-10 - pH reduction due to carbonation movement as a front from the concrete surface (Ballim, et al., 2009)

The carbonation process can be modelled using Equation 2.11, derived from Fick's first law of diffusion, which considers the relationship between carbonation depth (x) and time (t).

$$x = kt^n \tag{2.11}$$

Where k = carbonation coefficient – dependant on the diffusivity of the concrete, carbon dioxide concentration in the atmosphere and environmental conditions

n = Time exponent value which varies with concrete properties and exposure environment

The value of n could be less than 0.5 if the concrete is subject to wetting and drying cycles since the diffusion of carbon dioxide is slower through moisture-filled pores. Alternatively, the value of n may be greater than 0.5 in the case of cracked concrete (Ikotun, 2017). The depth of carbonation of concrete is often determined using a phenolphthalein pH indicator, which changes colour at a pH of approximately 9. Below 9 the indicator remains colourless while above this, it turns purple (Richardson, 2005).

2.3.4 Chloride-Induced Corrosion

Chloride ions may be present within the concrete pore structure through the mix constituents, the absorption of seawater, or de-icing salts in environments that are subject to freezing and thawing. The transfer of chloride ions through the concrete takes place generally through diffusion and absorption (Neville, 2011).

Chloride ions at the reinforcement level break down the protective film, γ-Fe_2O_3, surrounding the reinforcement and act as a catalyst to corrosion. The chloride ions are not consumed in the reaction but assist in breaking down the passive layer and allowing the steel to corrode faster. The typical reactions resulting in the corrosion of steel exposed to chlorides is shown in Figure 2-11 (Broomfield, 2007).

Chloride ions react with the steel to form hydrochloric acid (HCl), which break down the passive protective film, resulting in the formation of an anode, with adjacent steel forming the cathode. In Figure 2-11, it can be seen that the chloride ions react further with the Fe^{2+} to form chloride and oxychloride compounds ($FeCl_2$ and FeOCl) (Melchers & Li, 2006).

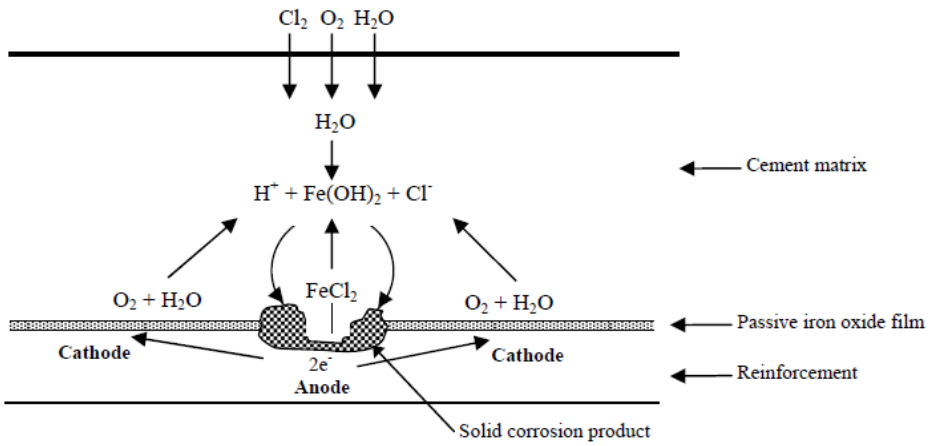


Figure 2-11 - Corrosion of reinforcement in concrete exposed to chloride ions (Broomfield, 2007)

2.3.4.1 Effect of Chloride Binding on Chloride-Induced Corrosion

Chlorides within the concrete matrix can be either free or bound. Free chlorides are those chloride ions dissolved in the concrete pore solution which are mobile and free to react with the steel and initiate and promote corrosion. Bound chlorides are those that are physically and chemically bound to the cement hydrates within the concrete. Chloride binding removes chlorides from the diffusion process resulting in a refined pore structure, and a change in the concentration gradient which drives ionic diffusion (Glass & Buenfield, 1997). For this

reason, the diffusion coefficient (D) referred to in Equation 2.2 is often modelled mathematically as varying with time (Alexander, et al., 2012).

It is significant to note that the binder types have different chloride binding capacities. It is for this reason that certain binders significantly increase the potential durability of concrete structures due to the binding with free chlorides. The effect of cement extenders on chloride binding are shown graphically in Figure 2-12 (Glass & Buenfield, 1997).

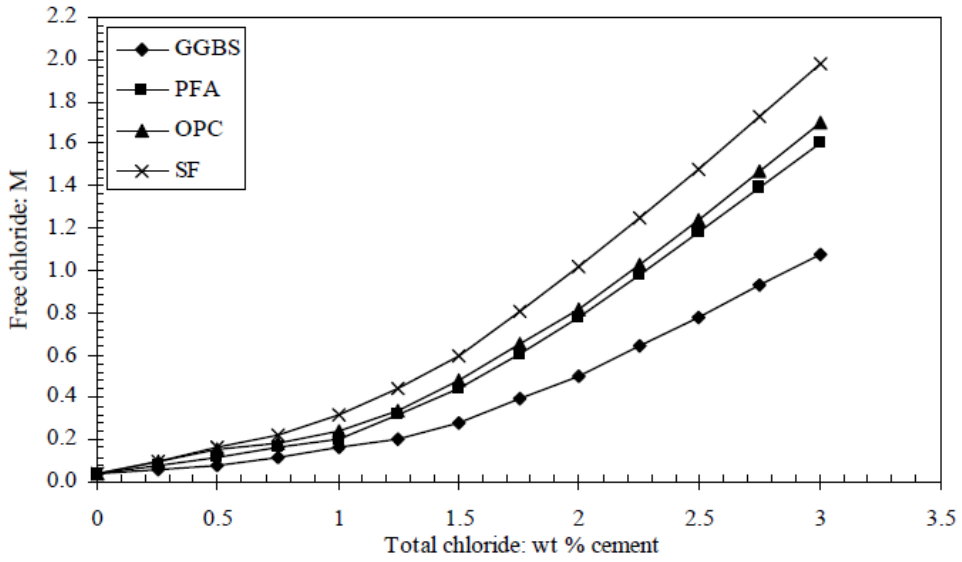


Figure 2-12 - Chloride binding relationships dependant on binder type and total concentration (Glass & Buenfield, 1997)

2.3.4.2 Chloride Threshold

Reinforcement corrosion in non-carbonated, alkaline concrete can only start once the chloride content at the steel surface has reached a certain threshold value, referred to as the *critical chloride content* or the *chloride threshold value* (Angst, et al., 2009). There are two common definitions of chloride threshold levels (Tuutti, 1982):

- Scientifically, the chloride threshold level is the chloride content required for depassivation of steel, which will initiate corrosion.
- In practical engineering terms, the chloride threshold level is the chloride content associated with visible deterioration of the reinforced concrete structure.

These two definitions are indicated graphically in Figure 2-13. In this report, the critical chloride threshold level will be the more precise scientific definition.

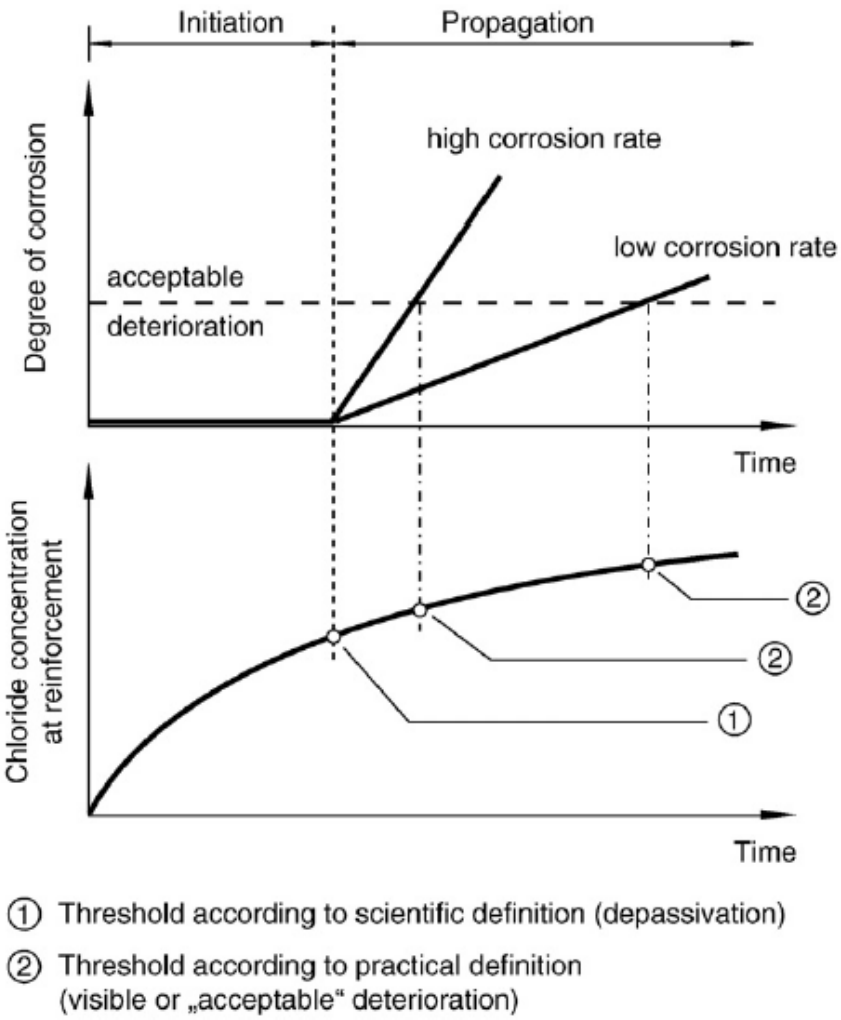


Figure 2-13 - Definitions of chloride threshold levels (Angst, et al., 2009)

Knowledge of chloride threshold levels is essential since this is a critical input parameter in modelling service life of concrete structures. Chloride threshold values are generally very uncertain since there are many contributing factors (Nilsson, et al., 1996).:

- cover thickness
- temperature
- relative humidity
- the electrical potential of the reinforcement
- the chemistry of the binder
- the proportion of total chlorides to that of free chlorides
- chloride to hydroxyl ion ratio

Chloride threshold levels are presented in several ways (Nilsson, et al., 1996):

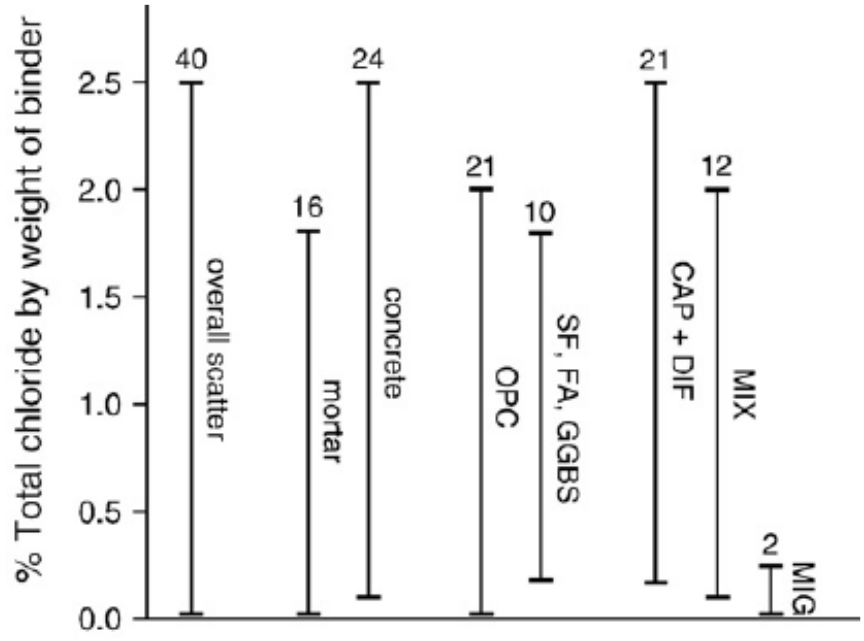
- Total chloride content by weight of concrete

- Total chloride content by weight of binder
- Free chloride concentration in the pore solution of the concrete
- Ratio between free chloride content and free hydroxide concentration

It is practically difficult to determine the free chlorides in a pore solution. Therefore, chloride threshold values are generally quoted as either chloride content by weight of concrete or weight of binder. Based on the service life models used in this report, chloride threshold values will be quoted as a total chlorides as a percentage of the weight of binder. This approach has also been shown to give the least spread of results (Glass & Buenfield, 1997).

There are conflicting values on critical chloride levels for corrosion activation in literature (Glass & Buenfield, 1997; Oh, et al., 2003; Castellote, et al., 2002; Breit, 1998; Mackechnie, 2001). These values vary quite drastically from one study to the next, ranging from 0.01% to 3%. Angst, et al (2009) reviewed several published values of critical chloride contents with the scatter of these values indicated in Figure 2-14. The numbers above the bars in the graph indicate the frequency of occurrence in the reviewed literature.

Based on the published values, the critical chloride content of 0.4-0.5%, used as suggested by Machecknie (2001) can be considered as reasonable to conservative.



Note:
The numbers above the bars in the graph indicate the frequency of occurrence in the reviewed literature.

Figure 2-14 - Scatter of chloride threshold values sorted by selected parameters (Angst, et al., 2009)

Abbreviation	Meaning
OPC	Concrete made with Ordinary Portland Cement
SF	Concrete made with Silica Fume

FA	Concrete made with Fly Ash
GGBS	Concrete made with Ground Granulated Blast Furnace Slag
CAP	Chloride Introduced by Capillary Suction
DIF	Chloride Introduced by Diffusion
MIX	Chlorides added to the Mix
MIG	Chloride Introduced by Migration

In addition to limiting chlorides, it is also imperative that cover concrete reduces the diffusion and permeation of carbon dioxide because carbonation at the steel reinforcing level will reduce the chloride threshold level to zero. The reason for this reduction in chloride threshold is because of the local lowering in pH as well as the loss in chloride binding capacity of the binders, resulting in more free chlorides in the pore solution (Nilsson, et al., 1996).

-—The acid-soluble chloride test is often used to determine total chloride ion content in a concrete (ASTM C1152, 2012). The method involves chloride profiling in which dust samples are obtained at different depths of a core and tested to determine total chloride ion content. The proportion of chloride content is expressed typically as the weight of binder (where this is known) or otherwise as a percentage of the weight of concrete (Richardson, 2005).

2.4 TEST METHODS FOR CONCRETE COVER PROPERTIES

Several tests have been developed for the quantification of potential durability of concrete. These tests generally measure the concrete's resistance to transport mechanisms through the pore structure of the concrete. Below is a summary of the more widely used test methods for concrete durability, concluding with the test methods used in South Africa for Durability Indexes.

2.4.1 International Tests on Cover Concrete

2.4.1.1 Gas Permeability Tests

Gas permeability tests measure the concrete's resistance to permeation and, consequently, its potential resistance to carbonation. A short summary is given of the more prevalent international permeability tests (Cembureau Test and Torrent Permeability Test) below, with details on the South African Oxygen Permeability Test given in 2.4.2.1.

a) Cembureau Permeability Test (Kollek, 1989)

In the Cembureau method, unidirectional gas flow between two parallel surfaces of the test specimen (typically 150 mm diameter by 50 mm thick) is caused by a constant absolute pressure difference of the test gas between the two surfaces. The schematic of the test method is presented in Figure 2-15. The gas flow depends on the pressure difference, testing area, specimen thickness and open porosity, and the viscosity of the test gas. Pressure is applied to the specimens in the permeability cells, and the gas flow through the specimen is measured at the flow bubble meters at the gas outlet ends. Flow rate results are used to calculate a gas permeability coefficient through the concrete specimen.

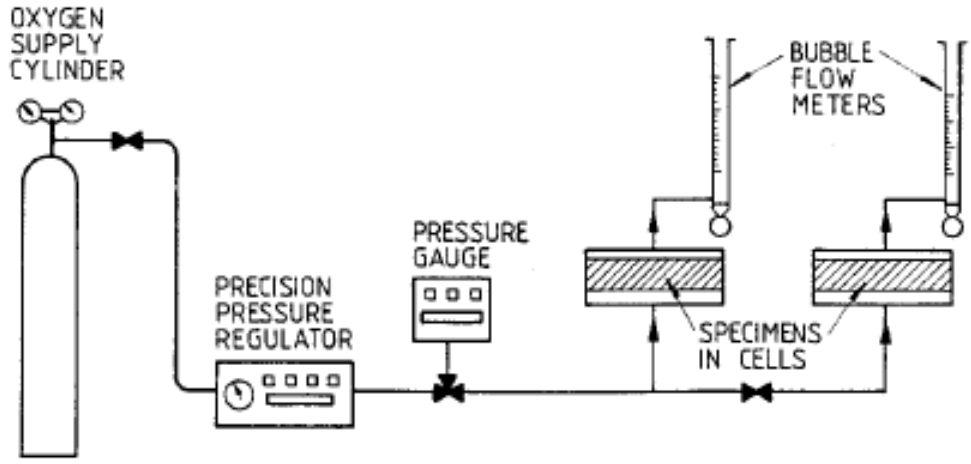


Figure 2-15 - Line diagram of Cembureau oxygen permeability apparatus (Kollek, 1989)

b) Torrent Permeability Test (Torrent, 1992)

In some European countries, oxygen permeability characteristics are determined in-situ using the Torrent permeability tester (TPT). Figure 2-16 shows a line diagram of the TPT test set up. The test equipment consists of a two-chamber vacuum cell and a regulator that balances the pressure in the inner (measuring) chamber and the outer (guard-ring) chamber. The outer guard ring prevents air from the surrounding areas from flowing into the pressure measurement chamber. The test is carried out by placing the cell on the concrete test surface and producing a vacuum. The rate at which the pressure rises in the inner chamber is recorded, and this rate is related to the permeability of the underlying concrete.

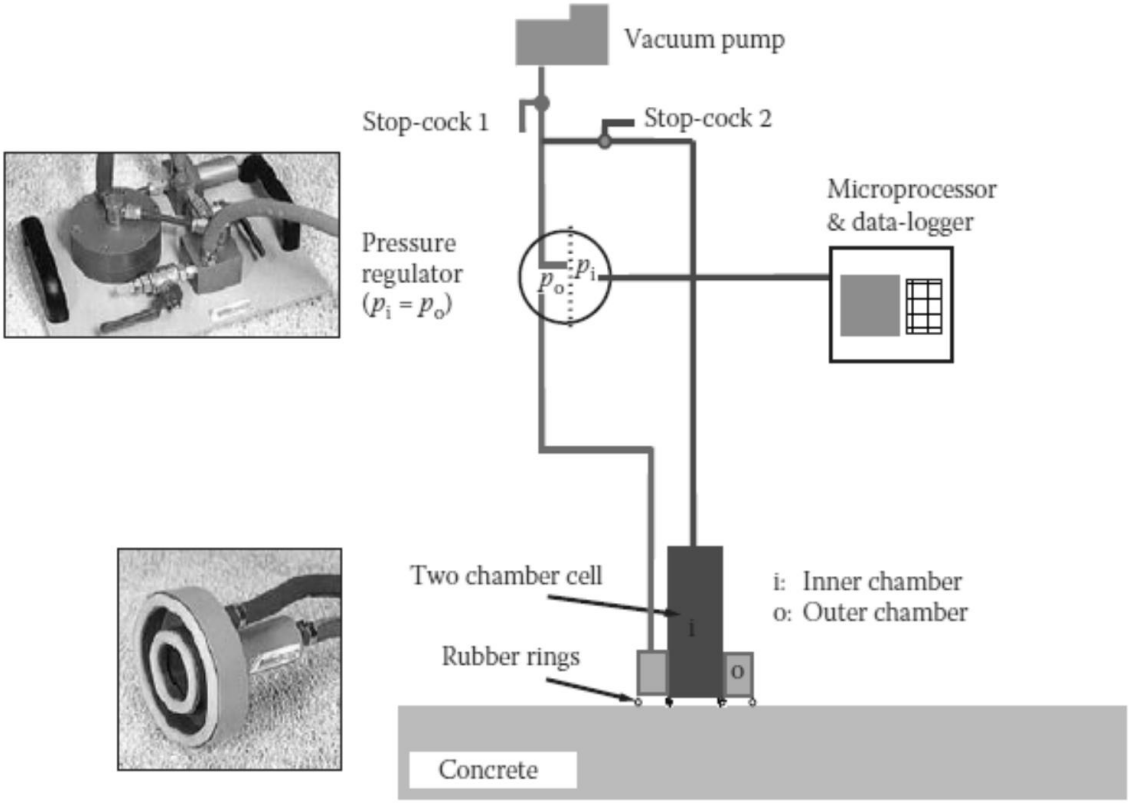


Figure 2-16 – Line Diagram of Torrent Permeability Test (Torrent, 1992)

2.4.1.2 Diffusion Tests

Diffusion is the movement of ions from a region of high to low concentration and is the primary mode of ingress of absorbed chloride ions in concrete. Classical diffusion tests take place over a longer period (e.g. bulk diffusion test), while accelerated diffusion tests will typically measure migration (which is the movement of ions in a solution under an electrical potential) and equate this to a diffusion coefficient. Commonly used diffusion (and migration) tests are detailed below. The bulk diffusion test is a longer duration test but measures actual diffusion, while the rapid chloride permeability test is an example of an accelerated test that measures migration.

a) Bulk Diffusion Test (NT Build 492, 1999; ASTM C1556 - 04, 2004)

In this test, samples are saturated with calcium hydroxide saturated water and then sealed on all sides except the top face and submerged in a 2,8 M NaCl solution (Figure 2-17), for a minimum of 35 days. After this time chloride profiling is done, by grinding off 0.5 mm segments of the specimens and measuring the chloride content at different depths. The chloride concentration profile is then used to determine the diffusion value and surface concentration (ASTM C1556 - 04, 2004).

The bulk diffusion test is not affected by the unknown implications of using an electrical field to accelerate the diffusion process, as with other accelerated diffusion tests (e.g. chloride conductivity test in 2.4.2.3).

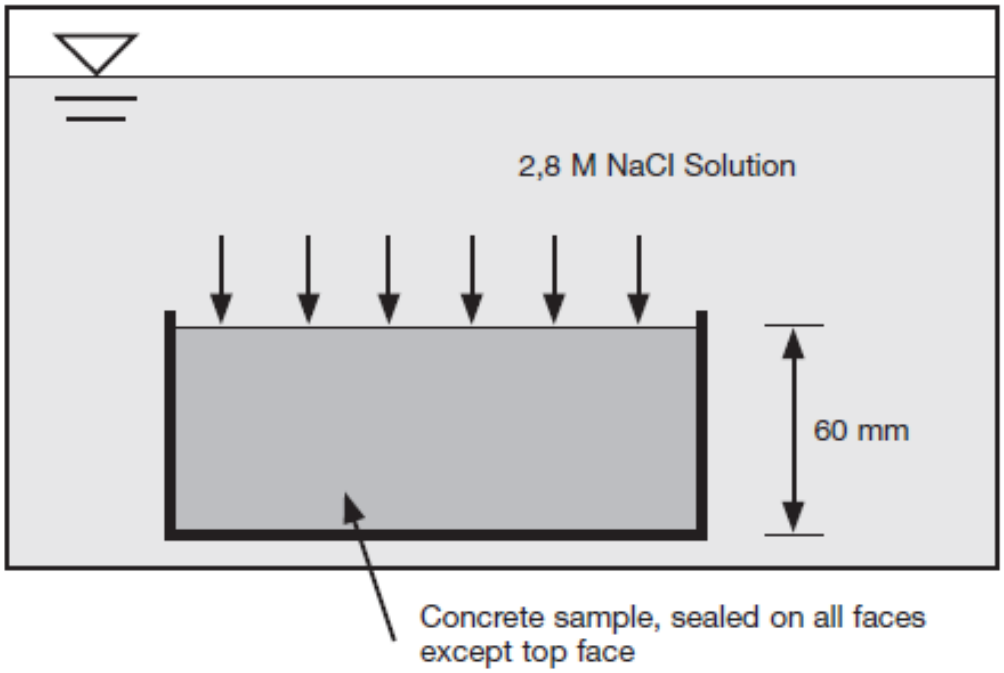


Figure 2-17 - Schematic of bulk diffusion test (ASTM C1556 - 04, 2004)

b) Rapid Chloride Permeability Test (ASTM C1202-19, 2019)

-The North American rapid chloride permeability test involves placing a water-saturated 50 mm thick 95 mm diameter specimen in a migration cell and applying a 60 V DC current for six hours Figure 2-18.

One cell of the migration device is filled with 3,0% NaCl solution and the other with 0,3 M NaOH solution, creating a chloride concentration difference between the two exposed faces of the specimen. The total charge passed is then determined and used to give a chloride migration rating. In this method, ionic flux is caused by both diffusion and conduction.

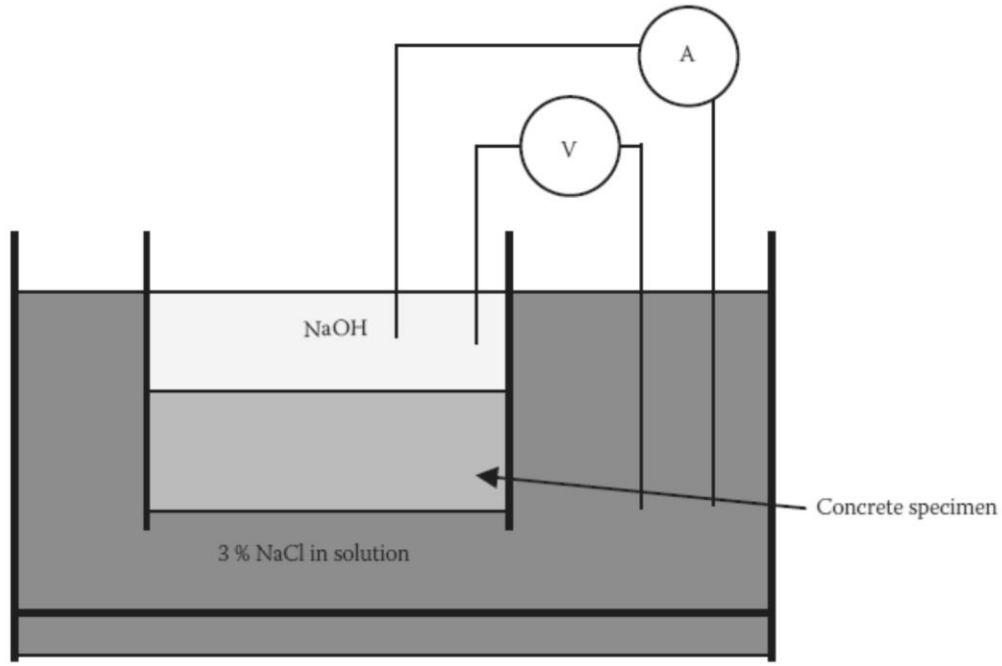


Figure 2-18 - Schematic of rapid chloride permeability test (ASTM C1202-19, 2019)

2.4.2 Durability Index Tests in South Africa

There has been an initiative in South Africa in recent years to incorporate Durability Index (DI) testing into the South African National Standards (Kessy, et al., 2015). Three concrete DI tests have been developed for use in South Africa for the quantification of concrete durability. The oxygen permeability (OPI) test, the water sorptivity test, and the chloride conductivity test are used to determine the concrete's permeability, absorption, and diffusion, respectively. These tests were developed under a collaborative research programme between the Universities of Cape Town and the Witwatersrand (Ballim, et al., 1999a). Test comparisons indicate the OPI and chloride conductivity tests correlate well with related test procedures in other countries (Beushausen & Alexander, 2008).

2.4.2.1 Oxygen Permeability Test (DI Testing Manual, 2018)

This test assesses the overall microstructure and macrostructure of the outer surface of cast concrete and is particularly sensitive to macro-voids and cracks, which act as "short-circuits" for the permeating gas. Thus, the test is beneficial in assessing the state of compaction, the presence of bleed voids and channels, and the degree of interconnectedness of the pore structure (Mackechnie, et al., 1999).

The Oxygen Permeability Index (OPI) test method consists of measuring the pressure decay of oxygen passed through a 30 mm thick slice of a (typically) 68 to 70 mm diameter core of concrete placed on a falling head parameter. The oxygen permeability index is defined as the

negative log of the coefficient of permeability. The test setup is shown in Figure 2-19. Three to four specimens are required per test. A pressure of 100-120 kPa is applied, and the pressure decay is measured until the pressure reached 50 kPa or 6 hours. The change in pressure with time is recorded (generally electronically with a data transducer) and the D'arcy coefficient of permeability is calculated according to Equation 2.12:

$$k = \frac{\omega V g d z}{RAT} \tag{2.12}$$

Where k = permeability coefficient (m/s)
ω = molecular mass of oxygen (i.e. 0.032 kg/mol)
V = volume of permeability cell (m^3)
g = Gravitational acceleration (i.e. 9.81 m/s^2)
d = Specimen thickness (m)
z = Slope of the linear regression line*
R = Universal gas constant (i.e. 8.313 Nm/Kmol)
A = Cross-sectional area of the specimen (m^2)
T = Temperature in Kelvin

*Note: z is the slope of the line of best fit of a plot of ln (P_0/P_t), against time forced through (0,0):

Where P_0 = initial pressure (kPa)
P_t = pressure (kPa) at a given time, t (s)

The D'arcy coefficient of permeability is used in the calculation of the Oxygen Permeability Index (OPI) according to Equation 2.13. The OPI value is given by the average of the individual OPI calculations for each of the four specimens.

$$OPI = -\log(k) \tag{2.13}$$

Common OPI values for South African concretes range from 8.5 to 10.5, a higher value indicating a lower permeability and thus a concrete of potentially higher quality (Ballim, et al., 2009).

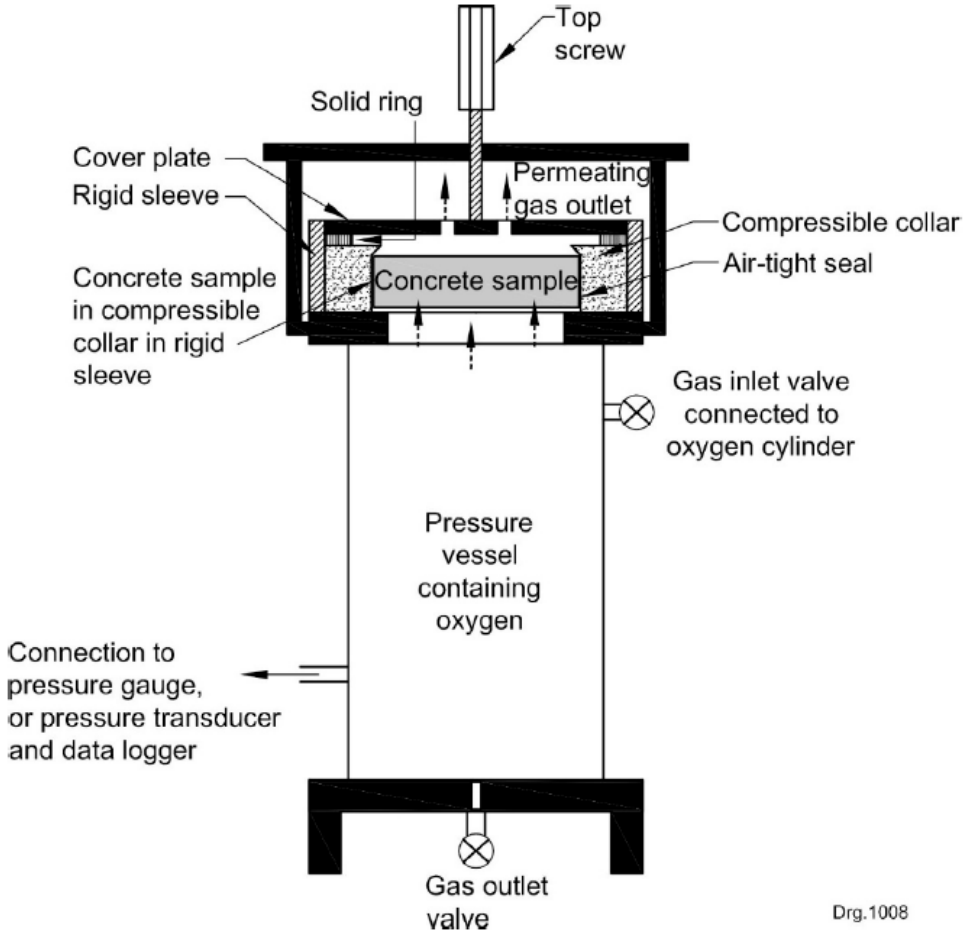


Figure 2-19 - Schematic of an oxygen permeameter (DI Testing Manual, 2018)

2.4.2.2 Water Sorptivity Test (DI Testing Manual, 2018)

This test measures the physical properties of a thin layer (5-15 mm) surface concrete, by means of capillary suction as indicated in Figure 2-20. The test is simple to carry out, but it is imperative to pre-condition the samples correctly to have a reproducible moisture condition at the start of a test. (Mackechnie, et al., 1999)

The sides of a 30 mm thick disc of a (typically) 70 mm +-2 diameter core are sealed to ensure unidirectional absorption from the bottom (usually exposed) face. The concrete samples are exposed to a few millimetres of water with the test surface facing downward. At regular time intervals, the specimens are removed from the water, and the mass of water absorbed is determined using an electronic balance. These measurements are taken at 3, 5, 7, 12, 16, 20, and 25 minutes. A linear relationship is observed when the mass of water is plotted against the square root of time, and the sorptivity of the concrete is determined from the slope of the straight line produced as per Equation 2.14:

$$M_{wt} = F\sqrt{t} \tag{2.14}$$

Where M_{wt} = Mass gained at each time interval (g)

t = Time from when specimen was first exposed to water (hr)

F = Slope of best line fit when plotting M_{wt} against time

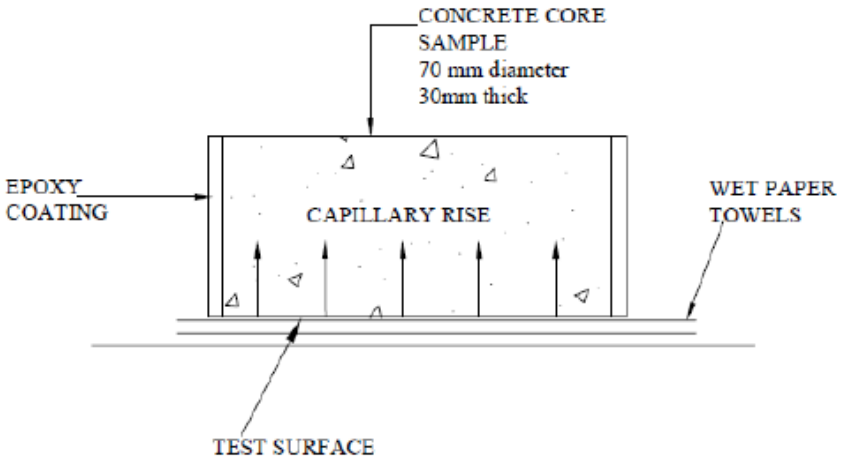


Figure 2-20 - Schematic of Water Sorptivity Test Arrangement (Alexander & Mackechnie, 2001)

The specimen is placed in a vacuum and saturated with calcium hydroxide for a period of 18 hours to determine the porosity and saturated mass. The sorptivity (S) of the specimen (in mm/$\sqrt{hr}$) is then calculated according to Equation 2.15. The total water sorptivity is calculated as the average of the Sorptivity values for the four specimens.

$$S = \frac{Fd}{M_{sv} - M_{s0}} \tag{2.15}$$

Where F = Slope of best fit line as per equation 2.14 above

M_{sv} = Vacuum saturated mass of specimen (g)

M_{s0} = Dry mass of specimen at time t=0

d = Average specimen thickness (mm)

The porosity (n) of the concrete specimen, calculated as a percentage, is determined according to Equation 2.16. A lower WSI indicates concrete with low sorptivity and consequently, higher durability.

$$n = \frac{M_{sv} - M_{s0}}{Adp_w} \times 100 \qquad (2.16)$$

Where A = Cross-sectional area of the specimen (mm^2)

p_w = Density of the water (g/mm^3)

2.4.2.3 Chloride Conductivity Test (DI Testing Manual, 2018)

This test is specifically intended to assess the resistance of concrete to ingress by chloride ions, such as would occur in marine environments. It is very sensitive to binder type and chemistry, and to the period of pre-saturation with chloride solution. It is intended as a test to allow proper material selection and mix optimisation prior to construction (Mackechnie, et al., 1999).

The South African chloride conductivity test apparatus consists of a two-cell conduction rig in which concrete core samples are exposed on either side to a 5 M NaCl solution. The core samples are preconditioned before testing to standardise the pore water solution (oven-dried at 50° C followed by 24 hours of vacuum saturation in a 5M NaCl chloride solution). The movement of chloride ions occurs due to the application of a 10 V potential difference. The chloride conductivity is determined by measuring the current flowing through the concrete specimen (Ballim, et al., 2009). A schematic of the Chloride Conductivity cell is shown in Figure 2-21.

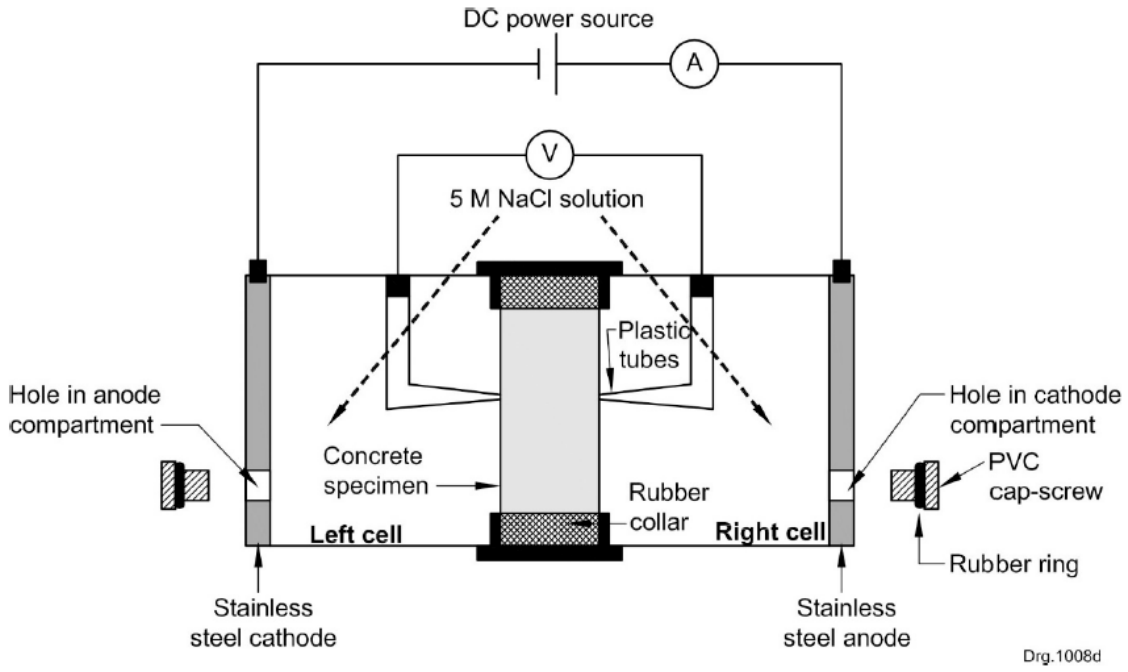


Figure 2-21 - Schematic of chloride conductivity cell (DI Testing Manual, 2018)

The chloride conductivity (σ) of the specimen is calculated in mS/cm according to Equation 2.17. The total chloride conductivity is calculated as the average of the chloride conductivity of the four specimens. A lower CCI indicates concrete with low chloride conductivity with more resistance to transportation of harmful ions through diffusion, and consequently an increase in potential durability

$$\sigma = \frac{id}{VA} \tag{2.17}$$

Where i = Electric current (mA)
d = Average thickness of specimen (cm)
V = Voltage difference (V)
A = Cross-sectional area of specimen (cm^2)

2.5 DURABILITY DESIGN OF CONCRETE STRUCTURES

For new structures, the service life is defined as "the time which a concrete fulfils its performance requirements, without non-intended maintenance" (Nilsson, 2003). Temporary structures and replaceable structural elements will have lower service lives (i.e. 1-25 years) while buildings and bridges may have higher service lives (i.e. 50-100 years). The service life requirements are either given by the owner of the structure or taken from codes of practice for the given structure (FIB, 2006).

Many concrete structures fail to fulfil their intended service life, resulting in major expenses for the owners of structures for maintenance and rehabilitation. There is, therefore, great value and incentive in extending the service life of new structures through a more detailed durability design approach. Modern approaches to the design of concrete structures include durability requirements as well as strength requirements (Ballim, et al., 2009).

The leading causes of durability issues with concrete are a result of corrosion of reinforcement caused by carbonation or the ingress of chlorides. The Service life of concrete structures is, therefore, highly dependent on the penetrability of the cover concrete.

2.5.1 Prescriptive Approach to Durability Design

Conventional structural design codes follow a prescriptive approach to durability design whereby the requirements to meet durability are specified for the Designer and Contractor

(SANS 10100-1, 2000; TMH7, 1981; BS 8110-1, 1997). This approach is limiting since it is difficult to measure or control the presumed durability which may require routine inspection of the structure for several years after completion. There are no real determinations on the concrete structure's resistance to the penetrability of harmful substances. This approach also assumes a direct correlation between compressive strength and durability since the acceptance of the as-built structure is based solely on strength requirements. Several researchers have shown this assumption to be invalid (Al-Amoudi, et al., 2009; Burrows & Mehta, 2001).

Durability specifications in the Standard Specifications for Highway Bridges used in Southern Africa (COLTO, 1998) follow the prescriptive method for ensuring concrete durability by limiting cover-to-steel, water-binder ratio, cement content and binder type. Mix designs are prepared within the set limits and target strengths specified from the limits in mix parameters. Additional minimum requirements are given for placing, compacting, and curing the concrete. Should the concrete meet the cube strengths specified as well as procedures for casting and curing, it will be considered as achieving the required durability (COLTO, 1998).

2.5.2 Performance-Based Durability Design

A performance-based approach includes the measurement of relevant concrete properties at the construction stage to assess the potential durability of the concrete structure. These concrete properties relate to the transport properties of the concrete and not only the strength (Muigai, et al., 2010). The as-built structure can also be checked to meet the requirements set out at design stage The FIB Model Code for Service Life Design (2006) gives four approaches to performance-based durability design: the full probabilistic approach, the partial factor design approach, the deemed to satisfy approach, and the avoidance of deterioration approach.

a) Full probabilistic approach

The full probabilistic approach is based on probabilistic models for deterioration and material resistance, using appropriate test methods with statistical evaluation. Full probabilistic approaches are not yet implemented as standalone methods (Alexander & Beushausen, 2019).

As shown in Figure 2-22, probabilistic design requires the time-dependent performance (R(t)) ('Resistance') of the structure to be larger than the target design requirements (S(t)) ('Loading'), as expressed in Equation 2.18:

$$R(t) - S(t) > 0 \tag{2.18}$$

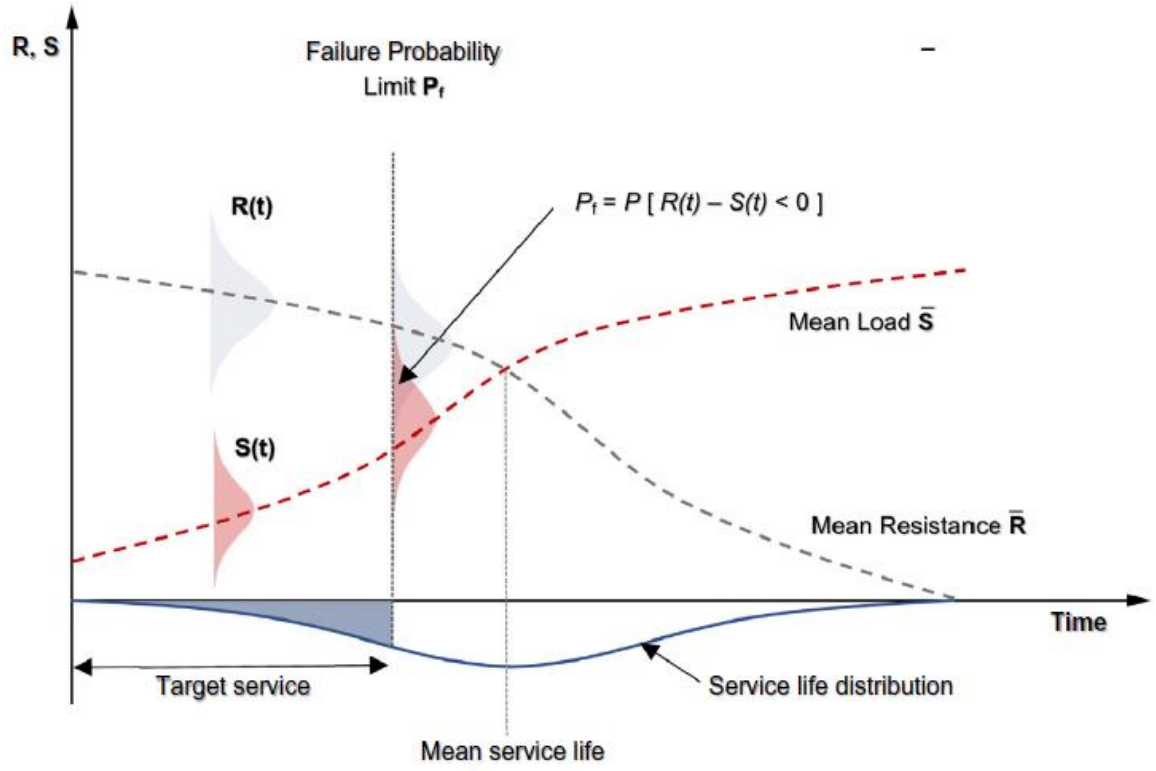


Figure 2-22 - Schematic representation of probabilistic service life design (Alexander & Beushausen, 2019)

The designer will use suitable service life prediction models (Section 2.5.4) to determine whether the structure will meet the desired lifespan.

b) Partial factor design approach

The partial factor approach (limit state method) to durability design is a semi-probabilistic method relying on limit state methodology. Limit states are commonly used in structural design codes (TMH7, 1981; SANS 10100-1, 2000; AS/NZS 1170.0, 2002) but less commonly used for durability design. The partial factor method incorporates mathematical models to describe the deterioration of the concrete structure up to a specified limit. The model consists of a set of variables that represent the loss of performance with time. The variables define material properties, geometric properties and environmental exposure conditions (Muigai, 2008).

ISO 13823 has incorporated durability design in the limit state approach for structural design (ISO 13823, 2008). In addition to the structural ultimate limit state (ULS) and serviceability limit state (SLS), an additional durability limit state (DLS) has been introduced. The DLS describes the point at which durability failure occurs (e.g. corrosion initiation).

c) Deemed-to-satisfy approach

The deemed-to-satisfy approach is an extension of the prescriptive approach to durability design, with a more systematic method of selecting material constituents. In addition to this, target values for performance tests on concrete transport properties are often specified. EN206-1 (2000) is an example of a deemed-to-satisfy approach. In this approach, the designer is required to select the most critical expected environmental exposure conditions and select the material parameters based on this environment. The difference between the deemed-to-satisfy approach and the prescriptive approach to durability design is that it relies on physical and chemical models for concrete rather than practical experience (Alexander & Beushausen, 2019).

d) Avoidance of deterioration approach

The avoidance-of-deterioration approach relies on reducing the risk of deterioration, specifically by the selection of materials that resist deterioration. These materials will either reduce the penetrability of the concrete microstructure or prevent corrosion of the embedded reinforcing steel. There are several methods used to avoid the deterioration of concrete structures (Schiessl, 1996):

1. Change the environment: Includes the application of protective coatings or membranes over the concrete to reduce the concrete's exposure to harmful substances in the environment.
2. Select non-reactive materials: This can be the reinforcing material (e.g. stainless steel, aluminium, galvanised steel) or the concrete constituents (e.g. sulphate resistant cement)
3. Inhibit the reaction: This involves protective measures that slow down, delay or reduce the rate of reaction (e.g. cathodic protection, air entrainment, corrosion inhibitors)

2.5.3 Durability Index based performance approach in South Africa

The DI approach in South Africa integrates service life models with DI tests to give performance specifications for concrete structures depending on the exposure environment. The approach uses a combination of the full probabilistic, partial factor, and deemed-to-satisfy approaches to durability design (Ballim, et al., 2009). The specifications rely on service life prediction models which have been validated with experimental results from several structures over a 20 year period (Mackechnie, 2001).

The DI tests in South Africa have been developed as simple, quick tests that are easy to carry out. They require minimal sample preparation and are conducted at an early age. Two of these test (Oxygen Permeability Index and Chloride Conductivity) have recently been incorporated into the South African National Standards (SANS 3001-CO3-2, 2015; SANS 3001-CO3-3, 2015).

The DI tests were produced for elements both in the laboratory and on-site concrete. Performance-based specifications were developed by Alexander, et al (2008), which consider a matrix of factors in a multi-factor approach to durability design. The factors included in the specifications are the environmental exposure conditions, required service life of the structure and the material properties of the concrete (Alexander, et al., 2008). These DI based performance specifications have been implemented in major freeway projects by the South African National Roads Agency Limited (SANRAL). This approach limits DI values based on the type of structure and binder type used in the concrete. If the concrete from site meets these requirements, it is "deemed-to-satisfy" the minimum durability requirements (Alexander, et al., 2008).

The limiting DI values are dependent on the environmental exposure conditions classified according to the EN206-1 categories and adjusted for South African conditions. The different classes of environment exposure are detailed in Table 2-2.

Table 2-2 - Environmental classes for South African conditions (adapted from EN206-1) (Alexander, et al., 2008)

Designation	Description
Carbonation Induced Corrosion	
XC1	Permanently dry or permanently wet
XC2	Wet, rarely dry
XC3	Moderate humidity (60-80%) – (External concrete sheltered from rain)
XC4	Cyclic wet and dry
Corrosion induced by chlorides from seawater	
XS1	Exposed to airborne salt but not in direct contact with seawater
XS2a*	Permanently submerged
XS2b*	XS2a + exposed to abrasion
XS3a*	Tidal. Splash and spray zones Buried elements in desert areas exposed to salt spray
XS3b*	XS3a + exposed to abrasion

**These subclauses have been added for South African coastal conditions*

For carbonating environments, the limiting values of OPI are given in Table 2-3. These are relevant for exposure conditions XC3 and XC4 (EN206-1, 2000). For the less critical exposure conditions XC1 and XC2, carbonation-induced corrosion is unlikely if minimum cover requirements are met. It is also evident from the Table that the designer can reduce the limiting OPI value by increasing the concrete cover.

Table 2-3 - Limiting OPI Values ('deemed-to-satisfy' approach) for carbonating conditions (Alexander, et al., 2008)

		Monumental Structures	
	Common Structures	Option 1	Option 2
Service Life (Years)	50	100	100
Minimum Cover (mm)	30	30	40
Minimum OPI	9.70	9.90	9.70

For marine environments, with concrete exposed to chlorides, the limiting chloride conductivity values are determined according to cover depth, service life and binder type used (Table 2-4). The limiting values are based on concrete cover of 50 mm.

Table 2-4 - Minimum Chloride Conductivity values ('deemed-to-satisfy' approach) for chloride conditions (Alexander, et al., 2008)

EN206-1 Class	70:30 CEM I: Fly Ash	50:50 CEM I: GGBS	50:50 CEM I: GGCS	90:10 CEM I: CSF
XS1	2.50	2.80	3.50	0.80
XS2a	2.15	2.30	2.90	0.50
XS2b, XS3a	1.10	1.35	1.60	0.35
XS3b	0.90	1.05	1.30	0.25

A two-level approach has been developed for DI based specifications where the responsibility of production of durable concrete is shared by the concrete supplier and the contractor. The concrete supplier is required to select appropriate constituent materials and proportion them suitably for a durable mix with a potential exceeding the design specifications. The contractor is then required to process the concrete on site adequately (placing, curing and finishing) to ensure that they achieve the limiting DI value provided in specifications (Alexander, et al., 2008).

2.5.4 Service life models

The service life of concrete structures is often specified as the time to corrosion activation as per the three-stage corrosion model (2.3.2). Several models have been produced for the quantification of concrete service life in a certain environment based on the exposure conditions and concrete material properties. Some of the more common service life prediction models are summarised in the following section.

2.5.4.1 International service life prediction models

a) Duracrete (DuraCrete, 1998)

DuraCrete is shortened form of *Dura*bility Design of Con*crete* Structures. The DuraCrete model is based on performance, reliability and design service life. The model uses the same approaches as structural design codes for limit state design. For the time to corrosion initiation, the model uses Crank's solution to Fick's second law of diffusion but considers the variability of the input parameters such as diffusion coefficient, cover depth and chloride threshold level. The model requires an accurate definition for environmental actions, and material parameters for concrete and reinforcement (Alexander, et al., 2012).

b) ClinConc (Tang, 1996)

ClinConc (a shortened form for *Cl-in-Con*crete) is a Swedish performance-based numerical model. The model takes chloride binding into account and uses the percentage of free chlorides in the calculations. The model is based on Crank's solution to Fick's second law of diffusion, using an input from the Rapid Chloride Migration test or the Bulk Diffusion Test (summarised in 2.4.1.2).

c) Life-365 (Life-365, 2018)

Life-365 is a North American probabilistic computer-based approach to service life prediction which uses a 1D and 2D finite difference methods to solve Fick's second law of diffusion. All calculations are theoretical and do not use values obtained from actual tests on the concrete structure. The model is typically used at the design stage to determine most suitable protective measures and parameters for a durable concrete design.

For predictions on time to corrosion initiation, the following parameters are input:

- Geographic location;
- Type of structure and nature of exposure;
- Depth of clear concrete cover to the reinforcing steel

- Details of each protection strategy scenario, such as water-cement ratio, type and quantity of supplementary cementitious materials and corrosion inhibitors, type of steel and coatings, and type and properties of membranes or sealers

2.5.4.2 South African Chloride Ingress Model (Mackechnie, 2001)

The South African Chloride Ingress model is a performance-based model for reinforced concrete in the marine environment. The model uses early age chloride diffusion results from the South African chloride conductivity test (Section 2.4.2.3). The chloride conductivity values are then converted to long-term diffusion coefficients used in Crank's solution to Fick's second law of diffusion, with additional input for cover depth and exposure environment. Marine environments in this model are classified according to EN206-1 (2000), modified for South African conditions. The steps in the durability design of concrete structures using the South African chloride ingress model are shown in the flow diagram in Figure 2-23.

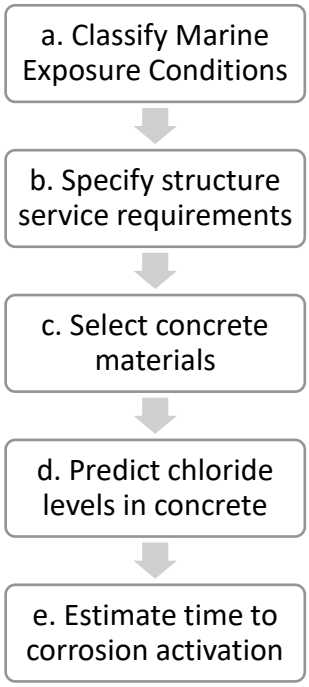


Figure 2-23- Flow diagram of design steps using the South African Chloride Ingress Model

a) Classify marine exposure conditions

The marine exposure category is classified according to guidelines adapted from EN206-1 (2001). Environments are classified as moderate, severe, very severe or extreme dependant on the type of exposure to seawater (splash zone or spray zone) and the proximity to the sea.

b) Specify structure service requirements

Service and structural requirements are defined before carrying out durability design since these factors may influence the design input. Factors include required service life, concrete grade, cover depth, crack-width limitations, dimensional stability (i.e. creep and shrinkage) and long-term risk of cracking.

c) Select concrete materials

Two approaches may be used when selecting concrete materials for marine structures: laboratory-based investigations and desk-top studies.

Laboratory-based investigations will require the determination of 28-day chloride conductivity values at the design stage. The results of the chloride conductivity test can be used to predict long-term diffusion coefficients using a nomogram developed by Mackechnie (2001).

The desktop study will use charts of predicted long-term diffusion coefficients according to the type of binder in the mix and the grade of concrete specified. Data on diffusion coefficients were developed from tests on Western Cape concretes with initial 3-days moist curing and standard compaction. The desk-top study approach does not allow for local factors such as material quality and site practice.

d) Predict chloride levels in concrete

The surface concentration of chlorides is required before a prediction can be made on the chloride levels in the concrete. Surface concentrations can be estimated based on the concrete exposure conditions and type of binder using tabulated values from measurements of marine structures in Western Cape conditions.

Once the diffusion coefficients and surface concentrations of chlorides have been determined, chloride levels within the concrete can be determined according to Equation 2.4 (repeated here):

$$C_{x,t} = C_s\left[1 - \text{erf}\left(\frac{x}{2\sqrt{D_a t}}\right)\right] \qquad (2.4)$$

e) Estimate time to corrosion activation

Time to corrosion activation is calculated as the time to the percentage chlorides reaching the chloride threshold level (Section 2.3.4.2) at the location of the steel. As noted previously, this is considered conservatively to be at around 0.4% by mass of binder.

2.6 SUMMARY

The Chapter reviewed several aspects of concrete durability, particularly in the marine environment exposed to chlorides. In order to successfully predict and estimate potential durability and service life of structures, several concepts were defined. These concepts include aspects of the concrete system such as the environmental exposure conditions and the factors that affect the microstructure of the concrete. The quality of the concrete microstructure will affect the concrete's ability to resist the penetration of harmful agents through the pore structure.

The most common cause of concrete durability failure in the marine environment is corrosion of steel reinforcement. Steel embedded in concrete will corrode expansively in the presence of moisture and oxygen. This corrosion is induced by carbon dioxide or chlorides which break down the natural protective film surrounding the steel. For chloride-induced corrosion, the initiation phase is when the chloride levels at the steel reach a chloride "threshold level".

Key to predicting the service life of concrete structures is the quantification of the transport properties through the concrete. There are accelerated laboratory-based tests that can be carried out on concrete specimens to determine the transport properties of the concrete. In South Africa, Durability Index (DI) tests have been developed to determine the concrete's resistance to permeation, sorptivity, and diffusion. These tests have recently been incorporated into the South African National Standards.

With some details on the transport properties of concrete, the expected service life of concrete structures can be determined as the time to corrosion activation. Several models have been developed for the determination of service life. In South Africa, a chloride ingress model has been developed for the service life prediction of concrete in the marine environment. The input parameters are the expected surface concentration of chlorides, the concrete cover, the binder type and the calculated chloride conductivity of the concrete. This model is also used at the designs stage to specify concrete mix specifications in the deemed-to-satisfy approach.

3 METHODOLOGY

3.1 PROJECT BACKGROUND

3.1.1 Introduction

Durability assessment was carried out on concrete samples from newly constructed bridges on the Swakopmund-Walvis Bay freeway upgrade in Namibia (Figure 3-1). The project consists of four post-tensioned concrete bridges with overall deck lengths ranging between 56 and 109 m. The two bridges over the Swakop River were constructed with more than 2,500 m^3 of concrete each while the other two interchange bridges (MR52 Interchange and Swakopmund Interchange) exceed 1,700 m^3 of concrete each. The total cost of the four bridges was approximately 110 Million Namibian dollars (N$).

The design specifications were intended to produce a concrete structure with a design lifespan of at least 100 years (TMH7, 1981). The project specifications followed a prescriptive durability design and specification approach, setting restrictions and limits on concrete-related aspects such as binder type and content, maximum water-to-cement ratio, placement, compaction, and curing. Figure 3-2 summarises the design and construction process adopted in the project.

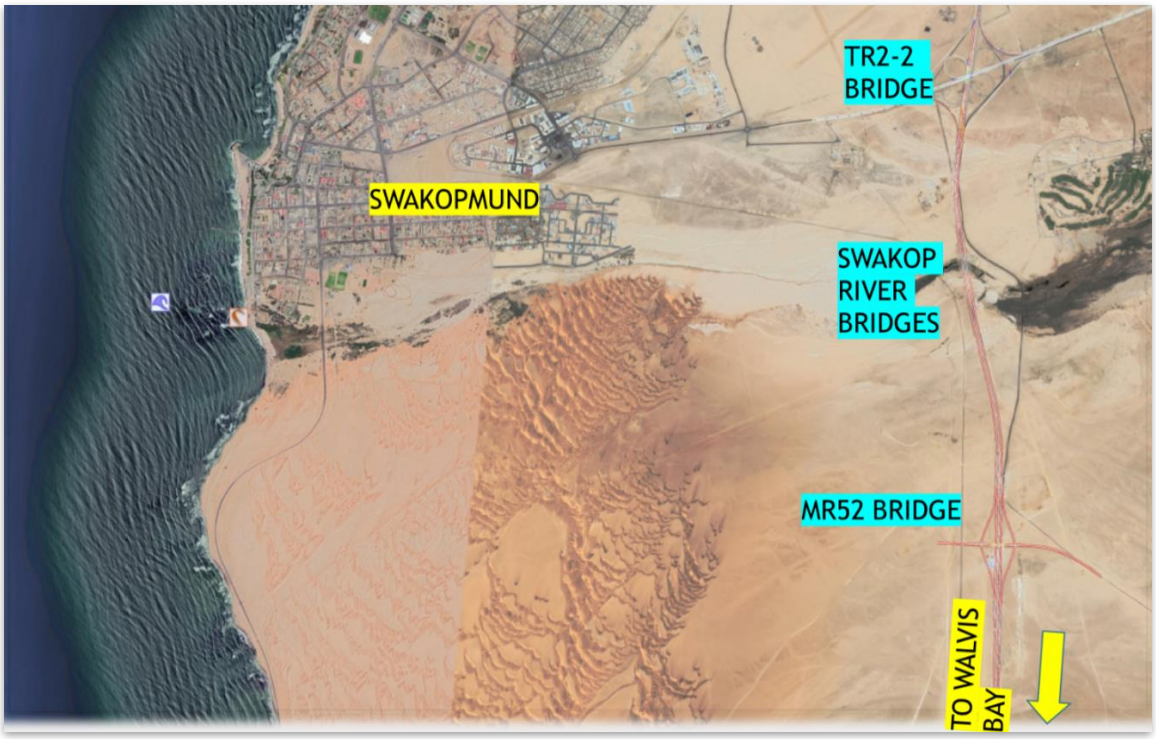


Figure 3-1 – Bridge locality plan

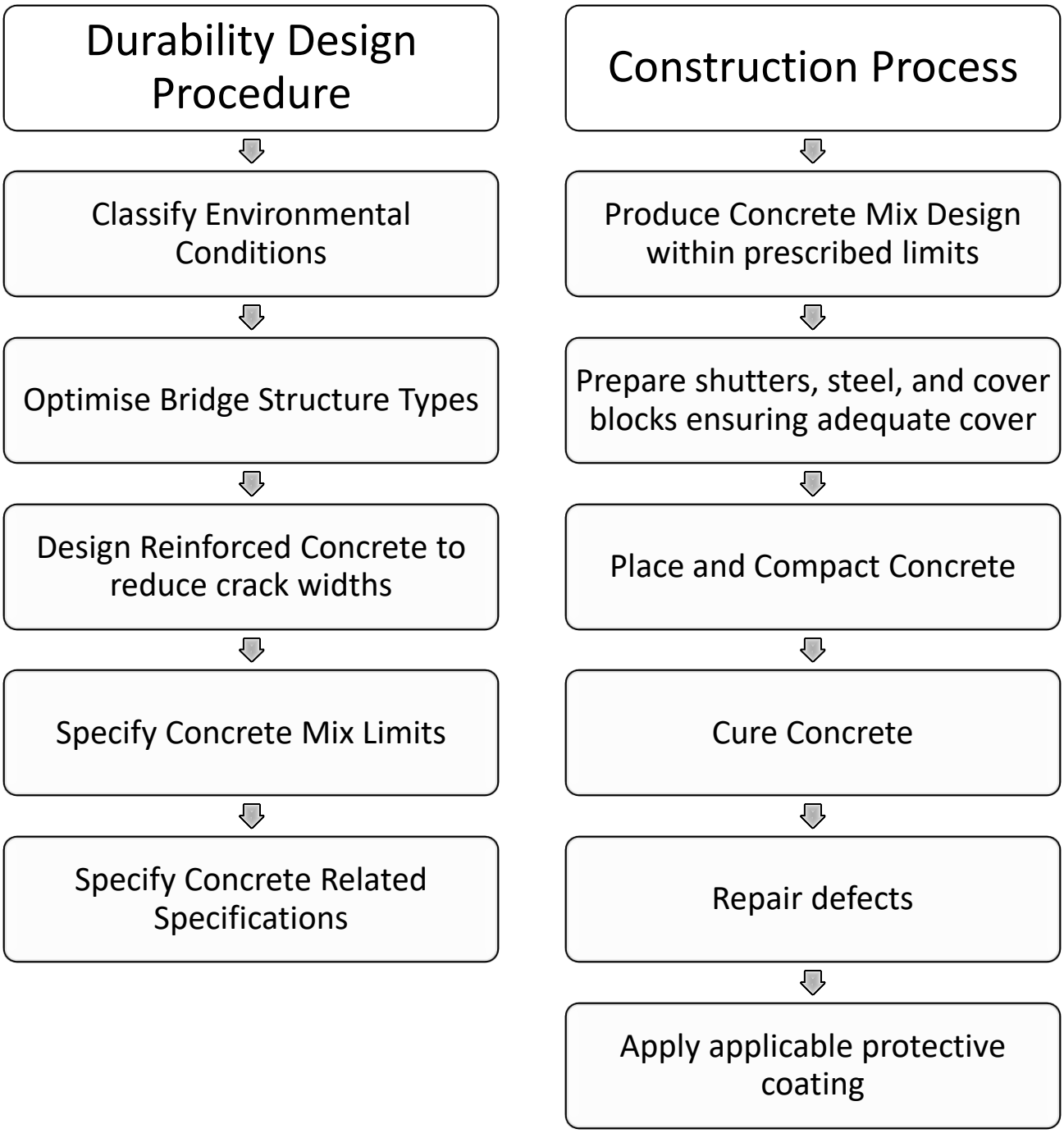


Figure 3-2 - Flow chart for design and construction of durable concrete structures

3.1.2 Environmental exposure conditions

The climate along the Namibian Coast is very conducive to corrosion of steel. Research has shown the Namibian Coast to be one of the most corrosive environments in Southern Africa (Bulley, 1986). A relevant case study on the corrosiveness of the Namibian Coast is the Omaruru River Bridge (1982) in Henties Bay. This bridge was constructed with 45 MPa concrete in the deck, and 50 mm cover. However, investigations only eleven years after the construction of the bridge indicated severe chloride attack on the concrete and main reinforcing steel bars in the deck and piers. Repair work was undertaken on the spalled concrete and the main reinforcing steel bars needed to be replaced. (Hess, 1996)

Long-term corrosion rates at the Walvis Bay military base (20 km from the coast) were recorded to average 0.85 mm/year for mild steel (Callaghan, 1991). This corrosion rate is substantially higher than recorded corrosion rates in Cape Town and Durban, measured in the same study. Several probable aspects of the coastal climate contribute to the harsh environment. These include the high relative humidity, the wetting-drying action of the fog, and the abundance of chlorides in the atmosphere (Bulley, 1986).

The coastal towns experience a high relative humidity ranging between 69 and 96% in Swakopmund and between 73 and 95% in Walvis Bay while the average rainfall between Swakopmund and Walvis Bay is only 10 mm per annum (Robertson, et al., 2012). This humidity is substantially higher than the dry inland with the relative humidity in Windhoek (18 -51%) and comparable to other corrosive coastal environments such as Durban (73-83%) and Cape Town (71-83%) (climatemps.com, 2017).

Strong South-westerly winds are carried over the Namibian coastline due to the South Atlantic anticyclone over the ocean to the Southwest of Namibia. The winds carry chlorides inland and cause foggy conditions. Between 113 and 140 fog days per annum have been recorded on the stretch of coastline between Swakopmund and Walvis Bay (Olivier, 1995). The foggy conditions exacerbate the corrosive environment due to cyclic wetting and drying of structures in the area.

3.1.2.1 Environmental Exposure Classification

Assessment of the environmental conditions indicated chloride-induced corrosion to be the most critical parameter affecting the durability of the concrete. Based on Eurocode EN206-1, the engineer classified the environment according to class XS1 for “structures exposed to airborne salt but not in direct contact with seawater or inland saline waters” (EN206-1, 2000).

The two critical sources of chlorides in this environment are:

a) Air-borne chlorides

b) Chlorides deposited on the surface of the deck from trucks transporting salt along the road and from the tyre tracks of vehicles approaching from the adjacent salt-gravel roads.

3.1.3 Design Considerations

3.1.3.1 Bridge Types

The final bridge design consisted of steel-reinforced concrete foundations, piers and abutments with a post-tensioned continuous bridge deck. The primary bridge elements are indicated in Figure 3-3.



Figure 3-3 - Bridge elements

a) Foundations

The interchange bridges consist of reinforced concrete spread-footing foundations generally supported by either soil with a 600 kPa bearing capacity or cast on bedrock. Due to the poor founding conditions in the riverbed, the Swakop River Bridges were constructed on reinforced concrete piles socketed between 1 and 4 metres in bedrock.

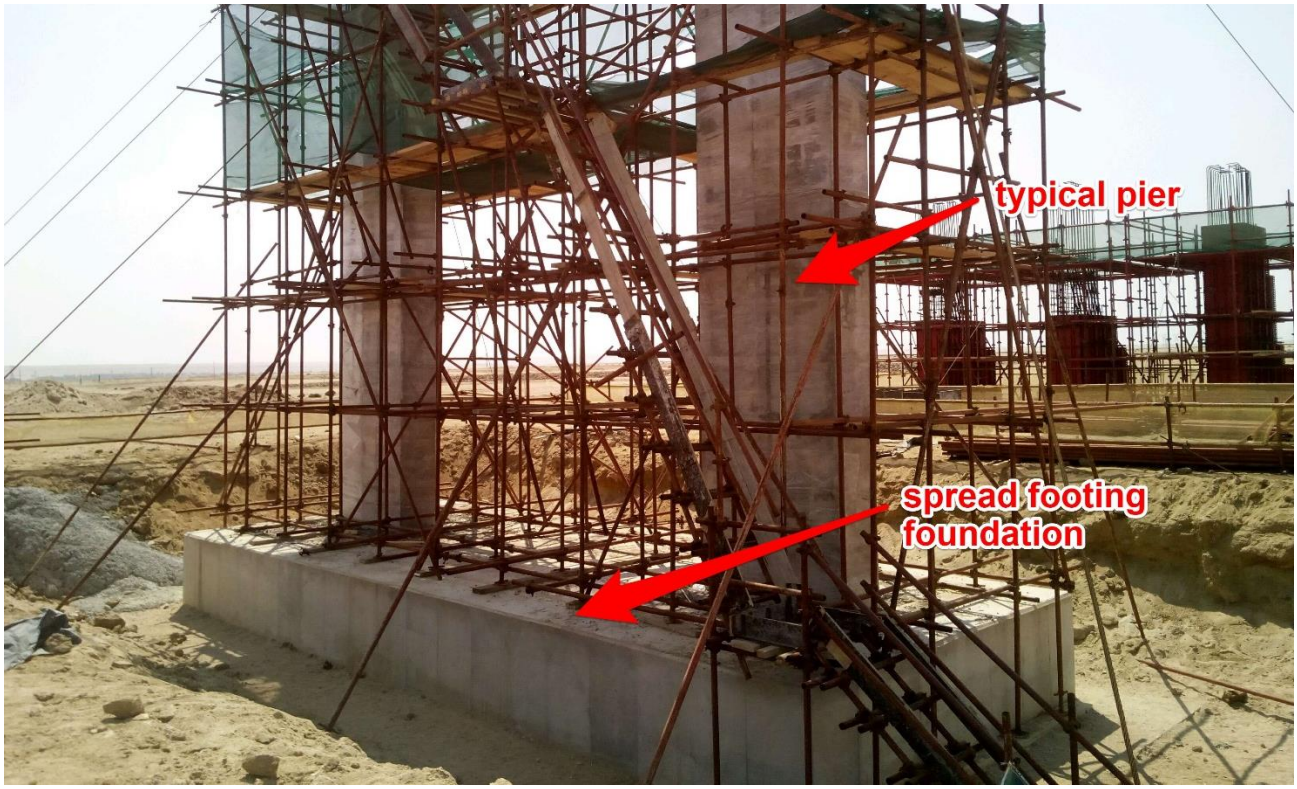


Figure 3-4 -Photograph of typical spread-footing foundations for interchange bridges



Figure 3-5 - Photograph of reinforcing steel bars extending from piles into the pile cap

b) Abutments

The abutments on the interchange bridges are spill-through abutments (Figure 3-6) while the Swakop River Bridges consist of a solid wall abutments (Figure 3-7).

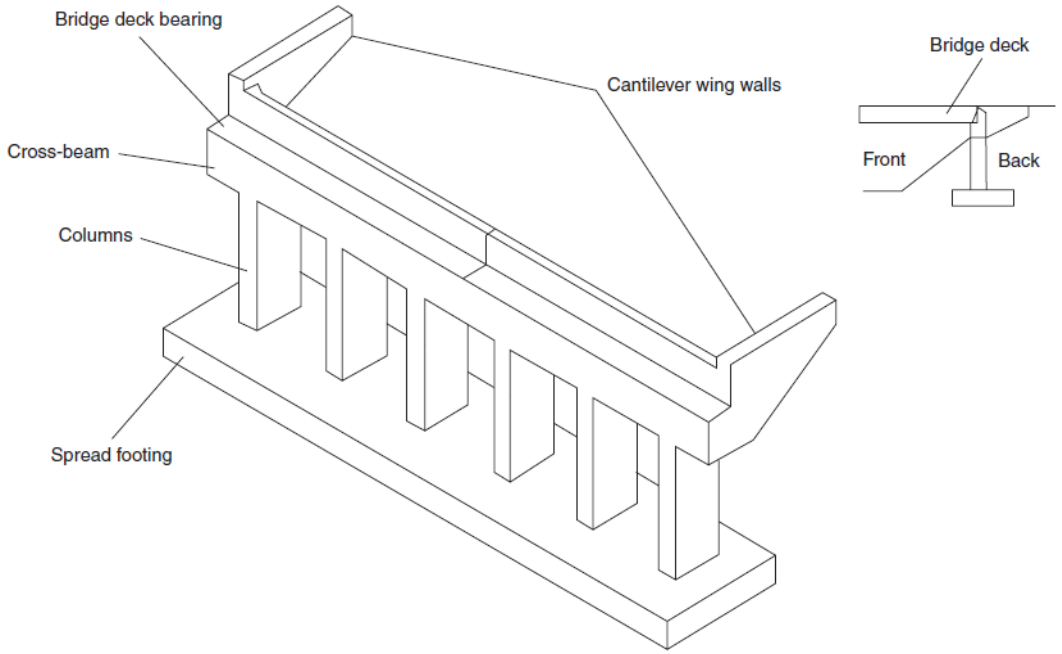


Figure 3-6 - Typical spill-through abutment (Lindsell, et al., 2008)

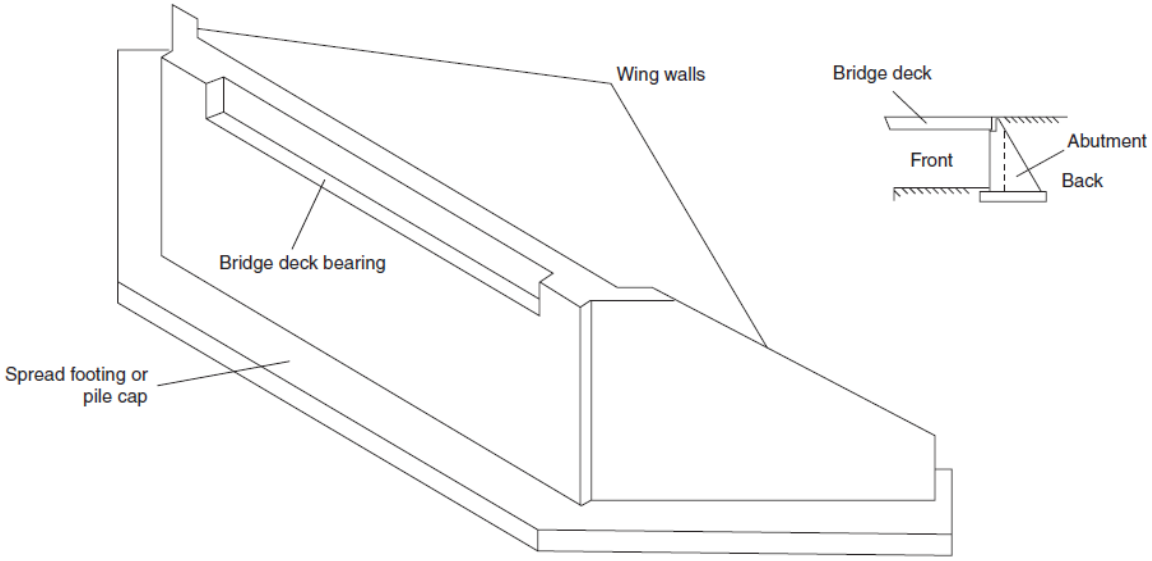


Figure 3-7 - Typical solid wall abutment (Lindsell, et al., 2008)

c) Piers

The piers are reinforced concrete hexagonal piers cast monolithically with the bridge deck (Figure 3-8). The river bridges were constructed with an additional solid wall between the pier columns, to channel water and prevent the build-up of debris against the piers.



Figure 3-8 - Typical bridge piers

d) Decks

All bridge decks are beam-and-slab structures with the beams internally post-tensioned and grouted (Figure 3-9).



Figure 3-9 - Typical deck cross-section

3.1.3.2 Cracking and Crack-width Limitations

The DI tests, as well as the South African Chloride Ingress Model, follow the assumption that aggressive agents move through the pore structure of the concrete and that the cover concrete prevents the penetration of these agents. The basis of this assumption is that the concrete is uncracked. The movement of aggressive agents through cracked concrete is beyond the scope of this study. It is critically important the design of concrete structures that theoretical design crack-widths are limited.

When the bridge structures were designed for this project, theoretical design crack-widths were reduced to below 0.2 mm by limiting the maximum tensile stresses in the concrete, increasing concrete cover, and decreasing the spacing between the reinforcing steel bars (TMH7, 1981). Critical cracks which were considered in the reinforced concrete designs included flexural and shear cracks, early thermal contraction cracks, and long-term drying shrinkage cracks.

a) Design for flexural and shear cracking

Structural design was carried out according to TMH7 (Parts 1,2 and 3), which classifies the environment as "severe" for "concrete surfaces exposed to driving rain or alternate wetting and drying" (TMH7, 1981). Flexural and shear cracks are dependent on the strain in the concrete and the distance from the expected crack location to the nearest reinforcing steel bar. Bridge members were designed to reduce the tensile strain and to reduce the spacing between the bars to ensure a maximum crack-width of 0.2 mm (TMH7, 1981). In addition, the bridge decks were designed as post-tensioned, ensuring only compressive forces throughout the depth of the deck for all load cases.

b) Early Thermal Cracks

Early thermal cracks occur when large concrete elements are cast against restrained surfaces. Large concrete elements experience a high initial temperature from within the member. The members contract as they cool down. Restraint to this thermal contraction will result in the development of tensile stresses. The concrete is still relatively weak in tension, and the stresses caused by restraint to temperature-related contraction can cause cracking (Clarke & Day, 2003).

Bridge elements particularly susceptible to early thermal contraction cracks are the bottoms sections of piers and abutments (Figure 3-10). The design theoretical crack widths were

reduced by placing additional longitudinal reinforcing at close spacing in the bottom portion of the wall or column adjacent to the footing as per design specifications (BD 28/87, 1987).

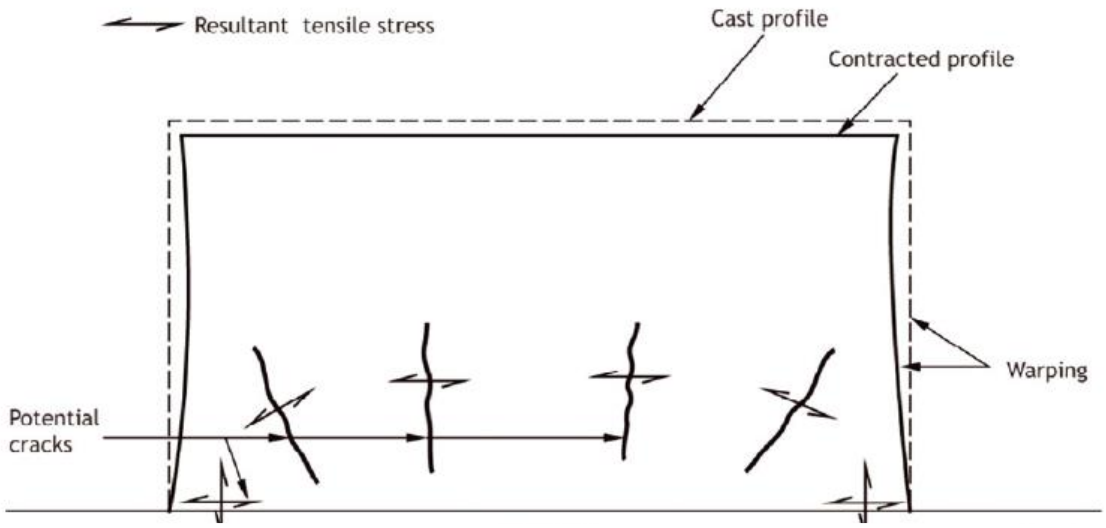


Figure 3-10 - Early thermal contraction cracks in concrete walls (The Concrete Centre, 2015)

c) Drying Shrinkage cracks

Drying shrinkage occurs as excess pore water not used in the hydration of cement is removed from the concrete as the elements dry. When this movement is opposed by a restraint, stresses will develop, and cracking will occur (Neville, 2011). Design for long-term shrinkage consisted of the close spacing of bars in a mesh configuration in tensile and compression zones in the structure. The reinforcing resists any local tensile strains that occur due to creep and shrinkage of the elements. The post-tensioning in the bridge decks and allowance of free movement at the abutments ensured that the long-term drying shrinkage in the bridge deck is considerably unrestrained and does not cause any cracks.

3.1.3.3 Concrete-related Specifications

The Consulting Engineers approached the project specifications from a prescriptive method of specifying parameters of the mix design along with specifying methods for placing, compacting, and curing the concrete. The project specifications are detailed below.

a) Binder Type and Content

The cement used for concrete was 50/50 blend of Portland cement and ground granulated blast furnace slag (GGBS). As noted in Chapter 2, GGBS influences the microstructure of the concrete resulting in concrete with fewer voids and decreased penetrability. GGBS was selected, particularly for its resistance to chloride-induced corrosion.

The minimum total cementitious content was limited to 380 kg/m^3. A high cement content was expected to result in concrete with more paste and subsequently, fewer voids. Dhir et al. (1996) found that a reduction in binder content results in an increase in the diffusion coefficients of chlorides into the concretes. The use of a minimum binder content may not ensure durability and it has been shown that equal durability can be achieved with the different binder types than with different contents (Dhir, et al., 1996).

The specified maximum w:c ratio was 0.50.

b) Concrete Cover

All concrete elements were designed with a minimum concrete cover of 50 mm. This is as per the recommendations of EN206-1 for structures classified as XS1 exposed to airborne chlorides (EN206-1, 2000). As noted in the literature review, cover to concrete is one of the most critical aspects in the prevention of corrosion of steel.

c) Concrete Placing and Compaction

All concrete elements were initially vibrated, and later re-vibrated before initial set to remove any additional air pockets within the structure. Re-vibration, typically one to two hours after placing the concrete, was considered an effective way of eliminating defects caused by bleeding and the prevention of plastic shrinkage and plastic settlement cracks. This was important in members with congested reinforcing and areas with changes in dimension (e.g. the beam-slab junction in the bridge decks). Re-vibration is not considered harmful provided the concrete is still plastic enough to respond to vibration. (Kellerman, 2009).

d) Curing Methods

The project specifications required curing with an impermeable membrane for ten days. The specified membrane for horizontal surfaces (such as the top of foundations and bridge decks) was a 500-micron polyethylene sheeting while the vertical elements were wrapped in a plastic wrap film. Standard specifications would generally require seven days of moist curing, but the inclusion of 50% GGBS as binder required additional curing due to the slower hydration of the slag (COLTO, 1998). The contractor was also given the allowance of leaving the shutters in place as a curing method.

e) Waterproofing and Protective and Coatings

Figure 3-11 shows the protective coatings and waterproofing applied to the bridge elements. All above-ground concrete elements were coated with a silane-based water-repellent

impregnation cream. Research has shown that silane treatment can offer residual protection to concrete for up to twenty years (Christodoulou, et al., 2012). All concrete faces in contact with soil were coated with a bitumen-based protective coating to prevent the ingress of liquids into the pore structure of the concrete. The top surfaces of bridge decks were coated with a waterproof spray-on polyurea. Correctly applied polyurea is completely impenetrable, UV resistant with a 25-year working life (EOTA, 2010).

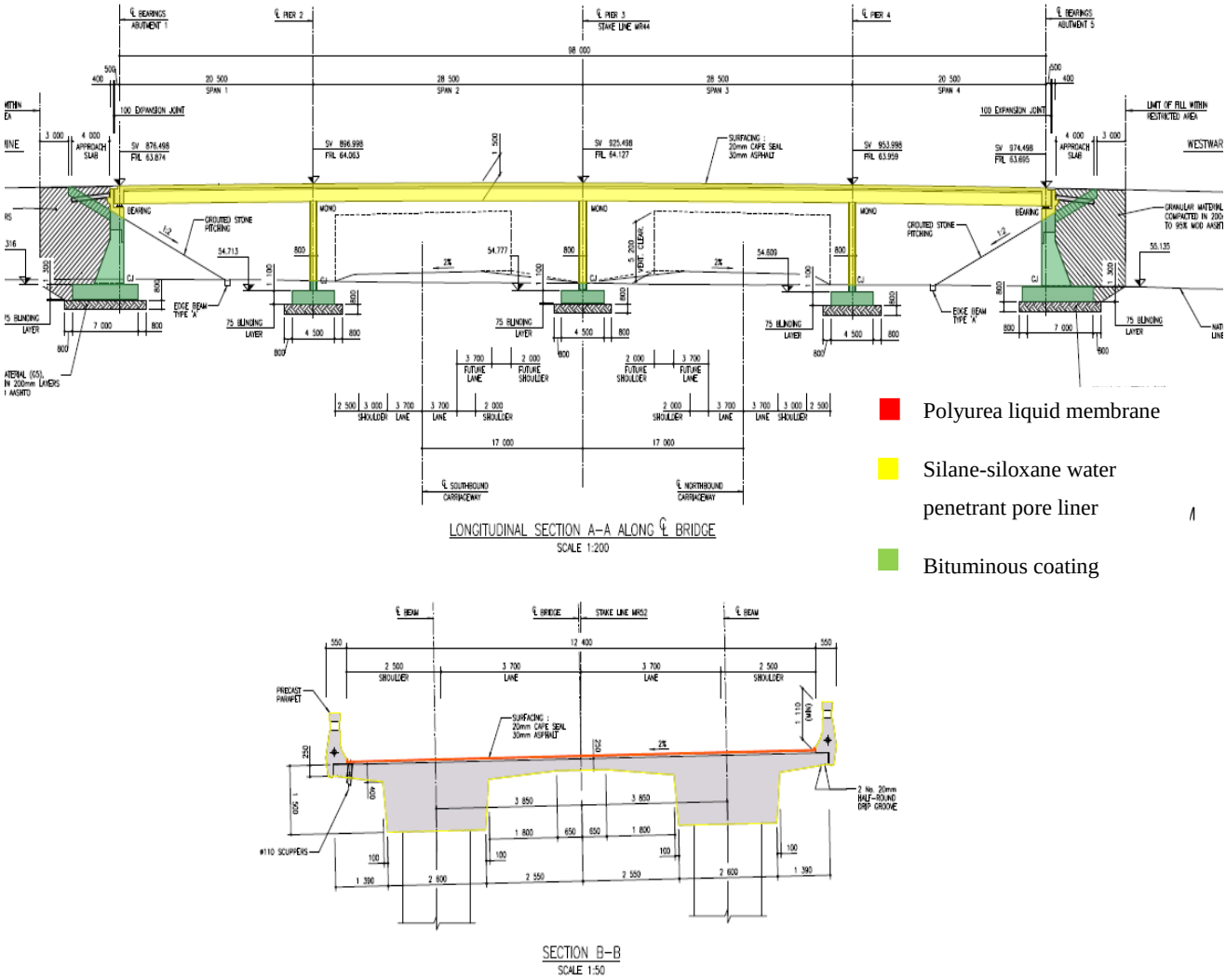


Figure 3-11 - Illustration of concrete protective coatings

3.1.4 Bridge Construction

Following the structural design and the concrete specifications detailed in Section 3.1.3, the contractor carried out the bridge construction. The construction process consisted of producing a concrete mix design within limits specified, followed by sound site practice as per the specifications and then carrying out remedial measures that did not reduce the potential durability of the concrete.

3.1.4.1 Concrete Mix Proportions

Following the limitations and specifications of 3.1.3.3, the ready-mix concrete supplier produced a mix as summarised in Table 3-1, which was used for all bridge elements.

Table 3-1 - Summary of concrete mix design

Design Constituents			
Constituent	Type and Source	Quantity (kg/m^3)	Specified Limit
Portland cement	CEM I 42.5 R	225	Min: 380 Max: 500
Slag	GGBS (imported from South Africa)	225	
Sand	Screened River Sand (Swakop River)	701	
19mm Stone	Granite (Walvis Bay)	1046	
Water	Potable water	185	
Admixture	Lignosulphonate superplasticiser (% binder content)	1.1%	
Design Outcomes			
w:b ratio		0.41	Max 0.50
Slump		120 – 180 mm	120 – 180 mm
28-day cube compressive strength		>60 MPa	Min: 40 MPa

a) Coarse Aggregate

Coarse aggregate was obtained from a local granite quarry. Petrographic analysis of the granite samples indicated 50-60% quartz crystals, 15-25% plagioclase, and 10-20% microcline amongst other minerals. Coarse aggregate was assessed for acceptance according to the limits in the Southern African standard on *Aggregates from Natural Sources* (SABS1083, 2002). A summary of the coarse aggregate grading is shown in Figure 3-12. As shown, the aggregates fall on the finer side of the grading envelope.

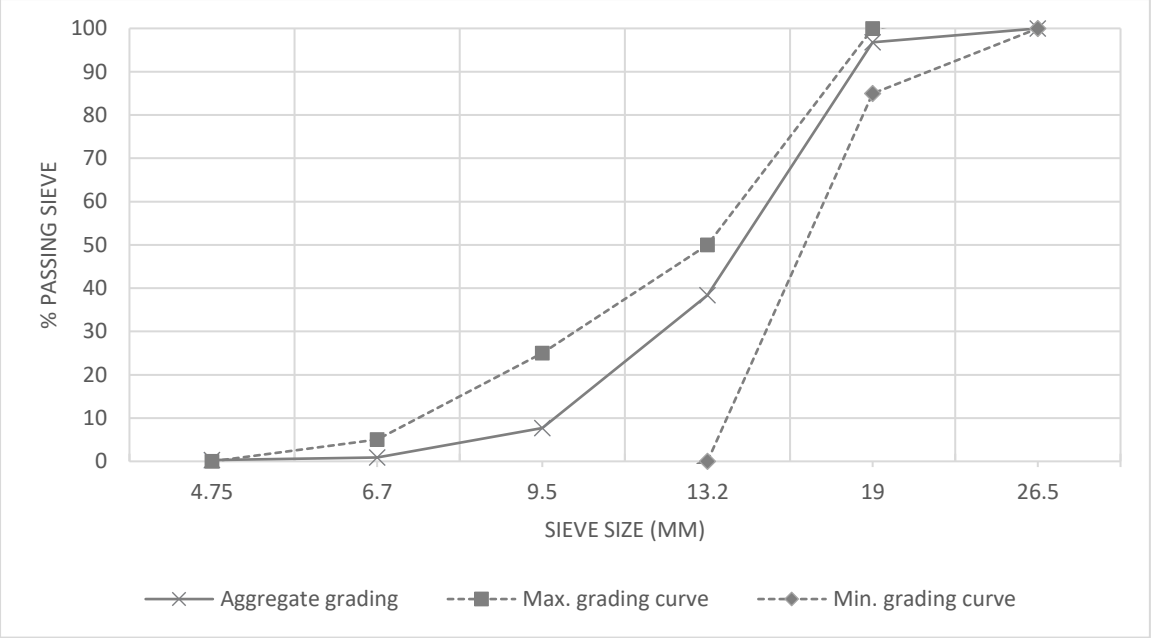


Figure 3-12 - Coarse aggregate grading curve

b) Fine Aggregate Analysis

Fine aggregate was obtained from river sand mined from the Swakop River. Petrographic analysis of the fine aggregate samples indicated 65-75% quartz crystals, 10-20% plagioclase, and 6% each of microcline and biotite, amongst other minerals. A summary of the fine aggregate grading is shown in Figure 3-13. The envelope used in the analysis is the sand grading envelope recommended by the *Cement and Concrete Institute of South Africa* (Grieve, 2009). As shown, the fine aggregate falls on the finer side of the grading envelope.

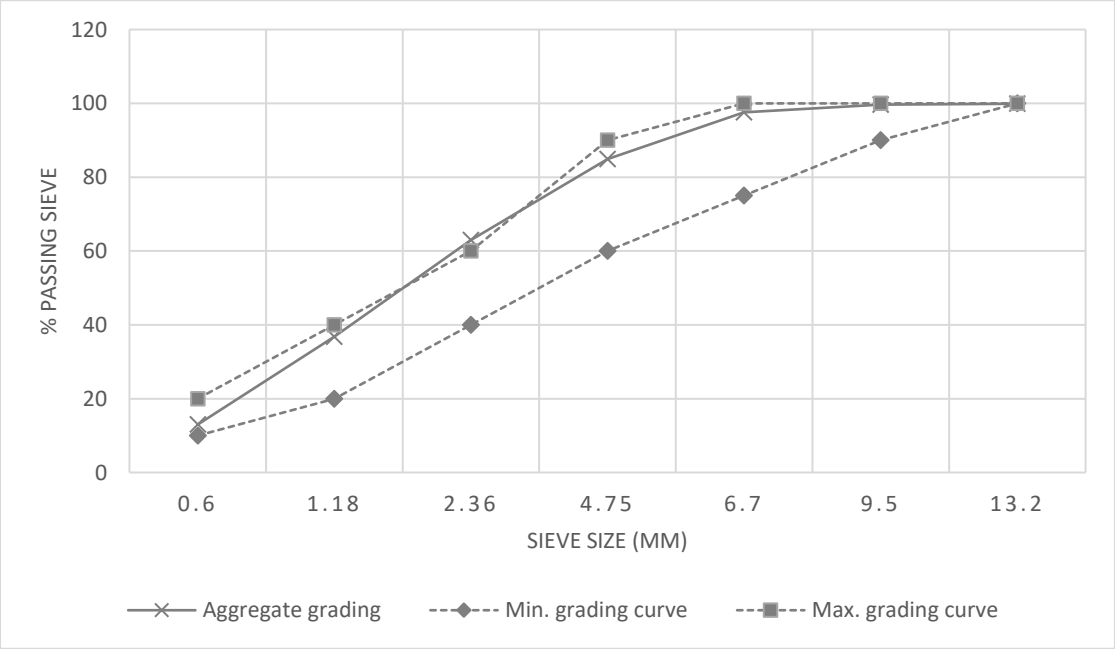


Figure 3-13 -Fine aggregate grading curve

3.1.5 Site Practices and Remedial Action

3.1.5.1 Variation in Material Properties

The cement and aggregates varied considerably after approval of the initial design. This was due to variabilities in the cement clinker at the source, and grading variations in the river sand. Throughout the project, samples of river sand showed a range in fineness modulus from 2.05 to 4.10 with a coefficient of variance of 24% as tested by the onsite laboratory.

Similarly, the cement properties, including fineness, water demand, setting time, and strength, varied considerably throughout the project. Figure 3-14 shows the variation in the cement properties over a year. These results were obtained by the production certificates obtained from the cement manufacturer. The percentage shown indicates the change from the initial reading.

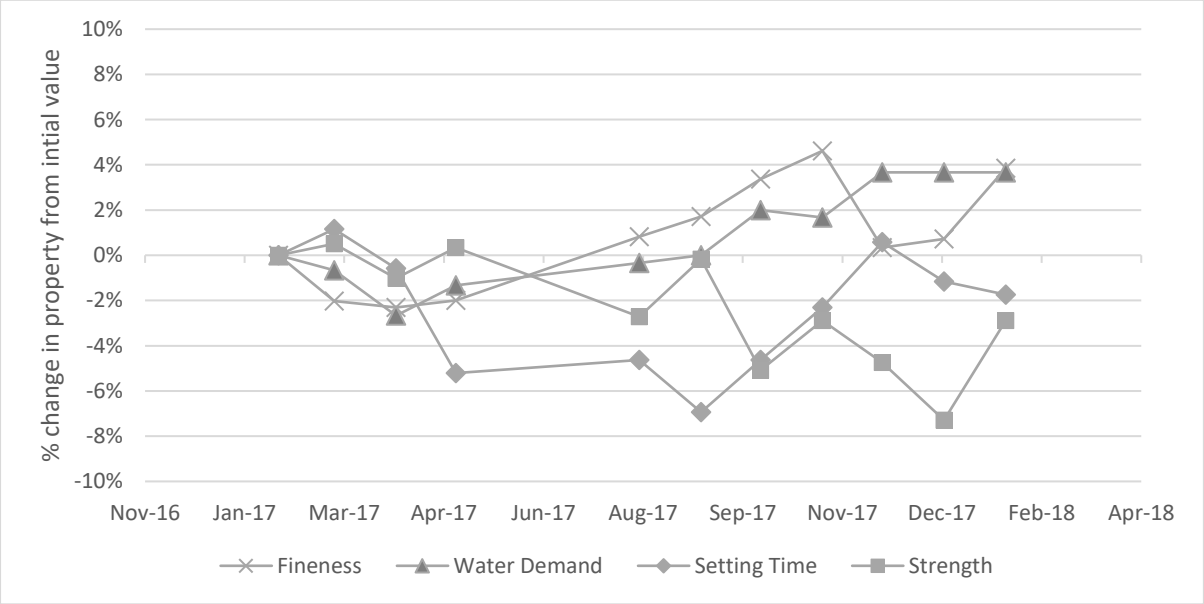


Figure 3-14 - Variation in cement properties over a year

3.1.5.2 Placing and Compaction

All concrete was produced by a local ready-mix concrete supplier and transported between and 20 km from the batch plant in 6 m^3 concrete trucks. Slump measurements did not always fall within the specified ranges and on several occasions. If the concrete arriving on site did not fall within the specified slump range, visual inspections on concrete consistency were carried out to confirm final acceptance or rejection of the concrete mix. This affected the compaction effort required. Figure 3-15 indicates an example of variations experienced in concrete slump measurements for one day's casting of a bridge deck. Due to the use of high-

range water-reducers, several slump measurements were above the design slumps. This required visual acceptance of the concrete consistency.

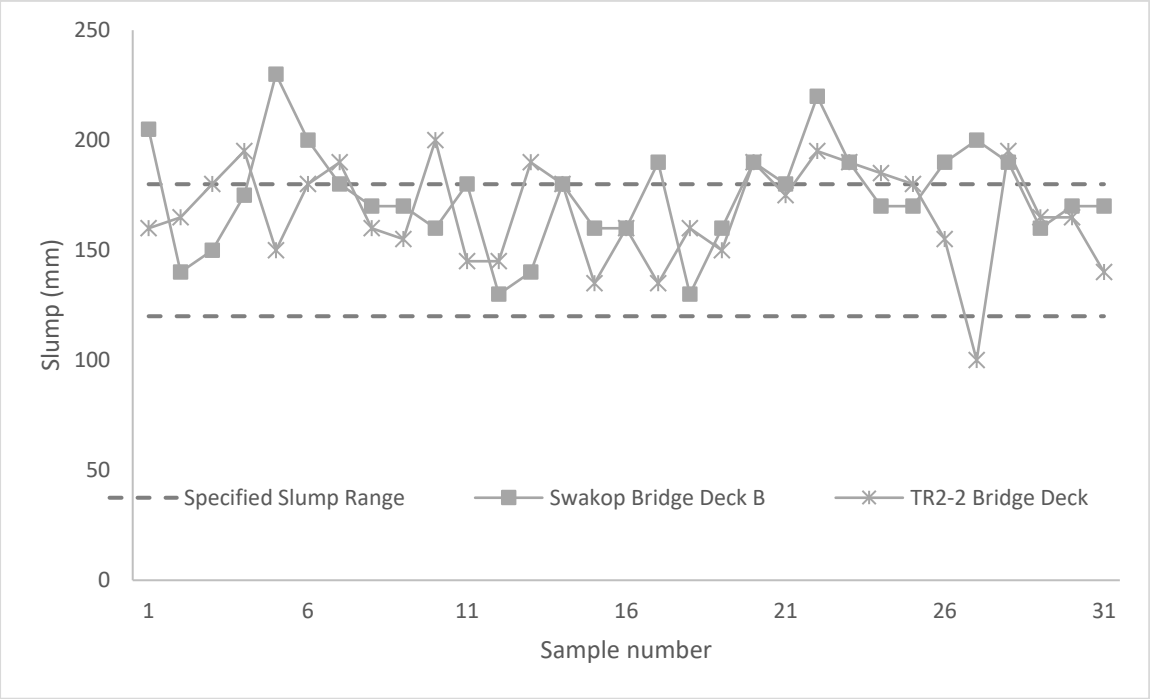


Figure 3-15 - Typical Variation in concrete slumps across several samples

The higher slumps created a risk of over-vibration of concrete, causing excessive bleeding and segregation. In several instances, this was observed when the shutters were stripped, as shown in Figure 3-16. Low slumps also caused early setting of concrete which at times resulted in cold joints as shown in in Figure 3-17.



Figure 3-16 - Bleeding on bridge pier



Figure 3-17 - Abutment footing showing inadequate vibration and cold joint

3.1.5.3 Curing

All curing was specified as either impermeable plastic membranes or leaving shutters in place for ten days. The contractor's approach for horizontal members (e.g. bridge decks) was to leave the shutters, and support work for the full ten days and keep the top unformed surface covered with plastic for this duration. For the vertical members, the general approach was to leave the elements in formwork for two days and wrap in an impermeable membrane for a further eight days for the total of ten days curing, although this practice varied from time to time with certain larger members being cured by formwork for the full ten days.

In practice, acceptable curing was not always efficiently implemented. This was a major concern when impermeable membranes were used during periods of heavy wind. Figure 3-18 shows the ineffective curing with an impermeable membrane during heavy winds.



Figure 3-18 - Inefficient curing with impermeable membrane

3.1.5.4 Repair of Defects

The construction practice did not always result in concrete without fault or blemishes. On several elements, grout loss and poor compaction resulted in honeycombing or excessive blowholes which would inevitably require repair work. Small sections of excessive blowholes were generally repaired with a cementitious mortar in combination with a bonding agent. Larger areas of honeycombing were chipped back to expose the concrete surfaces and repaired with a combination of concrete primer and non-shrink grout.



Figure 3-19 - Honeycomb on bridge deck diaphragm



Figure 3-20 - Excessive blowholes on bridge abutment

3.2 INVESTIGATIVE APPROACH

The investigative approach was to determine concrete quality and potential durability of the structures using the results of a combination of strength, durability indexes and concrete cover measurements. Sampling and testing of concrete cubes were carried out following the standard specifications used on the project (COLTO, 1998; TMH1, 1986). Concrete sampling for DI testing and cover tests were based on the SANRAL Project specifications for durability of concrete (Appendix 7.1).

Based on these specifications, concrete samples for the various test methods are defined as follows:

1. Concrete Compressive strength:
   For concrete compressive strength, one sample consists of three test specimens (i.e. 3×150 mm test cube). Specimens were crushed at 28-days and a concrete cube strength determined for each test. The average result of the three test determinations was calculated as one test result.
2. Durability Index Testing:
   For DI testing, one sample panel consists of a rectangular prism (600×400×150 mm) of which eight cores (70 mm diameter) were extracted for DI testing. The eight cores were cut to size (30 mm thick) and tested as per the DI Testing Manual (2018). Four of the discs were used in the Oxygen Permeability Index (OPI) test and Water Sorptivity Index (WSI) test. The same disc tested for OPI was tested for WSI. The other four were used for the chloride conductivity test.
3. Concrete Cover Testing:
   One test area for cover testing consists of a 1-m^2 area on the bridge element's surface randomly selected as a representative area for the entire element. Each test area represents a maximum of 30-m^2.

3.2.1 Compressive Strength

Compressive strength testing was carried out throughout the project as part of quality control and acceptance of the structures. Sampling and testing for concrete compressive strength were carried out by the onsite laboratory. Since the elements were based on a prescribed mix with a minimum cement content and maximum w:c ratio, the achieved compressive strength was generally above the characteristic strength required in the structural design (see mix design concrete strength >60 MPa).

The number of cube specimens taken per element is detailed in Table 3-2. The total number of samples (consisting of three 150 mm cube specimens each) exceeded the minimum sample size for acceptable statistical analysis detailed in the standard specifications (COLTO, 1998). Concrete test cubes were crushed and analysed as per the standard specifications (TMH1, 1986). Test results were used for acceptance control as well as a check on concrete consistency.

Table 3-2 - Sampling for cube strength tests

Bridge	Element	No. samples (3×150mm)	No. test determinations
MR52 Interchange	Footings	49	147
	Abutments	16	48
	Piers	9	27
	Deck	15	45
TR2-2 Interchange	Footings	25	75
	Abutments	16	48
	Pier	17	51
	Deck	15	45
Swakop River Bridges	Piles	325	975
	Pile Caps	58	174
	Abutments	51	153
	Piers	25	75
	Deck	73	219

3.2.2 Durability Index Testing

Durability Index (DI) tests were carried out as per the South African Durability Index Testing Manual (DI Testing Manual, 2018). The tests give an indication of the concrete's resistance to penetration of potential corrosive liquids, gases and ions. These tests can, therefore, be used to assess the potential durability of the as-built concrete. Depending on the expected environmental exposure of the concrete, tests results can be used to determine the concrete's potential lifespan as the time to corrosion activation of the reinforcing steel.

The tests, detailed in Section 2.4.2, were the Oxygen Permeability Index (OPI), Water Sorptivity, and Chloride Conductivity to determine the concrete's resistance to permeation, sorptivity and diffusion respectively. Core samples were taken from 150 mm test cubes and 600×400x×150 mm test panels cast on-site adjacent to the bridge element.

3.2.2.1 DI cores from Test Cubes

Eight specimens were cored from concrete test cubes (150 mm) cured in water with a temperature of 22-25˚C in the laboratory for 28 days. The optimum curing conditions, compaction, and laboratory temperature were expected to produce concrete which corresponds to the maximum potential durability of the concrete mix.

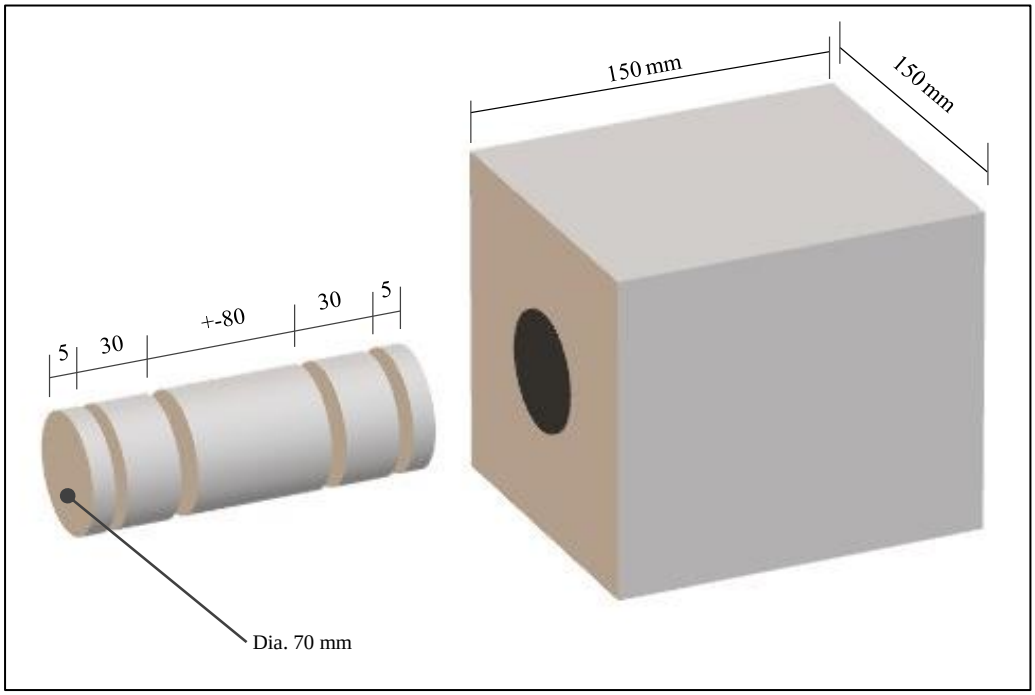


Figure 3-21 - Details of cores from 150 mm concrete cube

3.2.2.2 DI Cores from Test Panels

Durability Index testing was also carried out on representative concrete panels. The aim of the casting and sampling of test panels was to resemble the bridge elements in orientation and casting procedure. As per SANRAL's Project specifications, each test panel was constructed with the same concrete, shutter type, compaction and curing methods being used in the element being cast (including same vibrator frequency and curing method), and was left to cure for 28 days adjacent to the concrete element (Appendix 7.1). The dimensions of the test panels were 600×400×150 mm to allow for eight cores to be removed and cut into discs (4 discs for OPI and water sorptivity and 4 discs for chloride conductivity). The panels were cast in-situ adjacent to where the concrete element was placed. Test panels were constructed with timber shutters and compacted with internal poker vibrators with a vibration frequency of 10,000 cycles/minute (as used on the main structure).

Test panels were prepared as follows:

1. Panels representing the bridge decks, cast horizontally and covered with plastic for ten days. For the deck elements, the cover concrete tested for durability was the top, unformed surface representing the top of the bridge deck.



Figure 3-22 - Durability test panels representing bridge deck

2. Panels representing the bridge piers and abutments - cast vertically, stripped after two days, and wrapped in plastic for a further eight days.



Figure 3-23 -Durability test panels representing bridge piers and abutments

After the ten-day curing period detailed above, the panels were left next to the bridge element for a further 18 days (minimum of 28 days from cast date) without any additional curing before coring. Eight 70 mm diameter cores were extracted from the panels at random positions (Figure 3-24). The cores were cut into 30 mm discs as per the DI manual (DI Testing Manual, 2018). The discs were then wrapped in airtight plastic sheets and couriered by truck to Johannesburg for laboratory DI testing at the University of the Witwatersrand.

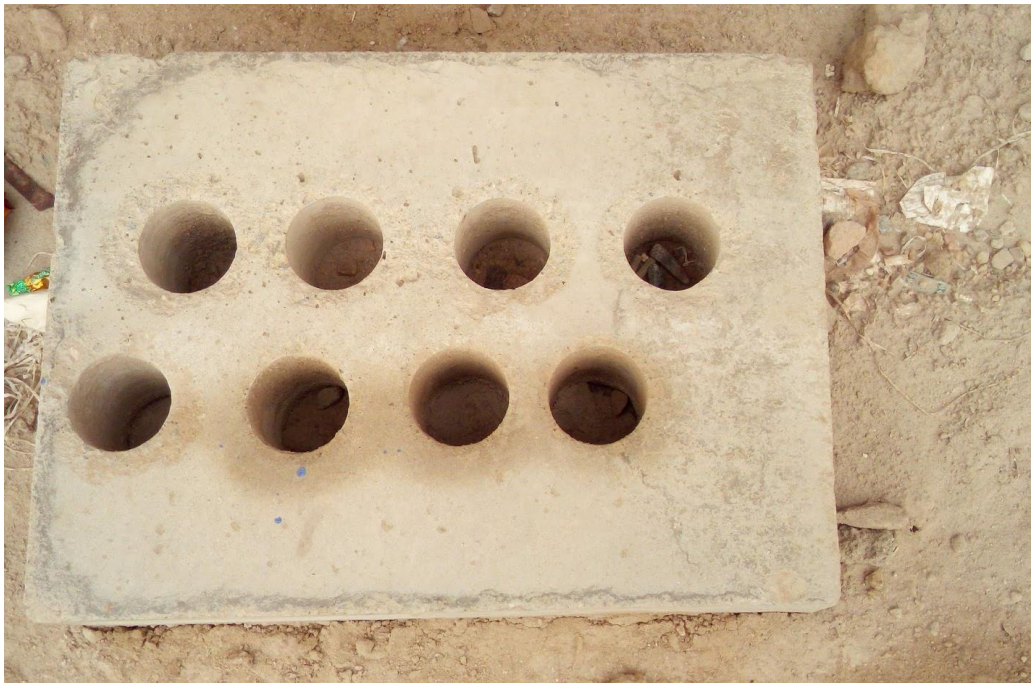


Figure 3-24 - Cores removed from sample panels

The number of test panels, cores and discs are summarised in Table 3-3 below.

Table 3-3- Number of samples and test specimens prepared for DI Testing

Sample type	Total number of samples and specimens prepared			
	No. test cube or test panel samples	Core specimens extracted and cut to discs	Test determinations	Test Results (ave. 4 test determinations)
Test cubes (150 mm)	4	8	OPI = 4 WSI = 4 CCI = 4	OPI = 1 WSI = 1 CCI = 1
Vertical test panels (600×400×150)	10	80	OPI = 40 WSI = 40 CCI = 40	OPI = 10 WSI = 10 CCI = 10
Horizontal test panels (600×400×150)	10	80	OPI = 40 WSI = 40 CCI = 40	OPI = 10 WSI = 10 CCI = 10
Total number	**21**	**168**	**OPI = 84** **WSI = 84** **CCI = 84**	**OPI = 21** **WSI = 21** **CCI = 21**

3.2.2.3 DI Testing Procedure

All DI tests were carried out in accordance with Durability Index Testing Manual (2018).

a) Oxygen Permeability Index (OPI) Test

The OPI Test procedure was performed in order to assess the air permeability of the concrete samples. Four OPI specimens were tested for each of the sample sets shown in Table 3-3. The samples were placed in the test apparatus (Figure 3-25) and subjected to oxygen pressurised

in the pressure vessel to 100kPa. The pressure decay was monitored and measured in order to determine the coefficient of permeability.

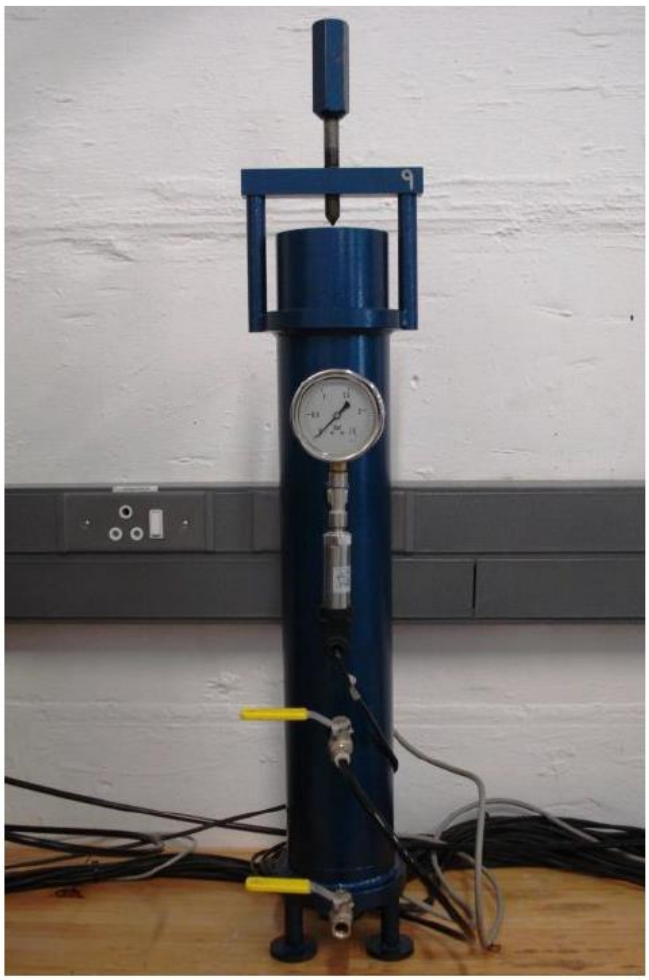


Figure 3-25 - OPI test equipment

b) Water Sorptivity Index (WSI) Test

The WSI Test procedure is performed by measuring the mass of water absorbed with time from the bottom of the concrete sample, of which the sides have been sealed with tape (Figure 3-26). Four WSI specimens were tested for each of the sample sets in Table 3-3. The concrete samples were exposed to a few millimetres of water with the test surface facing downward. At regular time intervals, the specimens were removed from the water, and the mass of water absorbed was determined. The concrete was then vacuum-saturated in water to determine the porosity. A linear relationship is observed when the mass of water is plotted against the square root of time, and the sorptivity of the concrete was determined from the slope of the straight line produced.



Figure 3-26 - Water sorptivity test set up

c) Chloride Conductivity Index (CCI) Test

The CCI Test procedure was performed in order to assess the chloride conductivity properties of the concrete samples. Four CCI specimens were tested for each of the sample sets shown in Table 3-3. The core samples were oven-dried at 50° C, followed by 24 hours of vacuum saturation in a 5M NaCl chloride solution. Thereafter, samples were placed in the testing rig, filled with the 5M NaCl solution and an electric potential difference of 10 V applied. The electric current is measured as a result of the movement of chloride ions through the concrete specimen. The current is inversely proportional to the concrete's resistivity.

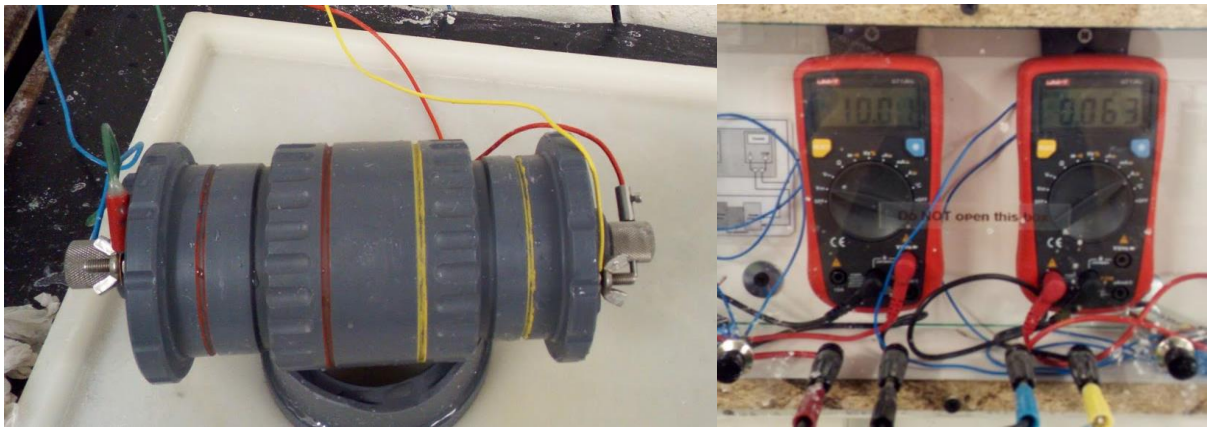


Figure 3-27 - CCI test set-up

3.2.3 Cover Testing

Cover to concrete is an essential element in the durability of steel-reinforced concrete elements. Reinforced concrete in the marine environment is particularly susceptible to chloride-induced corrosion of the reinforcing steel. The concrete cover acts as a protective

layer against the ingress of harmful agents. A reduction of concrete cover has a direct effect on the potential lifespan of the structure.

Obtaining adequate concrete cover is critical in the durability performance of the concrete. An accurate determination of as-built concrete cover was also crucial in the service life prediction of the bridge elements. All bridge decks, piers and abutments on the project were specified with a 50 mm cover to concrete. Concrete cover was checked before casting concrete for each element.

After casting and curing, bridge piers and abutments above ground and exposed to atmospheric chlorides were assessed for cover as per the cover testing procedure below, using an electromagnetic cover meter with an accuracy of 5.0-7.5 mm for cover readings less than 100 mm (Figure 3-28) (BS1881-204, 1988). Although the specified accuracy of the cover gauge is up 7.5 mm, the gauge was calibrated using a calibration test block before each use to ensure the accuracy of the reading, and there was no sign of any inaccuracies with the gauge.



Figure 3-28 - Electro-magnetic cover meter (Elcometer)

3.2.3.1 Cover Testing Procedure

a) Preparation

- At least one scan area was selected for each concrete face for each element. Figure 3-29 shows the typical locations for representative areas tested for cover.
- The cover meter was checked against a calibration box supplied with the equipment.
- The depth of cover was be determined with an electromagnetic cover meter and the results recorded electronically by the equipment software.

- Measurements were taken in accordance with the covermeter manufacturer's guidelines.



Figure 3-29 - Typical location of representative areas for cover testing on bridge substructure

The initial assessment and measurement of concrete cover was carried out according to the linear scan method detailed below.

b) Linear Scan Method

- Readings were taken perpendicular to the layer of rebar closest to the concrete surface for each scan area (+/- 30 per m^2).
- An average cover to reinforcement was determined for the test area.
- Readings were taken to identify individual bars within each 1.0 m^2. At least three cover readings, at 150mm spacing, per individual bar were recorded.

c) Detailed scan and corrective action

Where more than 10% of readings were below the specified lower limit, the area was re-scanned to identify every bar on the element and confirm the cover readings. Reinforcing bars with insufficient cover were marked on the surface and coatings were applied to these areas to provide "equivalent" concrete cover. Potential durability testing of these coatings is beyond the scope of this study.

3.2.4 Approach to service life prediction

Considering the marine environment and the structures' exposure to chlorides in the atmosphere, a South African Chloride Prediction Model (Mackechnie, 2001), was used to

estimate the service life of the structure based on the chloride conductivity and cover depth. The OPI and water sorptivity indexes and compressive strength gave further insight into the quality of the concrete.

The steps in the determination of potential durability and concrete lifespan using the chloride prediction model are summarised in Figure 3-30.

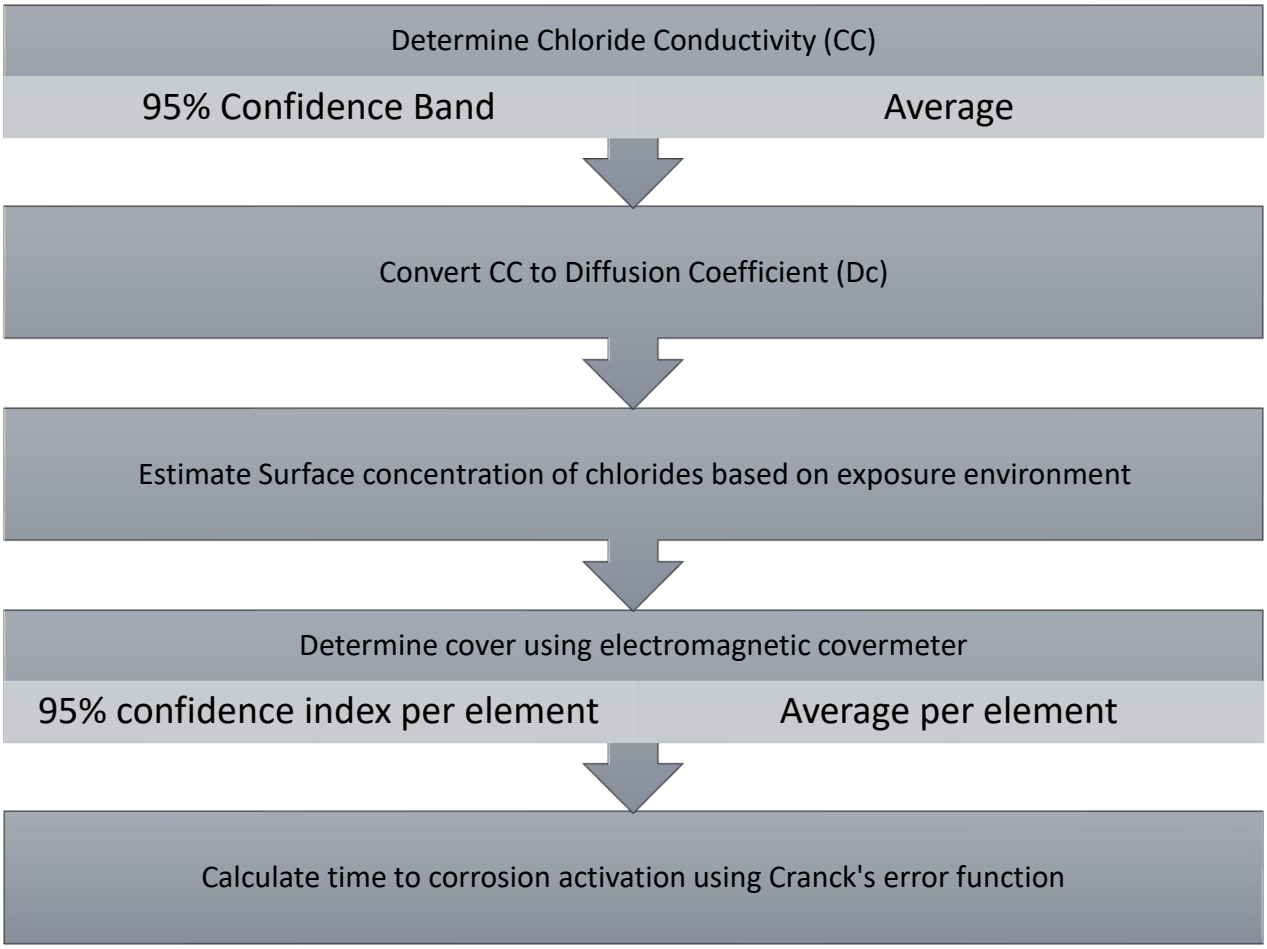


Figure 3-30 - Calculation of service life of structures

a) Determination of chloride conductivity index (DI tests)

The chloride conductivity index was calculated as per the Durability Index Testing Manual (2018). Based on the variability in test results, it would be inaccurate to determine a single value for chloride conductivity. A 95% confidence band was determined based on the spread of results. This band was calculated for vertical and horizontal elements. In addition to the confidence band, calculations of an absolute minimum service life were predicted using the maximum chloride conductivity determined.

b) Conversion of CCI to diffusion coefficient

A 50-year diffusion coefficient was determined from the chloride conductivity index using the nomogram developed by Mackechnie. This nomogram was developed empirically from results of concrete specimens on the West Coast of South Africa (Mackechnie, 2001).

c) Estimation surface concentration of chlorides and chloride levels at the reinforcement depth (Mackechnie, 2001)

Surface chloride concentrations were adopted from those determined empirically by Mackechnie (Mackechnie, 2001). Although the structures are not exposed to direct wave action and sea-spray, the coast of Namibia experiences very low rainfall, and there is no benefit of chlorides being washed off by rain as predicted in the model. For this reason, a surface concentration of 3.0% (i.e. equivalent to marine structures in the spray zone) was adopted (Table 3-4).

Table 3-4 - Chloride Surface Concentrations (% by mass of binder) (Mackechnie, 2001)

Concrete type	tidal / splash zone	spray zone
100% PC	3.0 – 4.0	1.5 – 2.0
10% SF	2.5 – 3.0	1.3 – 1.5
30% FA	4.5 – 5.0	2.3 – 2.5
50% SL	5.0 – 6.0	2.5 – 3.0

d) Determine cover reading for each element

Cover readings were determined for each element as described in 3.2.3. As with the CCI, it would be unrealistic to use a single value for concrete cover for the entire project. An average cover reading was determined as well as a 95% confidence band of the concrete cover for each element.

e) Calculation of time to corrosion activation (Mackechnie, 2001)

Using Crank's solution to Fick's second law of diffusion: chloride content, time to corrosion activation and, subsequently, the expected lifespan of the concrete structures was determined for each element using Equation 2.4 (reproduced here):

$$C_{x,t} = C_s\left[1 - \text{erf}\left(\frac{x}{2\sqrt{D_a t}}\right)\right] \qquad (2.4)$$

The time to corrosion activation is quantified as the time taken for chloride levels at the reinforcement to reach chloride threshold levels. These values were calculated according to Equation 2.4 with all symbols being constants except for the cover reading "x" which varies for each element. Chloride threshold is discussed in detail in the literature review. It is only necessary to note here that a chloride threshold value of 0.4% (expressed as a percentage weight of binder) was selected as sufficiently conservative for this analysis.

It is important to note that all service life predictions are based on the concrete's resistance to ingress of aggressive agents and do not include the additional lifespan afforded by the coatings of the elements. As noted in the literature review, these coatings can be expected to add 15-25 years to the structures. The calculations on service life prediction yielded an output of expected time to corrosion activation for each structural element exposed to chlorides.

3.3 SUMMARY

Experimental analysis and service life modelling was carried out on concrete from bridges on the Swakopmund-Walvis Bay freeway upgrade. The environment consists of high chloride concentrations due to the excessive fog, high humidity, and low rainfall in the area.

Concrete quality and potential durability were assessed by a combination of concrete strength, durability index testing and electromagnetic cover readings. The potential service life of the structure was calculated using the Chloride Corrosion Model based on the CCI determined in the laboratory and the in-situ concrete cover readings for each element.

4 RESULTS AND DISCUSSION

4.1 INTRODUCTION

This section covers the analysis of the results of concrete strength tests as well as Durability Index (DI) tests from concrete samples and cover assessment of as built bridges. Following an assessment of the upper and lower bounds of DI and cover, an estimate was made of the expected service life of the critical major vertical elements in the bridge element using the South African Chloride Prediction Model.

4.2 CONCRETE STRENGTH

Compressive strength testing was carried out throughout the bridge construction as part of quality control and acceptance of the bridge elements. The measured compressive strength was generally above the characteristic strength required in the structural design (see mix design concrete strength >60MPa, Section 3.1.4.1). The validity of test results was determined according to TMH1 (1986), and only valid 28-day test results were used in the statistical analysis:

- Three cubes are crushed per set and the average compressive strength is calculated to the nearest 0.5 MPa.
- If the difference between the highest and lowest results exceed 15% of the average, the test is regarded as invalid and the results discarded.
- The average of the three cubes is recorded as a single test result

The critical bridge elements selected for statistical analysis are the footings, abutments, piers and decks. The piles were not considered in this study. All concretes in the following assessment were of the same concrete mix design with nominally the same materials.

a) Numerical Summary

Table 4-2 and Figure 4-1 summarise the compressive strength results for all bridges in the project. The observations made from the results are the following:

i) The average strengths across all bridge elements vary between 44.8 MPa and 62.4 MPa and were generally substantially more than the design 28-day target characteristic strength of 40 MPa. These high strengths can be attributed to the concrete mix design dictated by the durability specifications (Section 3.1.3.3) which

required a maximum water-binder ratio and a minimum cement content, regardless of the target strength of the concrete. A low water-binder ratio along with high cement content will produce a higher strength concrete (Addis & Goodman, 2009). This was evident with the designed concrete mix producing a concrete strength exceeding 60 MPa.

ii) The standard deviation of compressive strength for all bridge elements varies between 2.8 and 5.7 MPa. McDonald (2009) gives a comparison of standard deviations for compressive strength and the relative quality control at the batch plant. These limits are reproduced in Table 4-1. Based on these limits, the quality control at the batch plant for this project can be considered satisfactory.

Table 4-1 - Examples of degrees of control at batching plant based on standard deviation (McDonald, 2009)

Control	Standard Deviation (MPa)
Poor	7
Average	6
Good	5

iii) If all valid test results from all concrete samples are considered in the analysis, the standard deviation is increased to **7.1 MPa** for a sample size of 353 valid test results. This suggests that although the quality control on any given day might be "good", there is significant variability in the test results throughout the entire project which is likely a result of variability in the concrete-making material properties with time. Most important would be the water demand of the cement and the aggregates which varied considerably with time (section 3.1.5). Logically, with a fixed cement content, a cement and aggregate with a higher water demand will significantly increase the w:b ratio in the concrete mix, reducing the concrete strength. Another significant factor is the strength of the cement which was shown to drop by as much as 8% from the cement used in the initial concrete mix design.

iv) It should be noted that a superior concrete strength does not necessarily equate to an increase in potential durability. The lack of correlation between compressive strength and durability is discussed in Section 1.3.

Table 4-2 - Summary of 28-day compressive strength results for all bridges

Bridge	Bridge Element	No. Test results (3×150mm cubes)	Average Strength (MPa)	Min (MPa)	Max (MPa)	Standard Deviation (MPa)	COV (%)
MR52 Interchange	Footings	49	55.2	44.1	67.0	5.7	10.3
	Abutments	16	57.9	47.3	67.1	6.2	10.7
	Piers	9	53.3	47.0	61.1	5.2	9.8
	Deck	15	51.4	38.7	61.8	5.4	10.6
TR2/2 Interchange	Footings	25	53.5	46.0	60.7	4.3	8.1
	Abutments	16	53.7	41.7	62.9	6.3	11.8
	Pier	17	65.2	60.7	71.3	3.1	4.8
	Deck	15	56.1	50.5	64.1	3.9	6.9
Swakop River A	Pile Caps	36	61.5	51.7	69.8	4.1	6.6
	Abutments	22	60.1	50.8	69.3	4.5	7.5
	Piers	12	49.3	44.7	54.5	2.8	5.7
	Deck	37	58.9	40.1	66.9	5.8	9.8
Swakop River B	Pile Caps	22	62.4	56.4	69.0	3.3	5.3
	Abutments	29	44.8	37.9	51.2	3.4	7.7
	Piers	13	49.4	43.8	56.7	3.8	7.7
	Deck	36	61.2	53.8	69.8	4.3	7.1

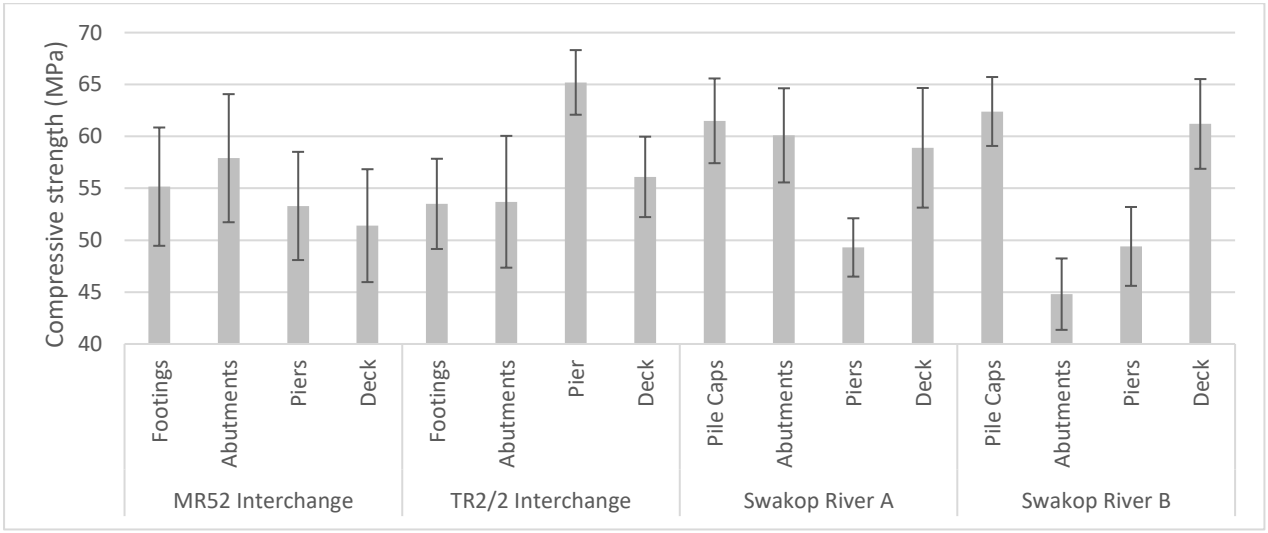


Figure 4-1 – Graphical summary of 28-day concrete strength results

b) Frequency distribution of compressive strength results

Figure 4-2 shows a frequency distribution histogram of 28-day compressive strength results for all the bridges. The results show a large spread confirming the high standard deviation discussed previously. The peak compressive strength varies from bridge to bridge. This is likely a result of the casting dates of the bridges, and the variations in weather conditions and material properties.

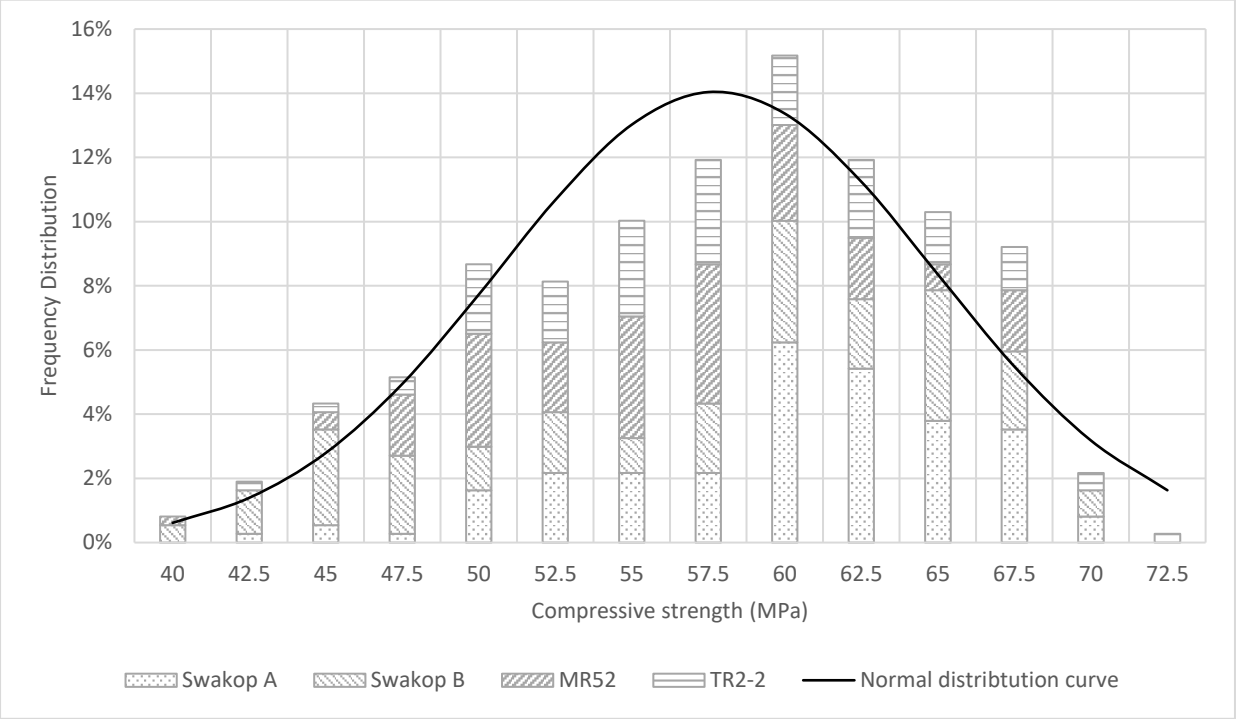


Figure 4-2 – Frequency distribution histogram of compressive strength for all bridges

4.3 DURABILITY INDEX RESULTS

4.3.1 General

In this section, the Durability Index testing results are summarised. Target values for DI test results were determined according to the 'deemed-to-satisfy' approach used in the SANRAL Specifications (Appendix 7.2) as a gauge of the quality of the concrete. Target values are dependent on environmental exposure conditions, cover to concrete and binder type. For assessment purposes the carbonating environment is assumed to be XC3 for "exterior concrete surfaces in moist areas" and for chloride-induced corrosion as XS1 for "structures exposed to airborne salt but not in direct contact with seawater or inland saline waters". Based on these exposure conditions, with a 50 mm concrete cover depth and a 50:50 CEMI: GGBS binder blend, the durability specification targets were extracted from Table 6000/1

(Appendix 7.2). These target values indicate a characteristic value for as-built concrete quality. DI test results are assessed according to the parameters of Table 4-3. Using these target values, an assessment is made on the quality of the as-built concrete. It is noted that the use of DI test panels in lieu of coring directly from the concrete structure have been investigated extensively and it has been shown that the DI results for all three tests (OPI, CCI, and WSI) show a good correlation between the in-situ concrete and the representative test panels (Ronny, 2011). Considering that the in-situ concrete is not directly tested, potential durability and service life predictions are limited by the suitability of repair methods and remedial work of defect concrete as discussed in Section 3.1.5.

Table 4-3 - Target DI acceptance values based on SANRAL specifications (Appendix 7.2)

DI Parameter	Target acceptance value	Unit
OPI	9.10 min	-log (k): k in m/s
CCI	2.20 max	mS/cm
WSI	10.0 max	$mm/\sqrt{hr}.$

The coefficient of variance within a single set of test determinations was calculated and is discussed in the subsequent sections. It is, however, sensible to note the expected variation in test determinations as an assessment of whether a test result (consisting of 3 to 4 test determinations) is valid. Gouws, et al (2001) studied the coefficient of variance of four test determinations for OPI and WSI and three test determinations for CCI from various concrete sources. The applicable coefficients of variance to this study are those from the actual structures and those from the wet-cured, ready-mix concrete as summarised in Table 4-4.

Table 4-4 - Coefficients of variation for DI tests (Gouws, et al., 2001)

Concrete Source	OPI	WSI	CCI
Actual Structures	3%	14%	13%
Wet-cured (material potential)	1%	7%	5%

The suitability of a test result (comprising of four test determinations) is assessed according to the expected coefficients of variation (COV) in Table 4-4. If the actual COV for a test result significantly exceeds the COV in Table 4-4, there is a possibility that the result is not valid and should be discarded. Regardless of the specified limits, the acceptance of a result relies on considerable “engineering judgement”.

The practicality of testing samples at 28 to 35 days proved difficult due to logistics in coring specimens, cutting discs, transporting the discs to university, and testing the discs, all within this timeframe after the concrete cast date. However, cores were drilled between 28 and 35 days as required and stored in sealed containers until such time as they could be cut to size. After drilling, the cores were stored for between 7 and 87 days until such time as the independent laboratory was available to cut the cores into discs. Thereafter, the discs were transported to the testing laboratory at the University (2-3 days travel time) after which they were placed into the oven within three days. The specimens were always kept sealed to ensure limited exposure to environmental effects. The effect of "age at testing" is summarised for each DI test in the subsequent sections. The age at testing is not used as a comparison of results for vertically and horizontally cast elements but to determine the general trend of DI test results with age.

4.3.2 Oxygen Permeability Index (OPI)

OPI tests were carried out on 18 samples consisting of four specimens each. Due to the transport and packaging issues, some specimens were damaged and were discarded. This meant that for some test results, only three specimens were tested.

a) Numerical Summary

Table 4-5 and Figure *4-3* show a summary of the OPI tests, with a standard deviation and coefficient of variance. The observations made from the computed results are the following:

i) All test results exceed the 'deemed-to-satisfy' OPI of 9.10 from Table 4-3. This indicates a generally low penetrability concrete with excellent resistance to ingress of gases through permeation. The lowest OPI value obtained was 9.61 which significantly exceeds the limit of 9.10.

ii)

iii) The highest OPI value obtained was 10.70 which is approximately equal to the maximum potential of the concrete mix of 10.69. This high result shows that a well compacted and cured concrete produced on site can reach the ultimate potential durability of the concrete mix.

iv) The COVs vary between 0.8% and 6.8% with an average of 2.2%. The average is slightly lower than COV of 3% in Table 4-4. The COV of 2.3% for the test cube is, however, higher than the 1% from Table 4-4.

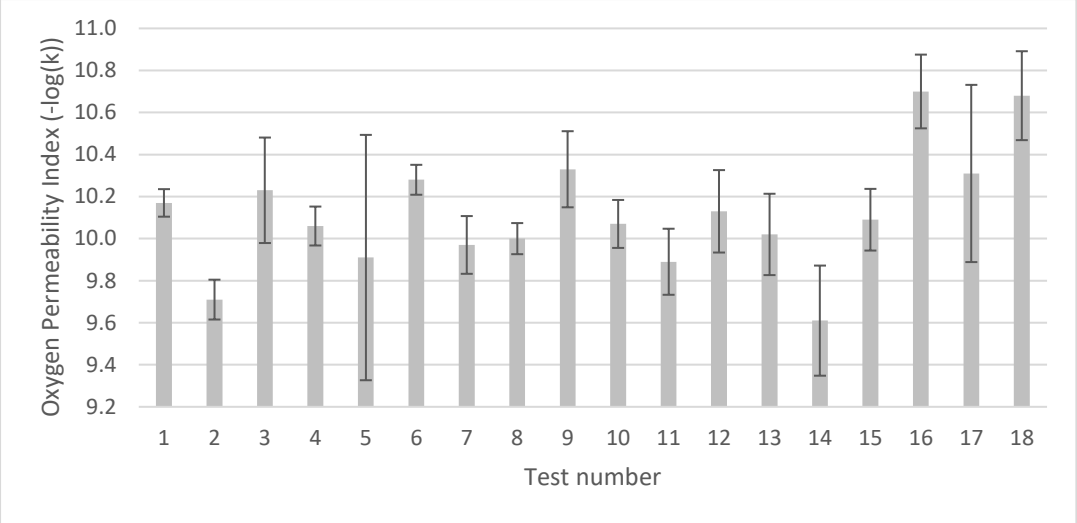


Figure 4-3 - *Graphical summary of OPI results*

Table 4-5 - Summary of OPI results

Element type	Test Number*	Age at Testing	OPI Test Result	Standard deviation	COV (%)
Horizontal	1	67	10.17	0.065	0.8
	2	88	9.71	0.095	1.1
	3	132	10.23	0.251	2.8
	4	132	10.06	0.093	1.1
	5	52	9.91	0.584	6.8
	6	66	10.28	0.071	0.8
	7	67	9.97	0.137	1.7
	8	60	10.00	0.074	0.9
Vertical	9	91	10.33	0.181	2.0
	10	60	10.07	0.114	1.3
	11	64	9.89	0.157	1.8
	12	63	10.13	0.196	2.2
	13	62	10.02	0.193	2.2
	14	61	9.61	0.262	3.1
	15	59	10.09	0.147	1.7
	16	63	10.70	0.175	1.9
	17	69	10.31	0.421	4.7
Test Cube	18	76	10.68	0.211	2.3

*Each test consists of the average of 3 to 4 discs

b) Frequency distribution

Figure 4-4 is a frequency distribution for the OPI results for the 17 specimens tested, excluding the specimen taken from the test cube. The frequency distribution shows a peak at 10.20. This peak is valid for horizontal and vertical elements and shows a reasonable consistency between the two types of elements. Slightly less than 70% of all OPI test results fall within the range of 10.0 – 10.4.

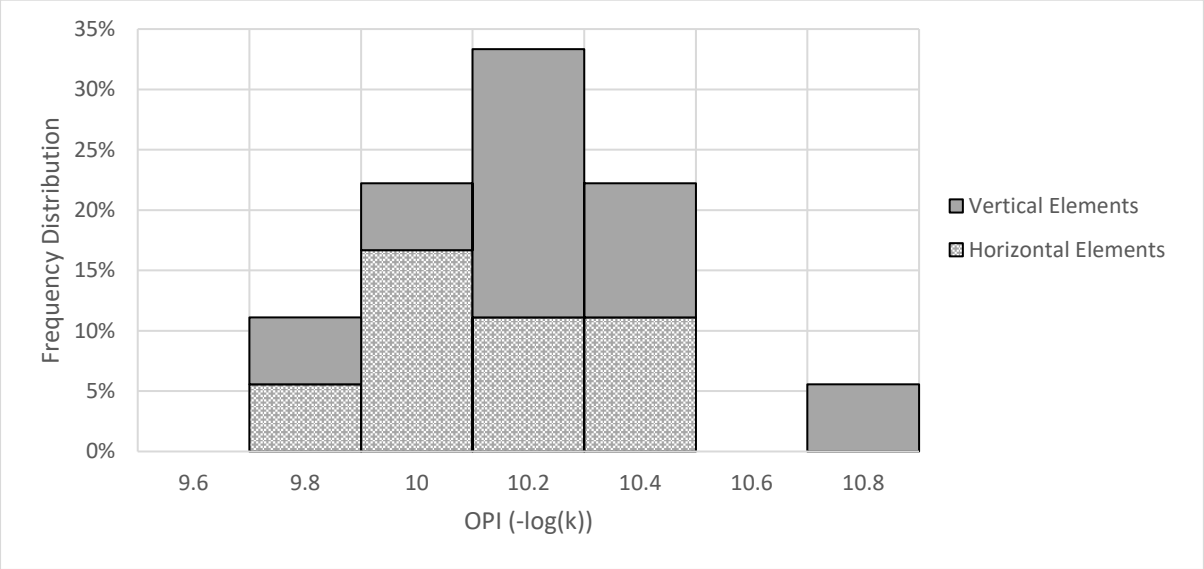


Figure 4-4 - Frequency distribution histogram for OPI test results

c) Boxplot comparison

Figure 4-5 is a boxplot comparing the variability in results for the vertical and horizontal elements. The average values for both element types are approximately equal although there is significantly less variance in results from horizontal elements than vertical elements. A possible reason for the narrow scatter of results for horizontal members may be due to the floating and finishing of the top surface which would result in a densification of the top layer. Several locations on the top surface of a horizontally cast element are less likely to have varying degrees of compaction compared to several locations at different depths of a vertically cast element.

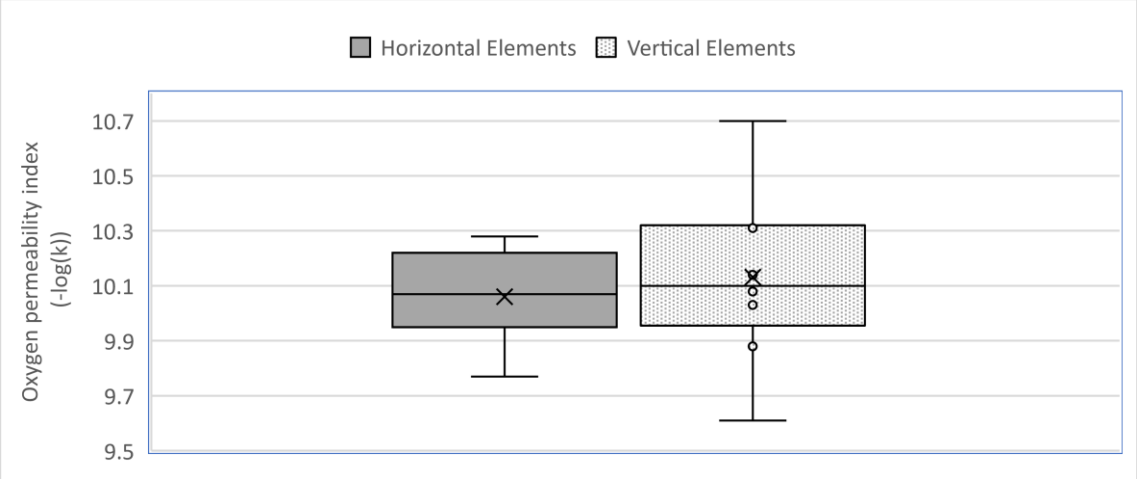


Figure 4-5 - Box plot comparison of OPI values for vertical and horizontal elements

d) Effect of testing age on OPI

Figure 4-6 shows the effect of the age at testing on the oxygen permeability index values in this study. There is no clear correlation between the two, and the OPI values obtained from the different tests are, therefore, compared without any adjustments.

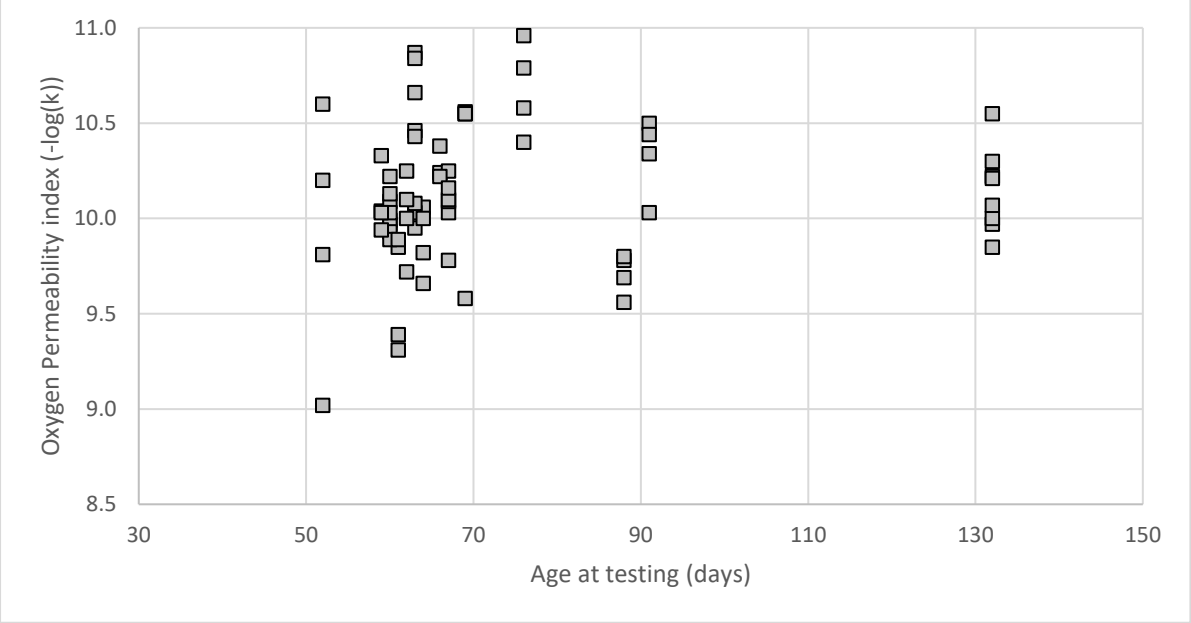


Figure 4-6 - Effect of age at testing on OPI

4.3.3 Water Sorptivity Index (WSI) and Porosity

WSI tests were carried out on the 18 samples used in the OPI test, again excluding the damaged specimens which were discarded. As part of the test procedure for WSI, the dry mass and saturated mass of the specimens were used to determine the porosity.

a) Numerical Summary

Table 4-6 numerical summary of the WSI and porosity results, with a standard deviation and coefficient of variance. The observations made from the computed results are the following:

i) The mean values for all tests are below the maximum 'deemed-to-satisfy' value of 10.0 $mm/\sqrt{hr}$ from Table 4-3. This indicates a generally low penetrability concrete with good resistance to ingress of liquids through sorptivity.

ii) The lowest value obtained from the test panel samples is 5.54 $mm/\sqrt{hr}$ which is slightly higher than the potential of the concrete mix of 4.91 $mm/\sqrt{hr}$.

iii) The coefficients of variance for the individual WSI tests vary between 3.6% and 29.2% with an average of 16.4%. The average of 16.4% is only slightly higher than the 14% from Table 4-4. The biggest cause for concern is the 20% COV for the test cube, which is substantially higher than the 7% in Table 4-4. In general, the results show a significant variation across test specimens and should be used with caution.

iv) The porosity results vary between 7.4% and 12.2%. The COV for each test is substantially lower than for the WSI test with an average COV of 9.5%.

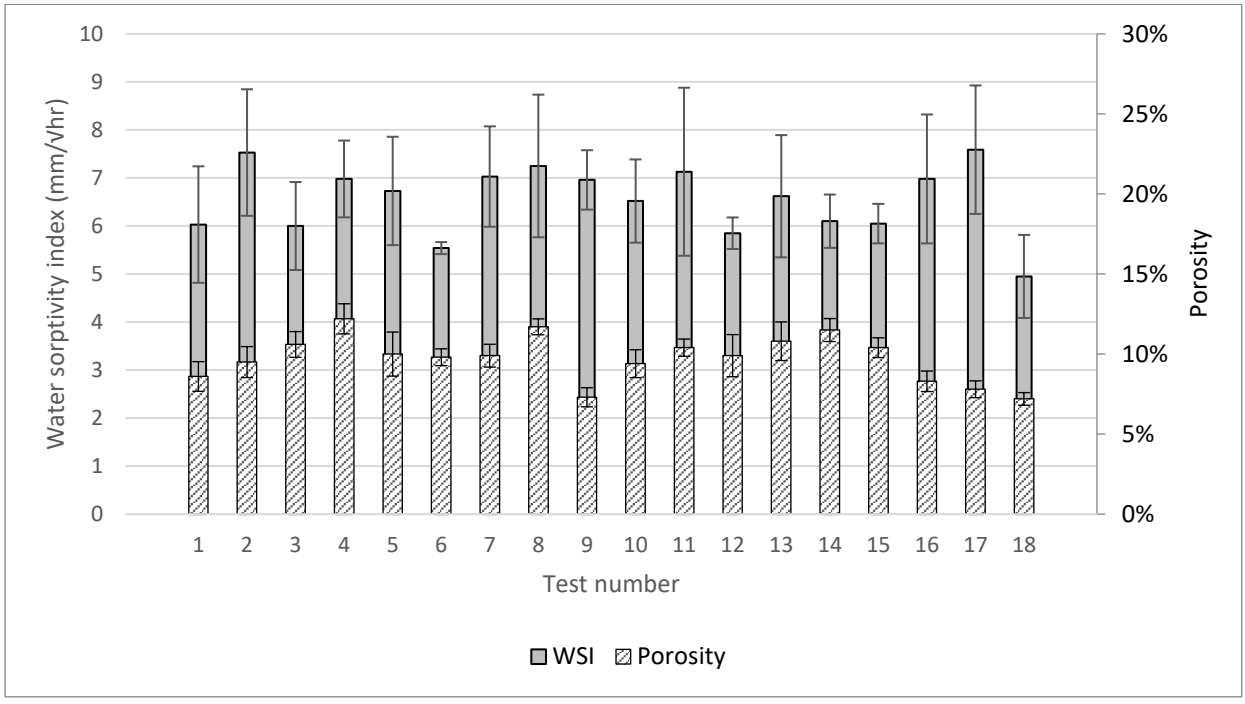


Figure 4-7 - Graphical summary of WSI and porosity result

Table 4-6 - Summary of WSI and porosity results

Element type	Test No.*	Age at testing	Water sorptivity index results			Porosity results		
			WSI (mm/$\sqrt{hr}$)	Standard deviation (mm/$\sqrt{hr}$)	COV (%)	Porosity (%)	Standard deviation (%)	COV (%)
Horizontal	1	67	6.03	1.213	24.6	8.6	0.9	13.2
	2	88	7.53	1.317	20.2	9.5	1.0	11.8
	3	132	6.00	0.916	17.6	10.6	0.8	8.7
	4	132	6.98	0.799	13.2	12.2	0.9	8.9
	5	52	6.73	1.128	19.4	10.0	1.4	16.0
	6	66	5.54	0.125	2.8	9.8	0.5	6.5
	7	67	7.03	1.045	18.2	9.9	0.7	8.8
	8	60	7.25	1.486	23.7	11.7	0.5	4.9
Vertical	9	91	6.96	0.618	10.2	7.3	0.6	9.5
	10	60	6.52	0.866	15.3	9.4	0.9	10.7
	11	64	7.13	1.750	28.3	10.4	0.5	6.0
	12	63	5.85	0.328	6.5	9.9	1.3	15.4
	13	62	6.62	1.273	22.2	10.8	1.2	12.9
	14	61	6.10	0.555	10.5	11.5	0.7	7.2
	15	59	6.05	0.411	7.8	10.4	0.6	6.8
	16	63	6.98	1.343	22.2	8.3	0.6	9.0
	17	69	7.59	1.338	20.4	7.8	0.5	7.7
Test Cube	18	76	4.95	0.865	20.2	7.2	0.4	6.3

*Each test consists of the average of 3 to 4 discs

b) Frequency Distribution

Figure 4-7 is a frequency distribution for the WSI results for the 17 specimens tested, excluding the specimen taken from the test cube. The frequency distribution shows a peak at 7.0 mm/$\sqrt{hr}$. This peak is valid for vertical elements while the horizontal elements show a wider scatter from 6.0 – 7.5 mm/$\sqrt{hr}$. The WSI test results are generally scattered. This is testament to there being a wide COV for individual tests and significant inconsistency across several test results.

Figure 4-8 is a frequency distribution for the porosity test results. This distribution shows a peak at 10% for horizontal elements and 11% for vertical elements.

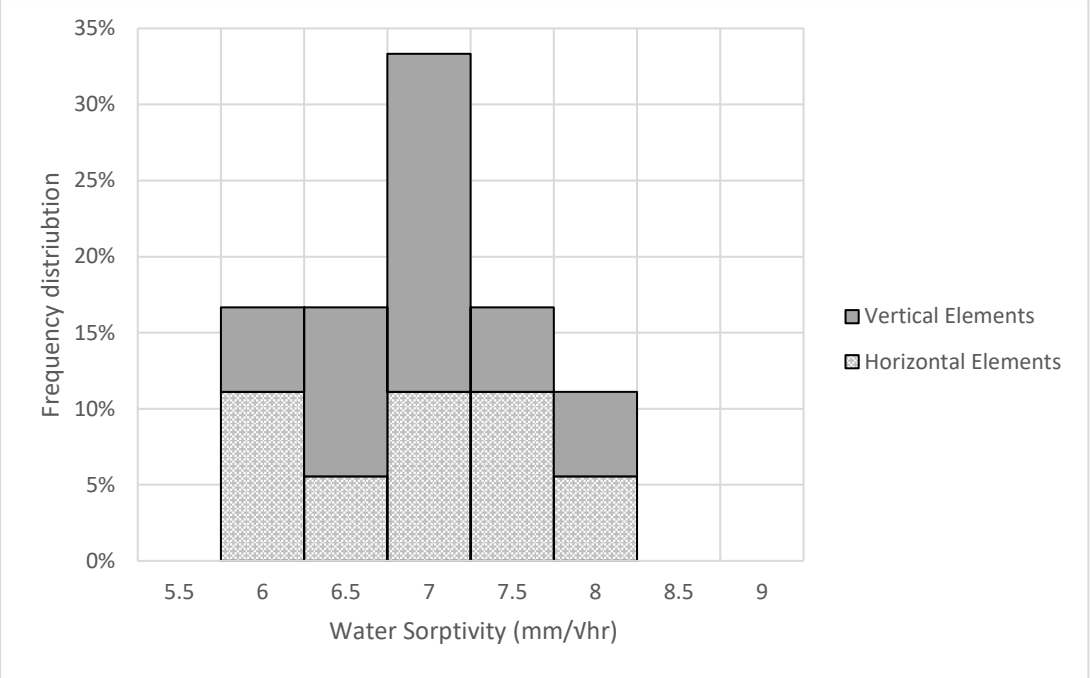


Figure 4-7 - Frequency distribution histogram for WSI results

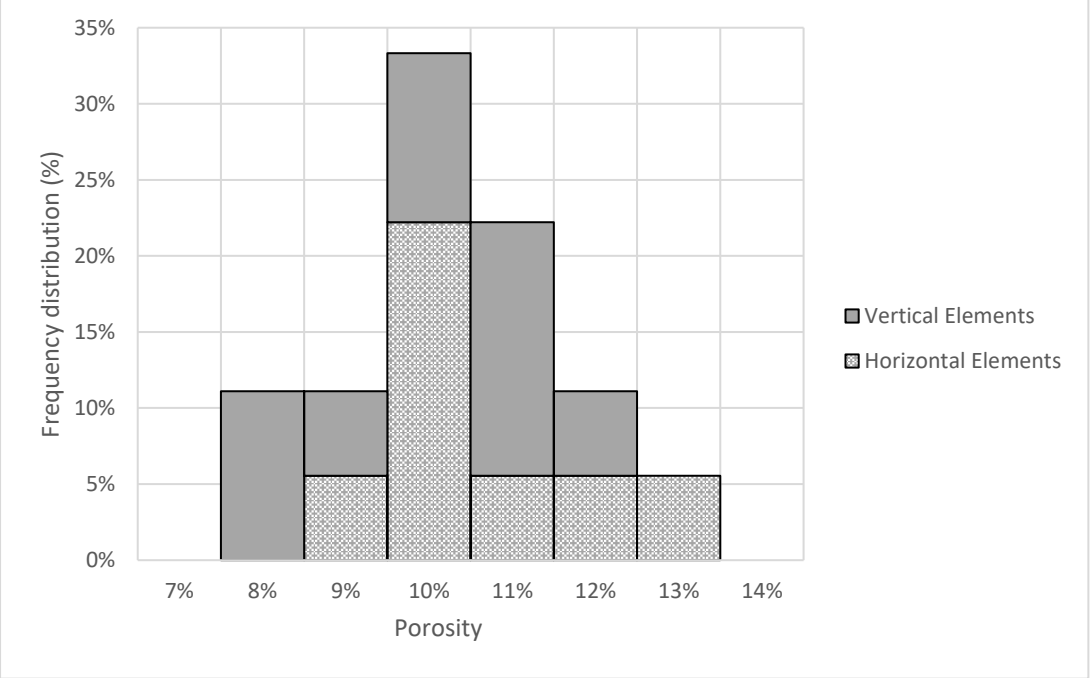


Figure 4-8 - Frequency distribution histogram for porosity results

c) Boxplot comparison

Figure 4-9 is a boxplot comparing the variability in WSI results for the vertical and horizontal elements. The average values and variance for both element types are approximately equal although there is a significant scatter of results.

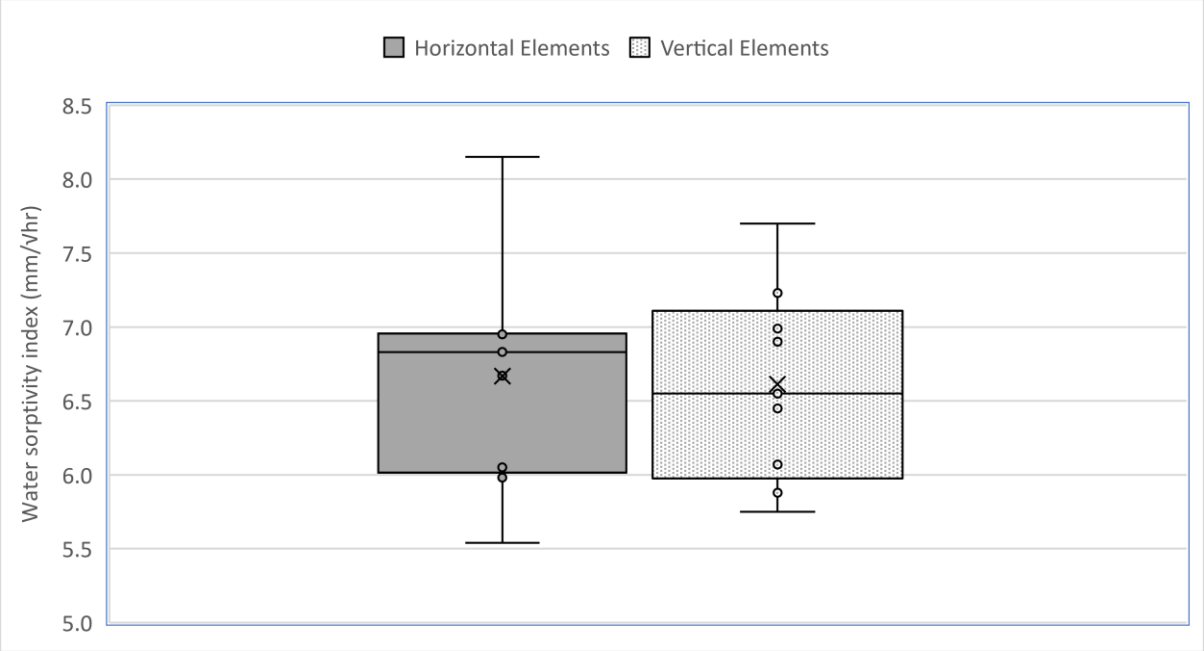


Figure 4-9 - Boxplot comparison of horizontal and vertical element WSI test results

d) Effect of age at testing on WSI

Figure 4-6 shows the effect of the age at testing on the Water Sorptivity Index values in this study. There is no clear correlation between the two, and WSI values obtained from the different tests are, therefore, compared without any adjustments.

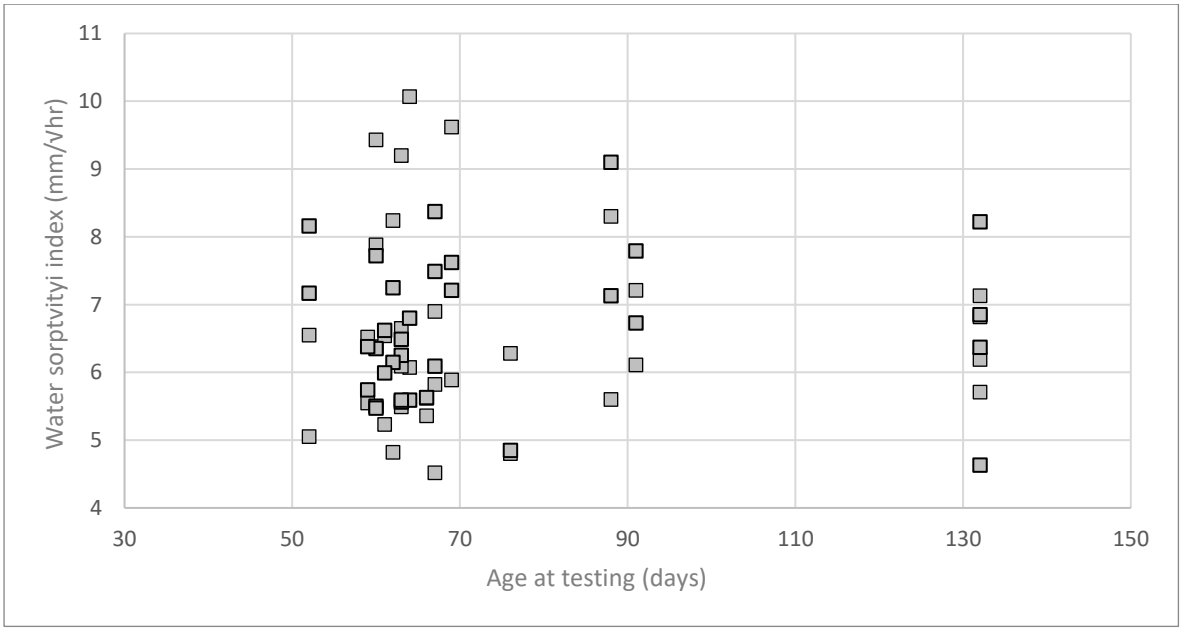


Figure 4-10 - Effect of age at testing on WSI

4.3.4 Chloride Conductivity Index (CCI)

a) Numerical Summary

Eighteen chloride conductivity tests were carried out on four test specimens each (or three where damaged specimens were discarded) as summarised in Table 4-7. Observations of the computed results are as follows:

i) All test results are significantly below the SANRAL 'deemed-to-satisfy' value of 2.20 mS/cm from Table 4-3. In general, the test results indicated a high-quality concrete with a very high resistance to chloride ingress.

ii) The lowest value obtained from the test panel samples is 0.48 which is higher than the potential of the concrete mix of 0.35 mS/cm. These lower CCI values would be considered suitable for even harsher environments according to the limitations in Appendix 7.2.

iii) The coefficients of variance for the individual CCI tests vary between 3.0% and 23.6% with an average of 8.8%. The average of 8.8% is significantly lower than the 13% from Table 4-4. Test result number 2 with a 23.6% COV is possibly an outlier considering it is the only COV greater than 15%. The COV for the test cube is 8.5% which is significantly higher than the 5% in Table 4-4. The CCI for the material potential could be more accurately determined with a greater sample size.

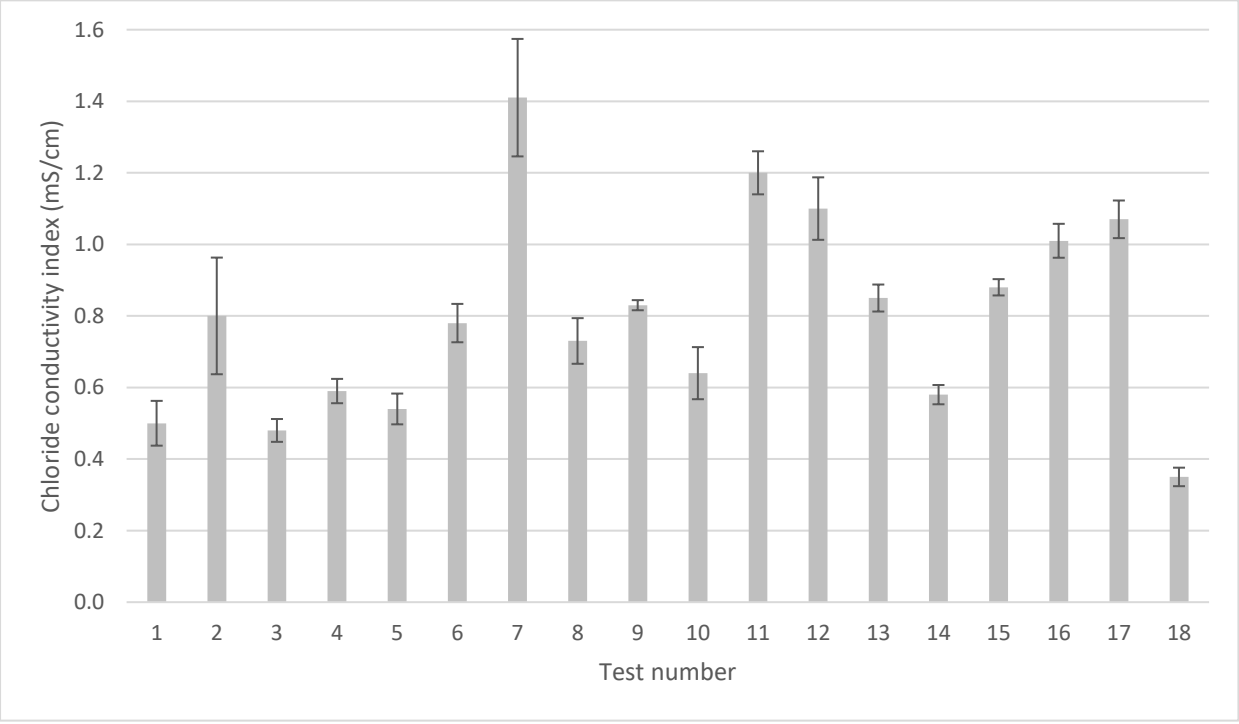


Figure 4-11 - Graphical summary of CCI results

Table 4-7 - CCI test results

Element type	Test Number*	Age at Testing (days)	Chloride Conductivity (mS/cm)	Standard deviation	COV (%)
Horizontal	1	90	0.50	0.063	14.4
	2	90	0.80	0.163	23.6
	3	89	0.48	0.032	7.6
	4	90	0.59	0.034	7.0
	5	132	0.54	0.043	9.2
	6	69	0.78	0.054	8.0
	7	66	1.41	0.164	14.3
	8	59	0.73	0.064	10.1
Vertical	9	58	0.83	0.014	2.0
	10	91	0.64	0.073	13.1
	11	50	1.20	0.060	6.1
	12	60	1.10	0.087	9.2
	13	59	0.85	0.038	5.5
	14	61	0.58	0.027	5.4
	15	64	0.88	0.023	3.0
	16	62	1.01	0.047	5.4
	17	58	1.07	0.053	5.7
Test Cube	18	132	0.35	0.026	8.5

*Each test consists of the average of 3 to 4 discs

b) Frequency distribution

Figure 4-12 is a frequency distribution for the CCI results for the 17 specimens tested, excluding the specimen taken from the test cube. The frequency distribution shows a peak at 0.75-1.0 mS/cm for horizontal elements and 1.25 mS/cm for vertical elements. There is minimal scatter across the results with slightly less than 60% of the results falling in the range of 0.75-1.0 mS/cm.

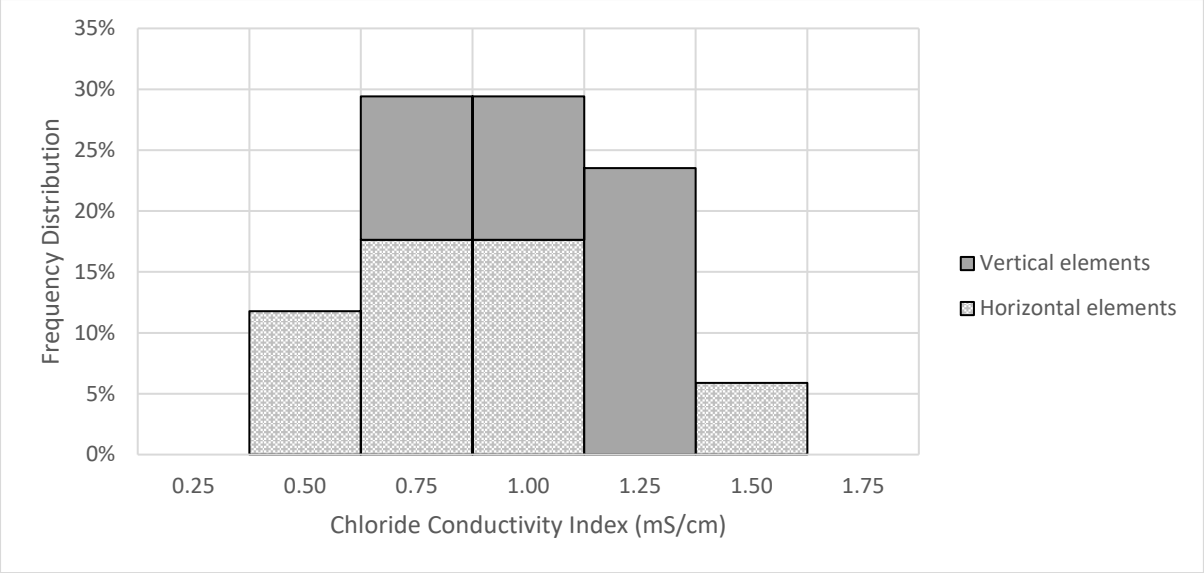


Figure 4-12 - Frequency distribution histogram for CCI results

c) Boxplot comparison

Figure 4-13 is a boxplot comparing the variability in CCI results for the vertical and horizontal elements. The average CCI for vertical elements is higher than for horizontal elements. As with the OPI test results, the CCI for horizontal elements is substantially better than for vertical elements (when the 1.41 mS/cm outlier is discarded). Possible reasons for the higher quality concrete are floating and finishing of the top surface, and bleeding along the face of the formwork for vertical elements as discussed in 4.3.2. The migration of bleed water could also result in excessive blowholes on the vertical faces of these elements, reducing the resistance to diffusion.

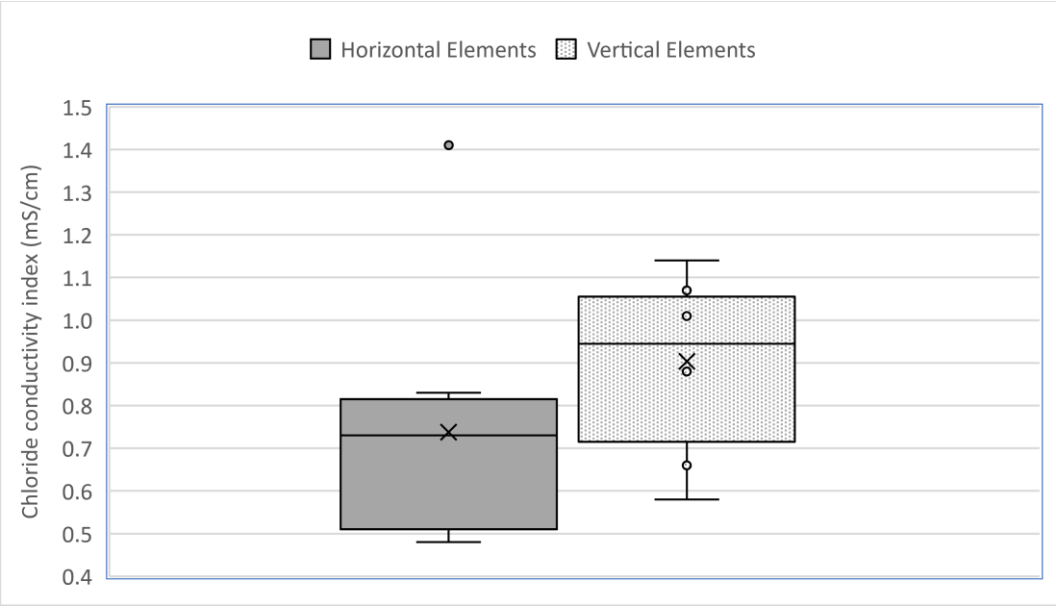


Figure 4-13 - Boxplot comparison of horizontal and vertical element CCI test results

d) Effect of age at testing on CCI

Figure 4-14 shows the effect of the age at testing on the Chloride Conductivity Index values in this study. Unlike with the OPI and WSI tests, there is a noticeable correlation between the measured CCI and the age at testing of the specimens. This is based on a very small sample size, however, and further tests would be required for a more accurate assessment of the effect of age at testing on CCI. With this in mind, the lower values of < 0.6 should be used with caution.

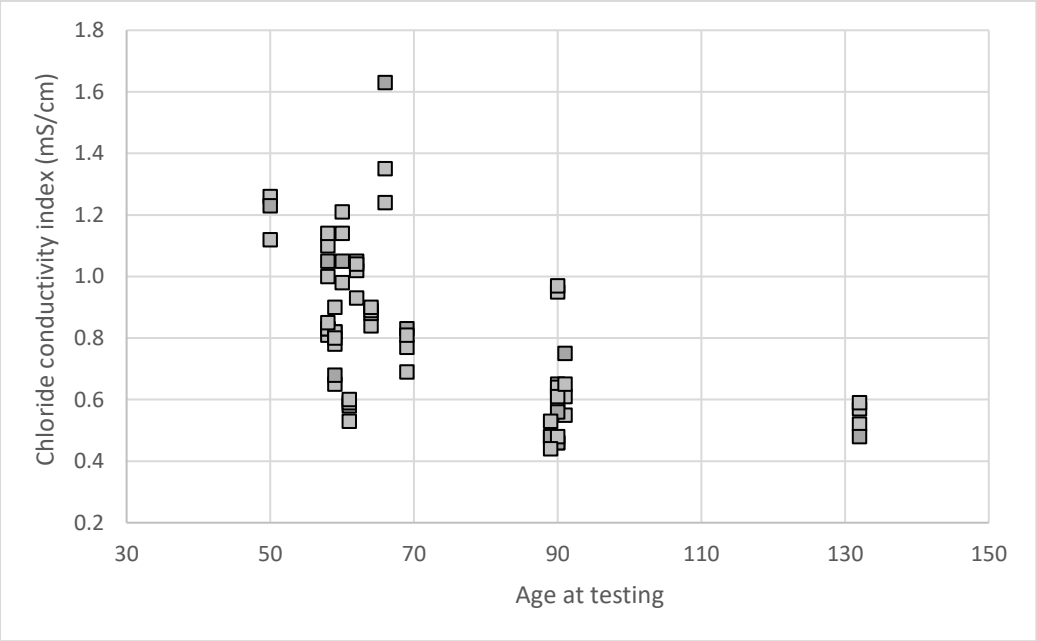


Figure 4-14 - Effect of age at testing in CCI

4.3.5 Summary

Table 4-8 summarises the DI test results. The material potential DI values are shown in the last row. In general, the results indicate a high quality concrete with high potential durability when compared with the 'deemed-to-satisfy' targets in Table 4-3. According to the approach, the target values for OPI is greater than 9.10 and for chloride conductivity less than 2.20. In addition to this, the suggested maximum sorptivity is less than 10 $\mathrm{mm}/\sqrt{\mathrm{hr}}$. The range of values for all DI parameters surpasses the target values.

Table 4-8 - Summary of DI test results

Element	No. Tests*	OPI range (- log(k))	WSI range ($mm/\sqrt{hr}$)	CCI range** (mS/cm)	Porosity range (%)
Total	17	9.61 – 10.70	5.54 – 7.70	0.48 – 1.41	7.40 – 12.2
Horizontal Elements	8	9.72 – 10.28	5.54 – 7.36	0.48 – 1.41	8.60 – 12.2
Vertical Elements	9	9.61 – 10.70	5.75 – 7.70	0.58 – 1.20	7.40 – 11.6
Potential (test cube)	1	10.69	4.91	0.35	7.2

*Each test consists of the average of 3 to 4 discs
** Values on the lower spectrum to be accepted with caution considering the effect of age at testing on the CCI results

4.4 COVER DEPTH

Cover tests were carried out on test areas of 1 m^2 per 30 m^2 in-situ concrete area, where several readings were taken. A minimum of three readings were taken for each bar located in the 1 m^2 area. For each structural element, several test areas were assessed as a representative area for the element. The specified concrete cover for all elements was 50 mm.

a) Numerical Summary

The cover test results are grouped per element and summarised in Table 4-9. A detailed list of the cover testing is given in Appendix 7.5. The observations made from the computed results are the following:

i) The average cover for 74% of the bridge elements exceeds the 50 mm minimum specified.
ii) Although the average cover exceeds the target for most of the bridge elements, the minimum cover could be considered the more critical aspect. This is because corrosion will initiate at a weak area with significantly low cover. Considering this, 45% of bridge elements have a minimum cover which is less than the target 50 mm. It is suggested that the extent of the inadequate cove be investigated, and remedial action taken if required.
iii) The highest average cover is 66.7 mm on Swakop River Bridge A – Pier 3. Although the cover is substantially higher than the specified 50 mm, it is important to note that this high cover will not necessarily equate to a superior durability performance. SANRAL specifications suggest that cover not exceed 60 mm to limit cracking of concrete which may substantially reduce durability (Appendix 7.2).

iv) The critical bridge elements where the average cover is significantly less than specified are on the MR52 Bridge: Abutment 1 and 5, and the TR2/2 Bridge pier. For all three of these elements the COV is 8%, indicating that the cover is consistently low in these regions.

v) The COVs vary between 5% and 20% with an average of 11%. The elements on the Swakop River Bridges with high COV and significantly high cover are the biggest cause for concern. A reasonable analysis of these cover test results would suggest poor quality control on these bridges.

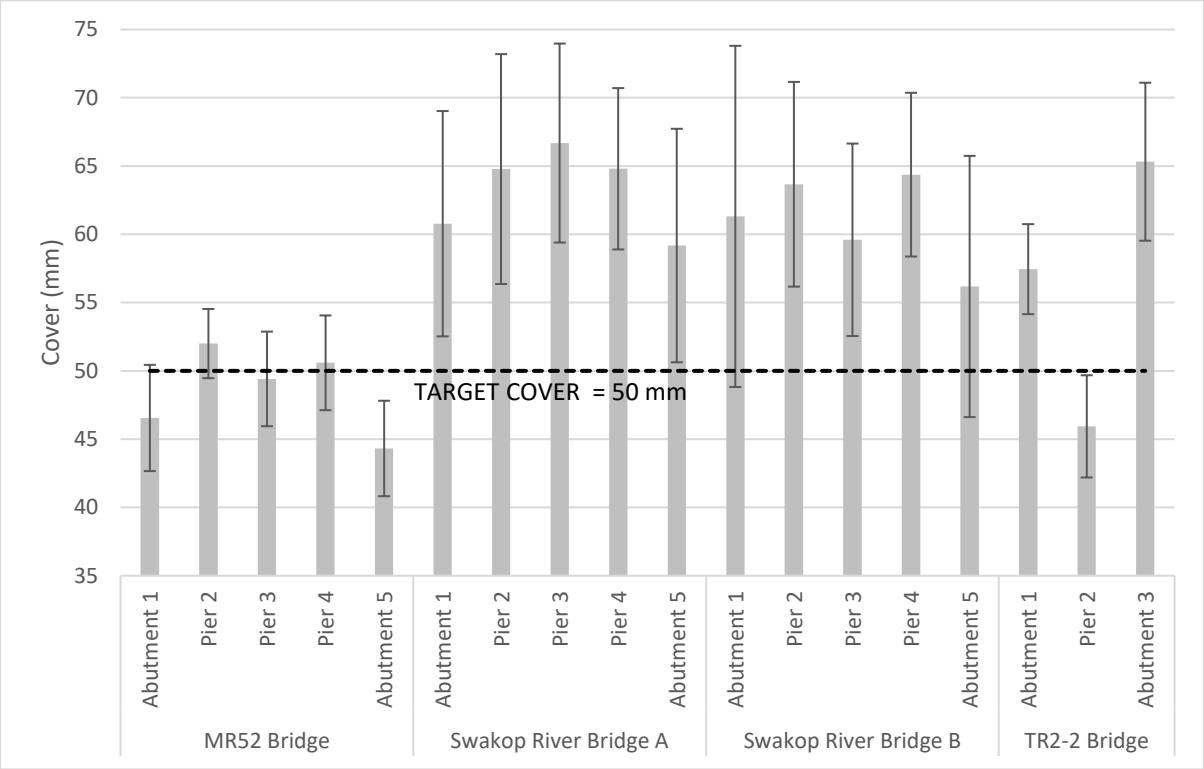


Figure 4-15 - Graphical summary of cover depth readings

Table 4-9 - Summary of cover test results

Bridge	Location	No. Test areas	No. Cover reading per test area	Total no. readings	Min. cover reading (mm)	Max cover reading (mm)	Ave. cover (mm)	Standard deviation (mm)	COV (%)
MR52 Bridge	Abutment 1	3	20	60	29	55	46.6	3.9	8.4
	Pier 2	4	25	100	45	56	52.0	2.5	4.9
	Pier 3	4	25	100	40	57	49.4	3.5	7.0
	Pier 4	4	25	100	42	57	50.6	3.5	6.9
	Abutment 5	3	20	60	36	54	44.3	3.5	7.9
Swakop River Bridge A	Abutment 1	3	15	45	51	79	60.8	8.3	13.6
	Pier 2	4	15	59	40	86	64.8	8.4	13.0
	Pier 3	4	15	60	45	78	66.7	7.3	10.9
	Pier 4	4	15	60	34	75	64.8	5.9	9.1
	Abutment 5	3	15	45	30	74	59.2	8.6	14.4
Swakop River Bridge B	Abutment 1	3	15	45	29	89	61.3	12.5	20.4
	Pier 2	4	15	60	37	74	63.7	7.5	11.8
	Pier 3	4	15	60	34	74	59.6	7.0	11.8
	Pier 4	4	15	54	53	81	64.4	6.0	9.3
	Abutment 5	3	15	45	44	78	56.2	9.6	17.0
TR2/2 Bridge	Abutment 1	4	15	60	48	62	57.5	3.3	5.7
	Pier 2	6	15	90	32	52	45.9	3.7	8.2
	Abutment 3	4	15	60	50	75	65.3	5.8	8.9

b) Frequency distribution

Figure 4-16 shows the frequency distribution for the cover readings for all bridges. Results indicate a narrower band for the MR52 Interchange Bridge although the peak cover is at 50-55 mm. The Swakop River Bridges show a generally high cover, peaking at 65 mm although the range of results is high, again indicating poor quality control. The TR2/2 Interchange Bridge peaks at 50 mm with several results either side of this.

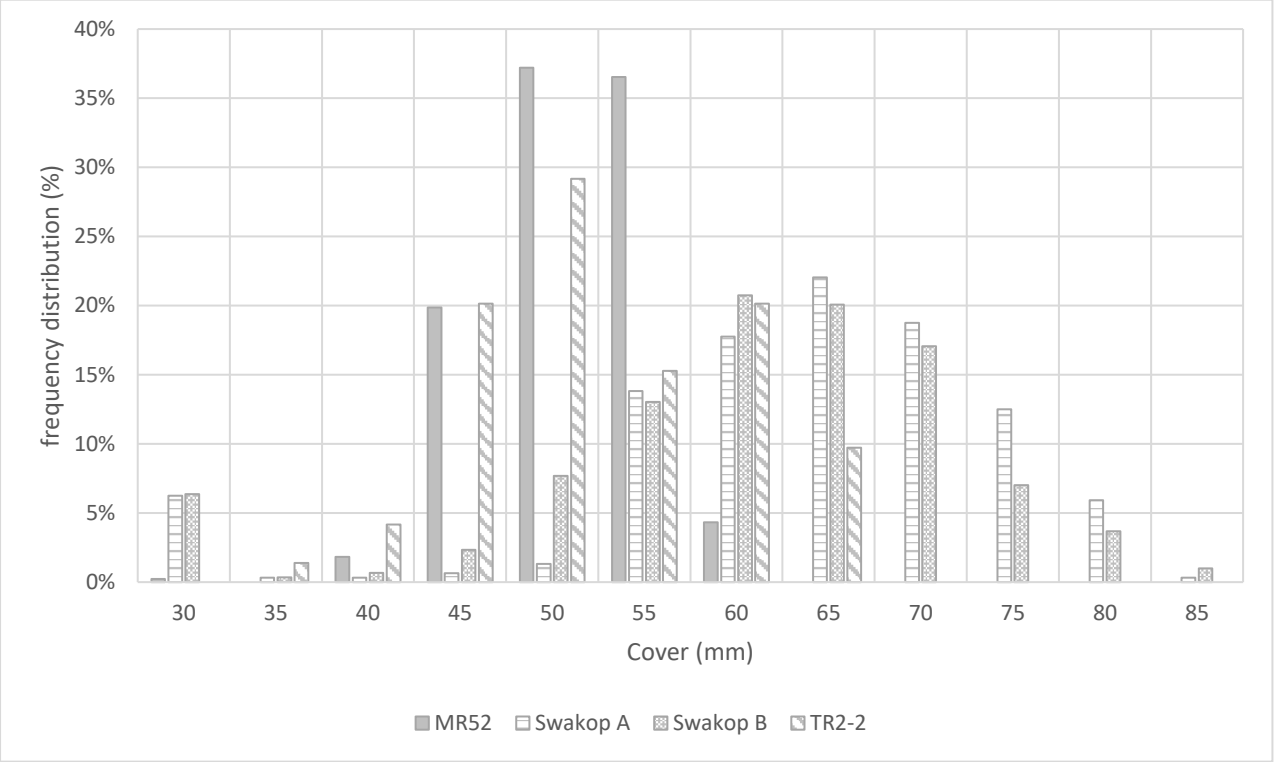


Figure 4-16 - Frequency distribution of cover readings for all bridges

c) Boxplot Comparison

Boxplots are plotted for each bridge to assess the cover readings across the different bridge elements. Figure 4-17 to Figure 4-20 are boxplot graphs for the four bridges. The following assessment is made from the boxplots:

1. MR52 Bridge:

   The bridge abutments are the most concerning with most of the readings well below the 50 mm cover limit. The cover readings for the abutment were, in fact, found unacceptable based on the project specifications and the Contractor was required to apply additional protection to the concrete faces to give equivalent cover-to-concrete. The cover on the piers is generally acceptable with most readings between 50 and 55 mm, while Pier 2 is slightly below the limit.

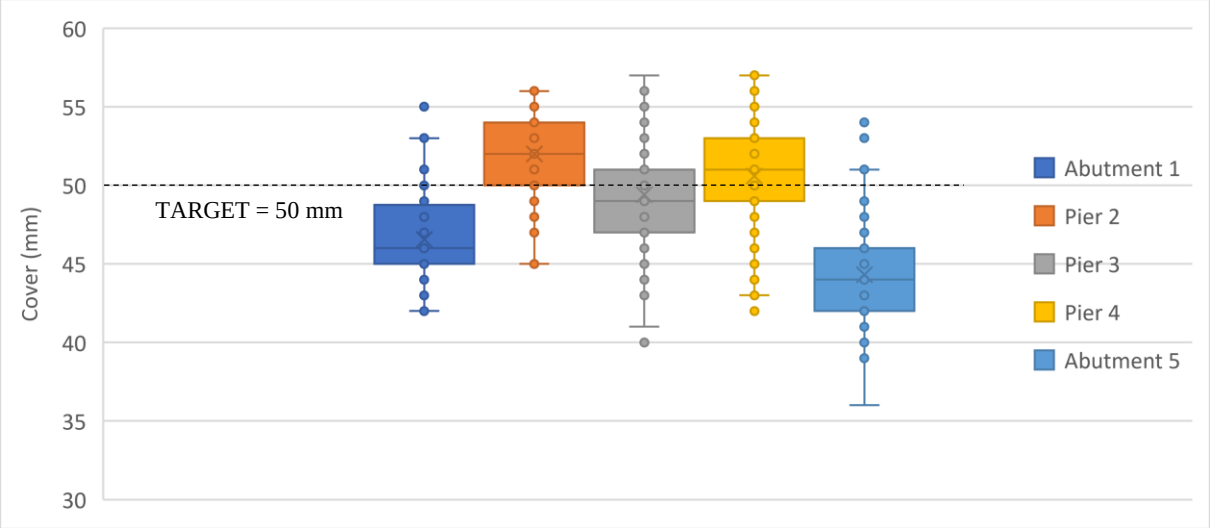


Figure 4-17 - MR52 Bridge cover readings - boxplot per element

2. Swakop River Bridge A

   Most of the cover readings for the bridge elements are substantially above the lower limit of 50 mm. There are a few outliers between 30 and 40 mm but it is likely that these readings are a binding wire or other metal considering that there are only one or two very low readings with the rest much higher. More concerning is the substantially high readings on the bridge piers which are likely to result in excessive cracking.

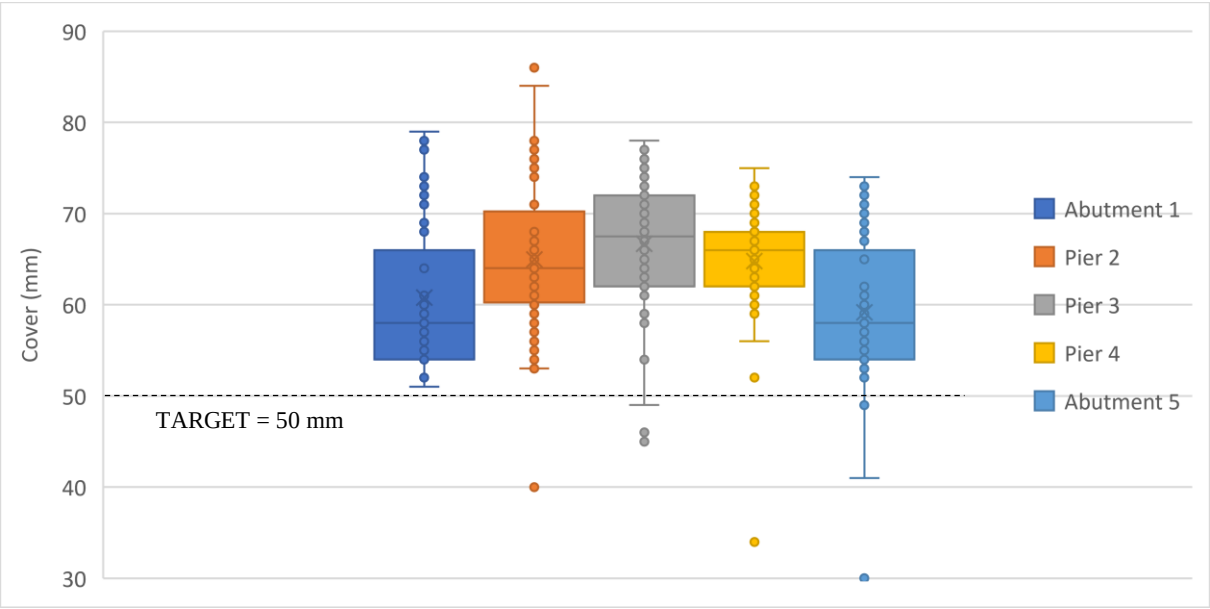


Figure 4-18 - Swakop River Bridge A cover readings - boxplot per element

3. Swakop River Bridge B

   The range of cover readings on Swakop River Bridge B are very similar to Bridge A with the abutment cover readings closer to specification and the pier cover readings in

a range above 55 mm. There is evidence of better quality-control with a narrower range of test results.

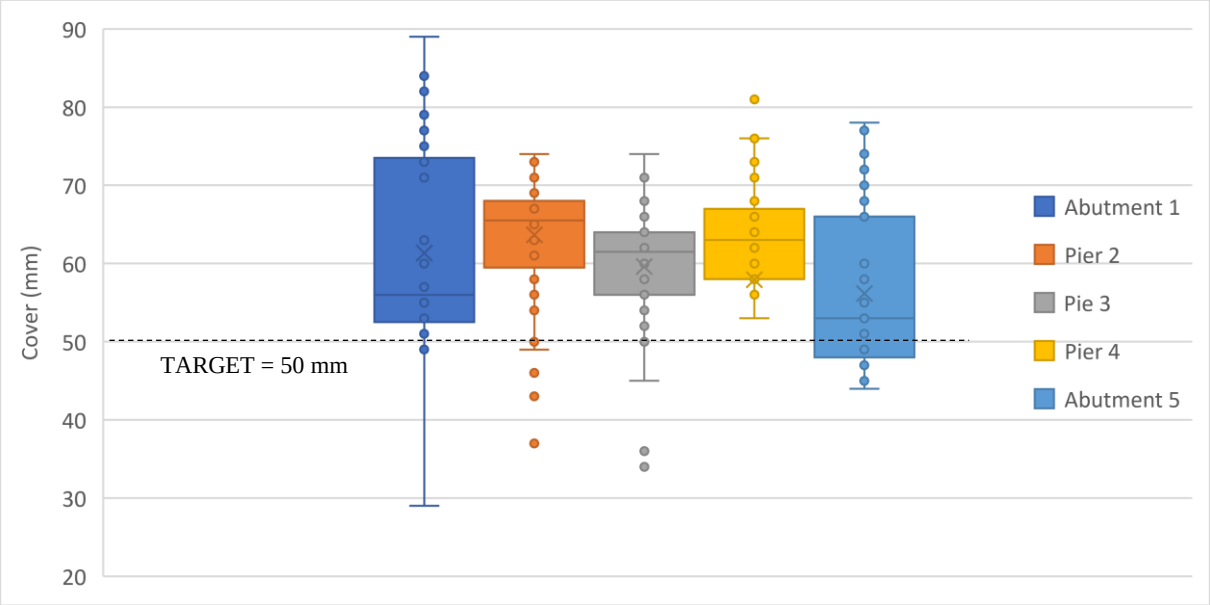


Figure 4-19 - Swakop River Bridge B cover readings - boxplot per element

4. TR2/2 Interchange Bridge

   Abutment 1 has a narrow range of results within no outliers and majority of results between 55 and 60 mm. The cover results on Pier 2 are concerning since most of the results are less than 50 mm with a significant scatter. As with MR52 Abutments, the cover results from TR2/2-Pier 2 were found unacceptable and the Contractor was instructed to apply additional coatings to achieve “equivalent” concrete cover. The cover readings for Abutment 3 are considerably higher than the prescribed 50 mm, with the mean value exceeding 65 mm. As with the Piers on the Swakop River Bridges, this excessive cover is a cause for concern since they may result in excessive cracking.

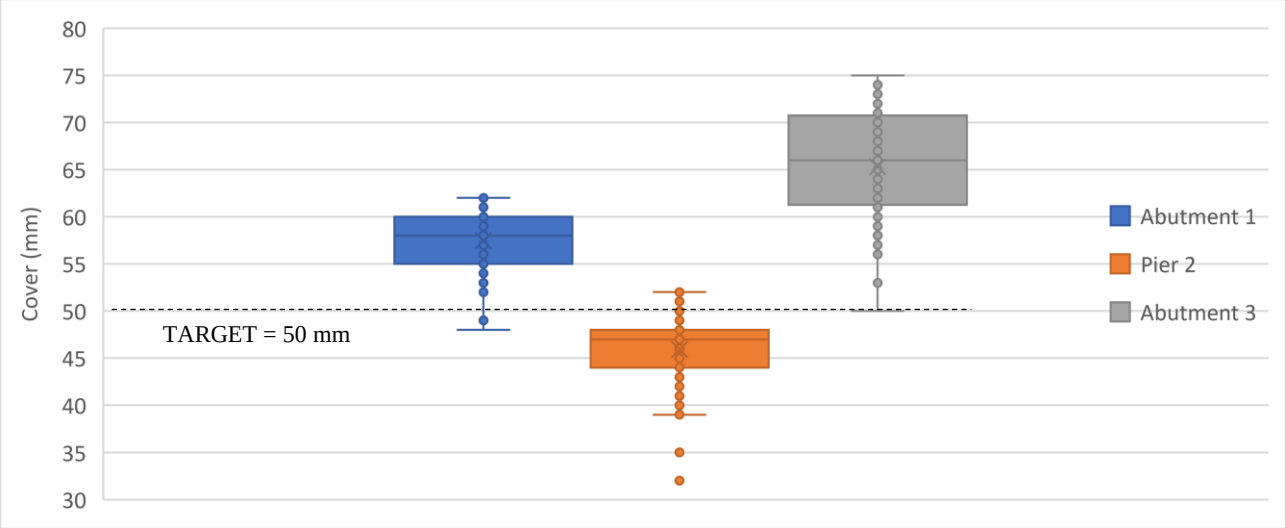


Figure 4-20 - TR2/2 Bridge cover readings - boxplot per element

4.5 SERVICE LIFE PREDICTION USING SA CHLORIDE INGRESS MODEL

Service life prediction was carried out on the critical major vertical elements in the bridges using the South African Chloride Prediction Model (Mackechnie, 2001) as detailed in 3.2.4 and Figure 3-30. The CCI results summarised in Section 4.3.4 are significantly higher for vertical elements than horizontal elements. In addition to this, the cover is often less than specified for these elements due to movements of the steel cage or shutters during concreting. Conversely, the cover to the top surface of horizontal elements is not expected to decrease during casting due to the weight of the steel cages (assuming the finished surface levels are reasonably accurate). Taking this into account, the critical elements for service life prediction consist of the piers and abutments above ground, exposed to chlorides.

4.5.1 Steps in service life calculations

a) Determination of chloride conductivity index (DI tests)

The first step in predicting the service life is determining a suitable value for CCI. Based on the variability in test results, it would be inaccurate to determine a single value for chloride conductivity. A 95% confidence band was determined based on the spread of results, assuming a normal distribution of results. The confidence band was calculated on all vertical elements. Equation 4.1 was used to determine the limiting value of the prediction interval band that will contain 95% of the CCI values for all bridge elements (Navidi, 2010).

$$\bar{X} \pm t_{n-1,\frac{\alpha}{2}} s \sqrt{1+\frac{1}{n}} \qquad (4.1)$$

Where	$\bar{X}$	=	Sample average
	t	=	Tolerance factor for t-distribution
	n	=	Sample size
	100(1+-α) %	=	% Upper and lower bound prediction intervals
	s	=	Standard deviation

For statistical analysis, all valid test determinations are used for the vertical elements. The applicable variables to determine a prediction interval are:

$\bar{X}$	=	0.91
t	=	2.045
n	=	30
100(1+-α) %	=	95%
s	=	0.219

Substituting the above values into Equation 4.1 gives a lower and upper bound prediction interval of **0.45-mS/cm** and **1.36-mS/cm** respectively. The upper bound CCI of 1.36-mS/cm is higher than any individual test result (maximum reading 1.26) and is therefore used as an ultimate maximum in the calculation of service life.

b) Conversion of CCI to diffusion coefficient

Instead of calculating a time-dependant diffusion coefficient, a long-term diffusion coefficient (50-year) was determined from the chloride conductivity index as suggested by Mackechnie (2001). From the nomogram in Figure 4-21, the 28 -day chloride conductivity index was first converted to a modified chloride conductivity index (which takes into account long-term effects such as chloride binding and continued cementing reactions), and then converted to a diffusion coefficient which can be used in service life prediction calculations.

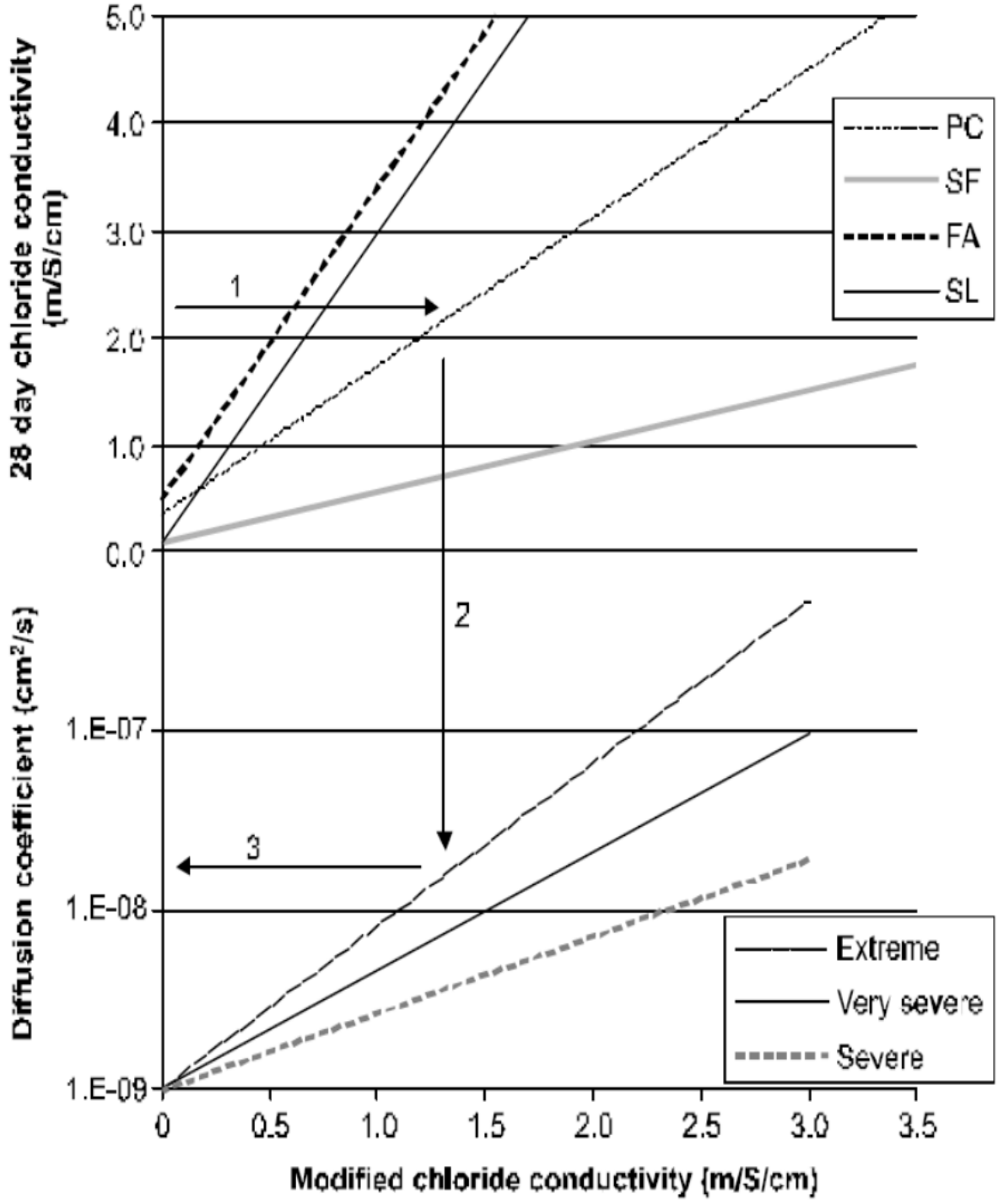


Figure 4-21 - Nomogram for the prediction of 50-year diffusion coefficient from chloride conductivity index

Using this nomogram, the calculated long-term diffusion coefficients were calculated for "very severe" environment (as determined in Section 3.1.2.1) with "structures exposed to airborne salt but not in direct contact with seawater or inland saline waters" (EN206-1, 2000). Using a concrete with a blended binder of 50: 50 GGBS: OPC, the long-term diffusion coefficients are summarised in Table 4-10.

Table 4-10 - Diffusion coefficients derived from chloride conductivity values (Adapted from Mackechnie (2001))

Limit	28-day CCI (mS/cm)	Modified long-term CCI (mS/cm)	Long-term diffusion coefficient (D_a) (cm^2/m)
Average	0.91	0.21	1.51 e-09
Lower Bound	0.45	0.05	1.14 e-09
Upper Bound	1.36	0.36	1.75 e-09

c) Estimation surface concentration of chlorides and chloride levels at the reinforcement depth (Mackechnie, 2001)

Surface chloride concentrations were adopted from those determined empirically by Mackechnie (2001). Although the bridges are not exposed to direct wave action and sea-spray, the coast of Namibia experiences very low rainfall, and there is no benefit of chlorides being washed off by rain as predicted in the model. For this reason, a surface concentration of 3.0% (i.e. equivalent to marine structures in the spray zone) was adopted.

d) Determine cover readings

Cover readings were determined for each element as described in Section 3.2.3. As with the CCI, it would be unrealistic to use a single value for concrete cover for the entire project. A 95% confidence band was calculated for the concrete cover, in addition to the cover readings per element. The confidence band was determined from all cover readings for all elements. With a large sample size, the confidence band, t, tends to 1.96 and Equation 4.1 reduces to:

$$\bar{X} \pm 1.96s \qquad (4.2)$$

Substituting the standard deviation and average cover readings, the minimum and maximum cover limits for each bridge element is summarised in Table 4-11.

Table 4-11 - Minimum and maximum cover limits

Bridge	Location	Average (mm)	Standard deviation (mm)	Lower limit (mm)	Upper limit (mm)
MR52 Bridge	Abutment 1	47	3.9	39	54
	Pier 2	52	2.5	47	57
	Pier 3	49	3.5	43	56
	Pier 4	51	3.5	44	57
	Abutment 5	44	3.5	37	51
Swakop River Bridge A	Abutment 1	61	8.3	45	77
	Pier 2	65	8.4	48	81
	Pier 3	67	7.3	52	81
	Pier 4	65	5.9	53	76
	Abutment 5	59	8.6	42	76
Swakop River Bridge B	Abutment 1	61	12.5	37	86
	Pier 2	64	7.5	49	78
	Pier 3	60	7.0	46	73
	Pier 4	64	6.0	53	76
	Abutment 5	56	9.6	37	75
TR2-2 Bridge	Abutment 1	57	3.3	51	64
	Pier 2	46	3.7	39	53
	Abutment 3	65	5.8	54	77

e) Calculation of time to corrosion activation (Mackechnie, 2001)

Using Crank's solution to Fick's second law of diffusion, the time to corrosion activation was calculated using Equation 2.4 as the time taken for the chloride concentration at the reinforcing steel level to reach the chloride threshold level (i.e. $C_{x,t} = 0.4\%$), with the surface concentration (C_s) of 3.0%.

For determination of service life, the maximum cover was capped at 60 mm as cover readings above this will tend to cause excessive cracking at which the service life model cannot be applied. Service life is calculated for three situations as an upper bound, average and lower bound limit:

1. Combination of average chloride conductivity and average cover to concrete as input parameters
2. Combination of upper bound on chloride conductivity and lower bound on cover to concrete as input parameters
3. Combination of lower bound on chloride conductivity and upper bound on cover to concrete as input parameters

Rationally, concrete cover and CCI for each bridge element will have a value that lies between these upper and lower bounds described above. Consequently, the potential service life of each bridge element will be a lifespan between the upper and lower bound extremes determined in the above combinations.

4.5.2 Calculated service life per element

The average service life is plotted for each element in Figure 4-22 along with error bars for the upper and lower bound service lives. From this graph the following assessment is made:

1. Average expected service life

   The average expected service life of the bridge elements exceeds 100-years for sixteen out of eighteen bridge elements. The two bridge elements whose average service life is less than 100-years are the MR52 Bridge Abutment 5 (92 years) and TR2/2 Pier (98 years). The average expected service life per element uses the average CCI of 0.91 mS/cm with the average cover-to-concrete per element. It can be concluded that the concrete mix has enough potential to achieve the 100-year design life if the concrete cover is kept above the minimum specified. On average, most of the bridge elements on this project will reach the intended service life before the onset of corrosion with just 11% of the bridge elements having an expected service life of less than 100-years.
2. Lower bound expected service life

   The absolute minimum service life of all bridge elements ranges between 56-years (MR52 Abutment 5) and 117-years (TR2/2 Abutment 3). These service lives are calculated using the maximum chloride conductivity of 1.36 mS/cm along with lower bound minimum confidence index per element. The large range between minimum service lives between elements amplifies the importance of concrete cover and concrete quality in the longevity of reinforced concrete bridges. The lower bound, minimum service life exceeds 100-years for only five out of eighteen bridge elements.

For practical purposes, this implies that most bridge elements will have at least a local onset of corrosion before the end design service life.

3. Upper bound expected service life
   The absolute maximum service life for all bridge elements is more than 160-years. Using a capped maximum cover of 60 mm the ultimate maximum expected service life is 220-years.

There is a significant difference between the upper limit and lower limit service lives per element when viewed from a generalised mathematical point of view only. A more detailed and systematic assessment needs to be made on the calculated service lives of the bridge elements for realistic figures to be determined. A recommended logical approach to the service calculation consists of the following:

1. The upper limit on service life is disregarded. Since there is only one CCI value used across the elements in the service life prediction, the range on upper limits on service life are solely dependent on concrete cover. It can be reasonably expected that external physical factors will reduce the service lives of the bridges long before the modelled service life reaches this upper bound. The external factor may be due to cracking, such as long-term shrinkage cracks or physical damage such as abrasion, weathering, or erosion.
2. The expected service life of the elements can be reasonably expected to fall within the range of the lower limit and the average service life.
3. Since the concrete cover is a major factor in these calculations, the lower limit can be assumed to result in the onset of corrosion in a local area with reduced cover while the average service life may be said to occur on a more widespread area. The two service life predictions (local onset of corrosion and widespread onset of corrosion) are summarised in Table 4-12.
4. The areas of concern are then areas where the lower limit is substantially lower than the target service life (e.g. < 70 years) or areas where the average service life is less than the target (i.e. 100 years).
5. Based on Table 4-12, there are five bridge elements (28%) where it is expected that the target service life will not be reached on a local area. The two most concerning elements are MR52 Bridge – Abutment 5 and TR2/2 Bridge – Pier 2, where the target service life is not likely to be met on a widespread area.

6. The principle cause of the low expected service life on these elements is the lack of cover. The Contractor was instructed to apply additional protective coatings and waterproofing to give equivalent concrete cover on these elements.

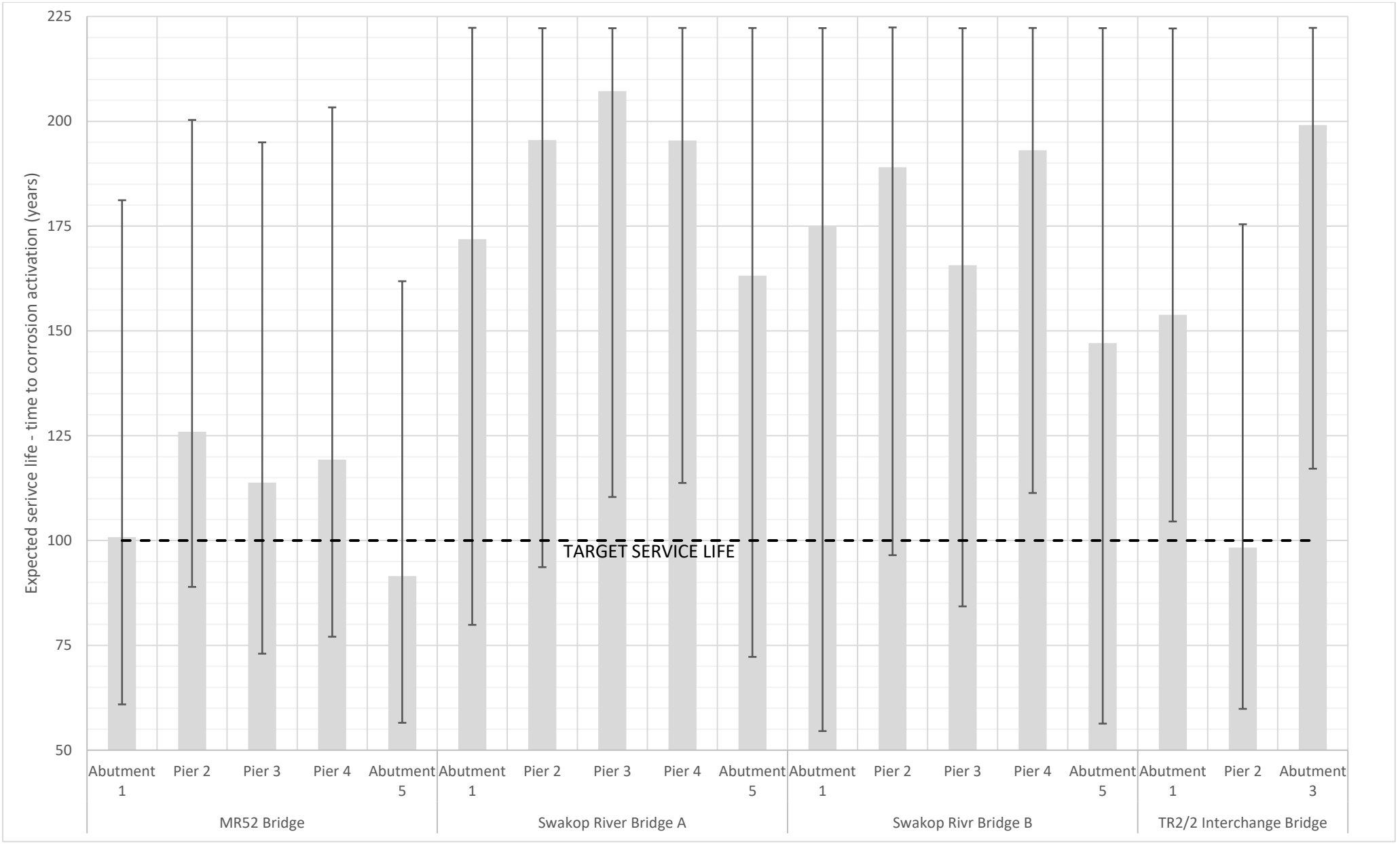


Figure 4-22 - Expected service life of bridge elements

Table 4-12 - Service life of bridge elements

Bridge	Location	Expected service life (years)		Comments
		Lower limit – local onset of corrosion	Average – widespread onset of corrosion	
MR52 Bridge	Abutment 1	61	101	Lower limit service life is a concern
	Pier 2	89	126	
	Pier 3	73	114	
	Pier 4	77	119	
	Abutment 5	57	92	Lower limit service life is a concern. Average service life less than target.
Swakop River Bridge A	Abutment 1	80	172	
	Pier 2	94	196	
	Pier 3	110	207	
	Pier 4	114	195	
	Abutment 5	72	163	
Swakop River Bridge B	Abutment 1	55	175	Lower limit service life is a concern
	Pier 2	97	189	
	Pier 3	84	166	
	Pier 4	111	193	
	Abutment 5	56	147	Lower limit service life is a concern
TR2-2 Bridge	Abutment 1	105	154	
	Pier 2	60	98	Lower limit service life is a concern. Average service life less than target.
	Abutment 3	117	199	

4.5.3 Sensitivity of SA chloride ingress model

There are several variables in the service life prediction model which have a significant effect on the final estimation of expected service life. These variables are the concrete cover, the chloride threshold and the surface concentration of chlorides. The following section analyses the SA Chloride Ingress Model's sensitivity to these variables.

a) Effect of cover (x)

The effect of concrete cover is considered in the previous section for determination of service life per element. This is indicated graphically in Figure 4-22 since the only variable between each element is the cover while average, minimum and maximum diffusion coefficients remain unchanged. Considering the same constants as Section 4.5.1, a comparison between service life and concrete cover depth is shown graphically in Figure 4-23. The specified cover for this project is 50 mm. This gives a lower bound service life of 100-years. If the cover is reduced to 45 mm, the service life is reduced to 80-years on the lower bound, while a 40 mm cover predicts a service life of only 60-years. Clearly, from this analysis, concrete cover is critically important in the service life of structures.

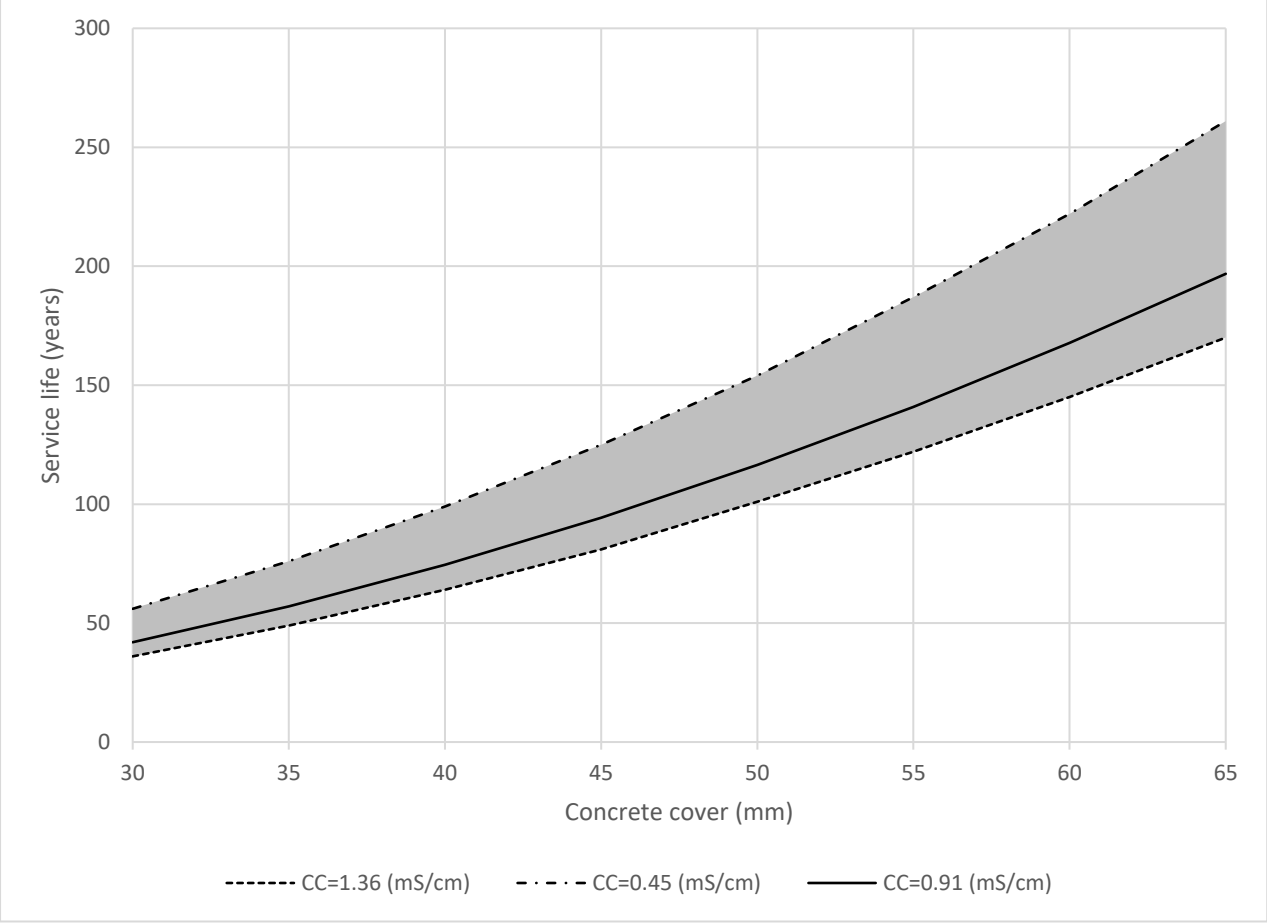


Figure 4-23 - Effect of concrete cover on service life

b) Effect of chloride threshold (Cx,t)

There are conflicting values on critical chloride levels for corrosion activation in literature (Glass & Buenfield, 1997; Oh, et al., 2003; Castellote, et al., 2002; Breit, 1998; Mackechnie, 2001). However, based on several published values (see 2.3.4.2), the critical chloride content of 0.4-0.5%, used as suggested by Machecknie (2001) can be considered as reasonable to conservative.

With other variables unchanged (cover of 50 mm aond surface concentration of chlorides 3%), Figure 4-24 shows the correlation between service life chloride threshold using the SA Chloride Ingress Model. Using a chloride threshold value of 0.4%, expected service life is approximately 100-years using the lower bound diffusion coefficient. As this increases, the expected service life increases considerably to 120-years for a chloride threshold of 0.5% and exceeding 200-years for a chloride thrshold of 0.9%. Based on this, it can be reasonably accepted the SA Chloride Ingress Model is on the conservative end in terms of chloride threshold value.

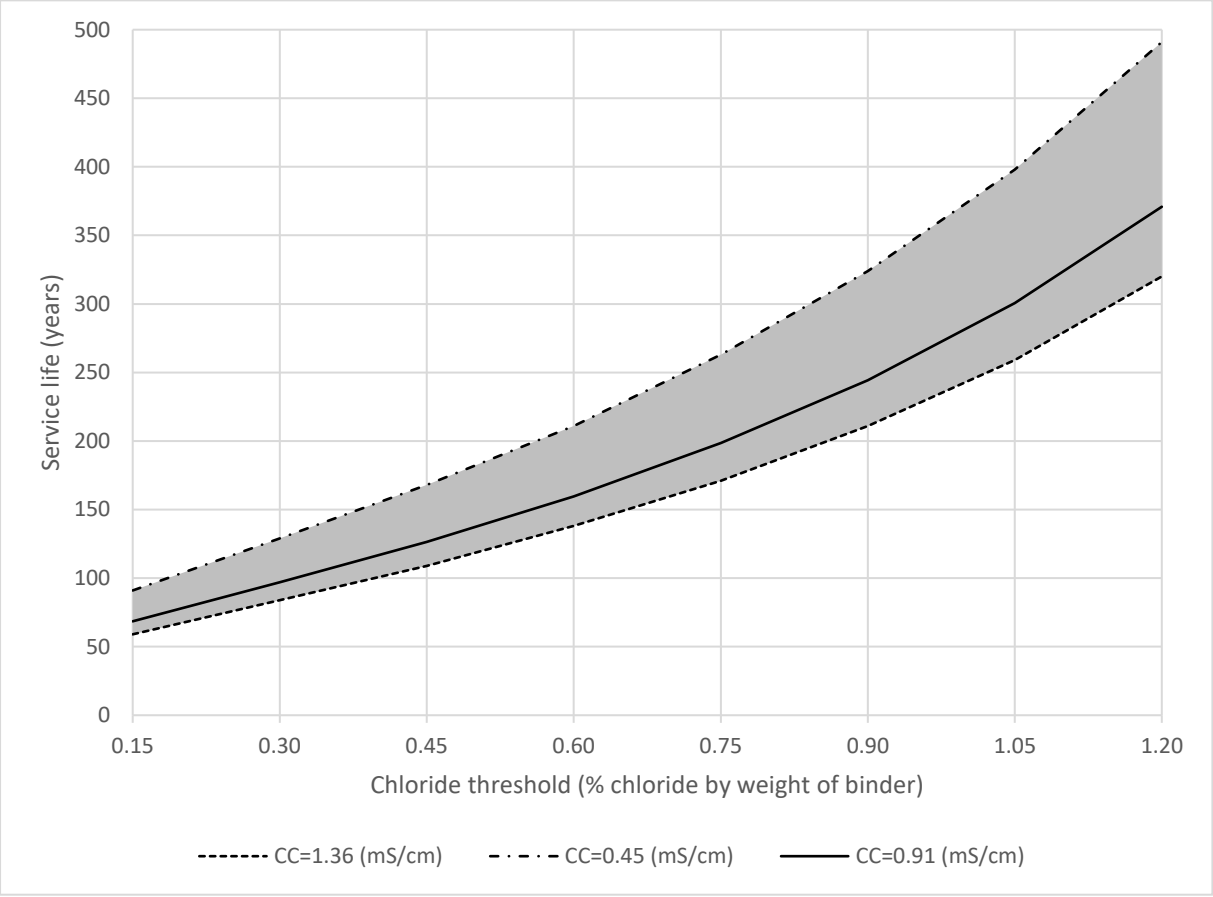


Figure 4-24 - Effect of chloride threshold on service life

c) Effect of surface concentration of chlorides (Cs)

For this project, surface chloride concentrations were adopted from those determined empirically by Mackechnie (Mackechnie, 2001). The empirical values are based on South African coastal conditions. There are, however, no published values of surface concentration of chlorides for Namibian coastal conditions and an estimate on exposure environment was required. Particularly difficult in this was the fact that although the bridges are not exposed to direct wave action and sea-spray, the coast of Namibia experiences very low rainfall, and there is no benefit of chlorides being washed off by rain as predicted in the model. For this reason, a surface concentration of 3.0% was considered conservative. The effect of varying surface concentrations of chlorides (with constant concrete cover of 50 mm and chloride threshold of 0.4%) is indicated graphically in Figure 4-25.

A service life of 100 years is expected for a 3.0% surface concentration of chlorides. If this is reduced to 2.5% (as a lower limit for concrete in the coastal spray zone), the service life is increased to 115 years. If, however, the same concrete is used in the tidal/ splash zone (with 5.0-6.0% surface concentration of chlorides) the service life is reduced to 65-70 years.

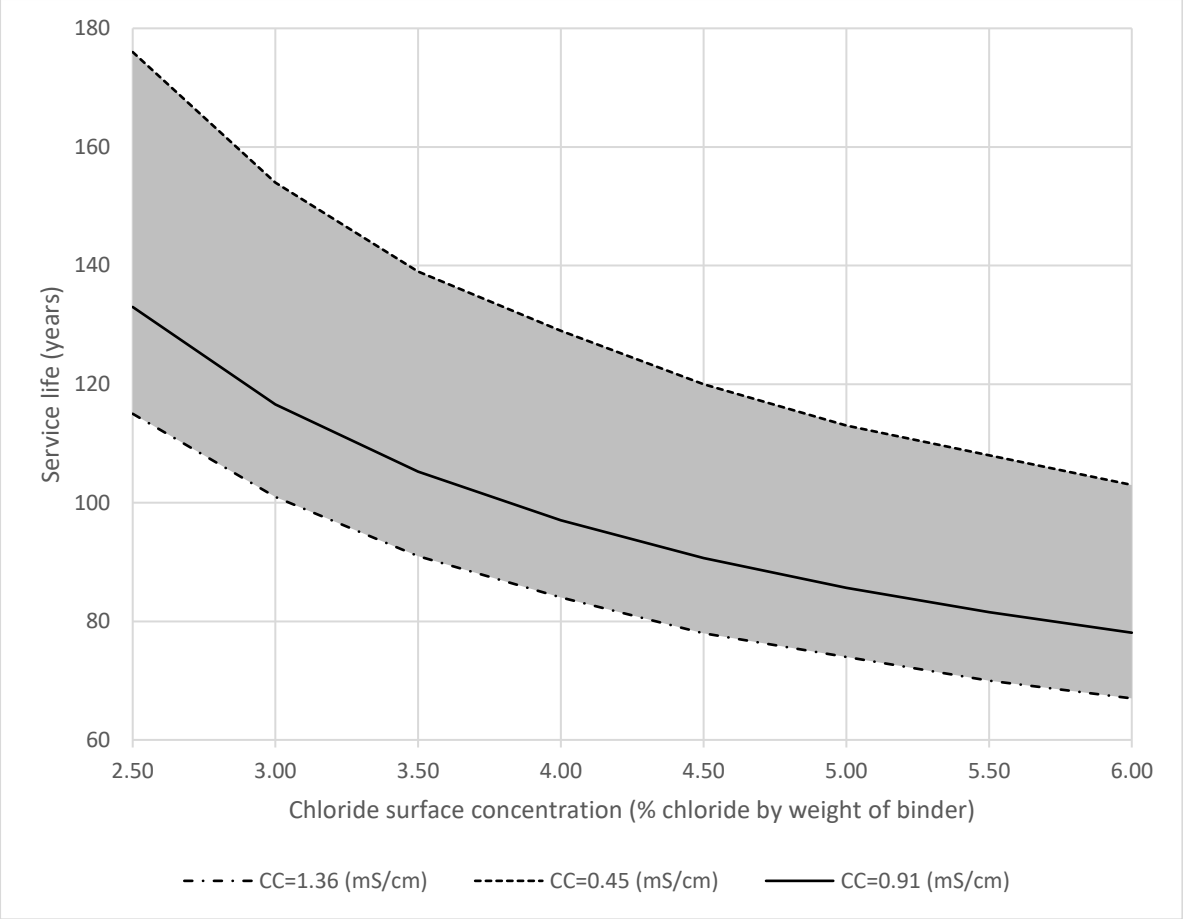


Figure 4-25 - Effect of surface concentration of chlorides on service life

4.6 SUMMARY

This chapter covered the assessment of the results of concrete strength tests as well as Durability Index tests from concrete samples and cover assessment of as built bridges. The prescribed concrete mix with limits on cement content and water: binder ratio resulted in a high-strength concrete which reached target strengths well above the design target strength of 40 MPa, with most concrete strengths exceeding 50 MPa.

The Durability Index test results indicate a high quality concrete with significant resistance to penetration by aggressive agents through permeation, diffusion and absorption, when compared to the 'deemed-to-satisfy' indexes used by SANRAL (Appendix 7.2). Importantly, the concrete used in this project has a high resistance penetration of chloride ions through diffusion, which is the principle transport mechanism of concrete exposed to chlorides.

Cover readings using an electro-magnetic cover-meter indicated significant variation in cover across several elements. The minimum cover requirement of 50 mm was generally exceeded although some bridge elements showed particularly low cover. Also, of concern were several bridge elements with excessive cover >60 mm which may crack and reduce the concrete durability.

The service life of the concrete elements was calculated as the time-to-corrosion activation using the South African Chloride Ingress Model. The expected service life of the structural elements varies between 55 and 220 years for the lower bound and upper bound. The upper bound limits are considered as improbable since they are based on bridge elements with excessive cover which are likely to crack in the long term. More realistic service lives are between the lower bound and average values. Using these limits, 28% of bridge elements show areas of early local failure and 11% of the bridge elements show a very low service life and would require additional protective measures.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 INTRODUCTION

Experimental analysis and service life modelling was carried out on concrete from bridges on the Swakopmund-Walvis Bay Freeway Upgrade. The environment consists of high chloride concentrations due to the excessive fog brought about by coastal South-westerly winds, high humidity, and low rainfall in the area.

Concrete quality and potential durability were assessed by a combination of concrete strength, Durability Index (DI) testing and electromagnetic cover readings. The potential service life of the bridge was calculated using the Chloride Corrosion Model with the CCI results determined in the laboratory and the in-situ concrete cover readings for each element. The elements in the assessment of concrete quality, durability, and service life consisted of the following.

(i) ***Evaluation of concrete consistency through concrete compressive strength***

For concrete compressive strength, one sample consists of three test specimens (i.e. 3×150 mm test cube). Specimens were crushed at 28 days and a concrete cube strength determined for each test. The average result of the three test determinations was calculated as one test result.

(ii) ***Assessment of potential durability using DI tests***

Core samples were taken from 150 mm test cubes and 600×400x×150 test panels cast on-site adjacent to the bridge element. DI test results are compared to the 'deemed-to-satisfy' limits used by SANRAL in their performance-based approach to durability design (Appendix 7.1).

(iii) ***Measurement of in-situ cover using electromagnetic covermeter***

All bridge decks, piers and abutments on the project were specified with a 50 mm cover to steel. Concrete cover was checked before casting concrete for each element. After casting and curing, bridge piers and abutments were assessed for cover using an electromagnetic covermeter. A minimum of one test area for cover testing consisted of a 1 m^2 area on the bridge elements surface randomly selected as a representative area for the entire element. Each test area represented a maximum of 30 m^2.

(iv) ***Determination of expected service life using the South African Chloride Ingress Model***

The South African Chloride Ingress Model (Mackechnie, 2001) was used to determine the time to corrosion activation and consequent service life of the new bridges. The expected service life prediction is based on a 95% confidence band for chloride conductivity and cover-to-concrete from the test results. The chloride conductivity values were converted to a diffusion coefficient based on empirical nomograms developed by Mackechnie (2001). Additional parameters used in this model are the surface concentration of chlorides (based on the exposure environment of the structure), and the chloride threshold, which has been researched and published by several researchers (Glass & Buenfield, 1997; Oh, et al., 2003; Castellote, et al., 2002; Breit, 1998; Mackechnie, 2001). An expected service life was determined for the various elements in the bridge.

5.2 CONCLUSIONS

5.2.1 Concrete Quality and Durability

Concrete quality and durability were assessed by variations in average 28-day compressive strength, Durability Index testing and concrete cover testing. The test results are summarised in Table 5-1.

a) Compressive Strength

The average compressive strength achieved was 56 MPa which is substantially higher than the specified 40 MPa. The high strength is attributed to the prescribed mix for durability, to which the concrete was designed, which consisted of a high cement content and a low w/b ratio. The standard deviation from all valid test results was 7.1 MPa. This variability suggests significant variations in material properties with time.

b) Durability Index (DI) results

The material potential DI values indicate an adequate potential durability when compared with the 'deemed-to-satisfy' targets specified by SANRAL (Appendix 7.1 and 7.2). According to the approach, the target values for OPI is greater than 9.1 (log scale) and for chloride conductivity less than 2.20 mS/cm. In addition to this, the suggested maximum sorptivity is less than 10 $mm/\sqrt{hr}$. The range of values for all DI parameters surpasses the target values.

c) Concrete cover depth

The minimum cover depth is considered the most critical aspect since corrosion will initiate at the weakest area and spread. Considering this, 45% of bridge elements have a minimum cover which is less than the target 50 mm. The effect of the inadequate cover is evident in the potential service lives of the bridge elementss (Section 5.2.2).

Table 5-1 - Summary of all test results

Parameter	Potential (Test Cube)	Target	Minimum	Maximum	Average	COV (%)
Compressive strength (MPa)	Mix design = 60MPa	40	37.9	71.3	56.5	9.9
OPI (-log(k))	10.7	9.1	9.6	10.7	10.1	2.5
WSI ($mm/\sqrt{hr}$)	5.0	10	5.5	7.7	6.6	9.3
Porosity (%)	7.2	-	7.4	12.2	9.89	13.6
CCI (mS/cm)	0.35	2.2	0.6	1.2	0.82	32.2
Cover Depth (mm)	-	50	29	89	52	14.1

5.2.2 Potential Service Life

The service life of the concrete elements was calculated as the time-to-corrosion activation using the South African Chloride Ingress Model (Mackechnie, 2001). The expected service life of the structural elements varies between 55 and 220 years for the lower bound and upper bound, respectively. The upper bound limits are considered as improbable since they are based on bridge elements with excessive cover which are likely to crack well before the potential service life is achieved. More realistic service life estimates are between the lower bound and average values. The interpretations of the lower bound and average values are the following:

- Lower bound service life: Results in the onset of corrosion in areas with low cover depths.
- Average service life: This is the result of areas with a general cover problem, resulting in the average cover depth being below specified. The assumption is that this widespread lack of cover will result in more extensive corrosion of the bridge elements, and a substantially reduced service life.

Using the two limits specified above, the expected service life of the bridges is summarised in Table 5-2.

Table 5-2 - Expected service life of bridges

	Expected service life (years)	
Location	Lower limit – local onset of corrosion	Average – widespread onset of corrosion
MR52 Bridge	57 – 89	92 – 119
Swakop River Bridge A	72 – 114	163 – 207
Swakop River Bridge B	55 – 111	147 – 193
TR2-2 Bridge	60 – 117	98 – 199

With this approach, 28% of bridge elements show areas of early local failure with potential service life as low as 56 years, and 11% of the bridge elements have a potential wider area of failure before the 100 year design life. The lower service life is attributed to inadequate cover to concrete.

5.3 RECOMMENDATIONS

5.3.1 Project recommendations

The quality of in-situ concrete on this project is generally satisfactory in terms of concrete strength and DI tests. However, there are portions of the bridges with local defects, areas of poor compaction and honeycombing, and most notably, areas with reduced cover. For the longevity of the bridges, these issues need to be addressed. The suggested repair consists of the following:

- Defects in concrete:
  Areas that show clear signs of poor concrete such as honeycombing, excessive bleeding and segregation should be repaired. These repairs will generally consist of breaking back the concrete surface to sound concrete, followed by bonded overlays or patch repairs to the area with a suitable cementitious-based repair compound.
- Insufficient cover to steel:
  The concrete cover testing with an electromagnetic cover meter indicated areas of severely deficient cover. The effects of the reduced cover are clear in the service life predictions of the elements. It is suggested that if deficient cover is determined locally that the entire element be scanned to determine the extent of the deficient cover.

Thereafter, a suitable waterproofing should be applied to give “equivalent” cover to concrete.

5.3.2 Implementation of a Durability Index based approach

The South African Durability Index approach is widely used in South Africa, but several design approaches still use specifications that are based on the strength of concrete to assess potential durability. For new structures, basing the acceptance solely on concrete strength would reassure the Engineer that the structure will not fail when loaded within the limits of the structural design. This reassurance, however, will only be valid for as long as the concrete or reinforcing steel does not undergo additional physical or chemical deterioration.

A performance-based approach similar to the specifications used by SANRAL is considered a more suitable method for or the major concrete works in this environment. The DI testing carried out on the project indicated a high-quality concrete, but also showed variability across several elements. If this method is used on a larger scale, weaknesses can be identified in construction processes, curing, and mix design. This will allow owners of concrete infrastructure to make more informed decision on inspections and maintenance procedures in the future. The DI results can also be used to predict the potential service life of the bridges, which can aid in setting up maintenance plans.

5.3.3 Improvements to SANRAL Durability Index approach

The SANRAL Durability Index (DI) approach is detailed in Appendixes 7.1 and 7.2. This is a performance-based approach which limits the DI values based on the exposure environment and concrete cover. The approach and limiting DI values have been developed from the South African “framework for use of durability indexes in performance-based design and specifications for reinforced concrete structures” (Alexander, et al., 2008). DI tests are carried out on representative concrete panels of 600×400×150 mm to allow for eight cores to be removed and cut into discs (4 discs for OPI and water sorptivity and 4 discs for chloride conductivity). The panels are cast in-situ adjacent to where the concrete element is placed, using the same shutter type and compaction method as the in-situ structure.

a) Use of representative test panels for quality control

The principle limitation on the SANRAL DI based approach is the construction of representative panels for the bridge elements. Although these are cast according with the same concrete, shutter type and curing, it is possible that the concrete in the panel may not

achieve the same DI values of the in situ concrete. A recommended future study would be a detailed comparison of DI values from in-situ concrete with those from representative panels.

b) Recommendations on Durability Index limits

The SANRAL performance-based approach gives limits on DI values based on the exposure environment and concrete cover. For the design of Highway Bridges in South Africa, the design life is 100 years (TMH7, 1981). It is expected, therefore, that the limiting DI-values will be based on achieving this service life. As an example, the exposure environment of "very severe" along with the design cover to concrete of 50 mm with 50:50 blend of OPC and GGBS is used to determine potential service life.

Using the South African Chloride Ingress Model with these values for various surface concentrations of chlorides, the expected service life is shown in Table 5-3. The potential service life is calculated using Crank's error function (Equation 2.4, repeated here):

$$C_{x,t} = C_s \left[1 - \text{erf}\left(\frac{x}{2\sqrt{D_a t}}\right)\right] \qquad (2.4)$$

Where $C_{x,t}$ = Chloride concentration at depth x at a given time t
= 0.4 (chloride threshold value using SA Chloride Ingress Model
x = Depth of reinforcement
= 50 mm
t = Potential service life (years)
C_s = surface concentration of chlorides (g / m3)
= 2.5 – 6.0 % for a very severe environment
D_a = Apparent chloride diffusion coefficient
= 2.27 e^{-9} for a CCI of 2.2 mS/cm from Nomogram (Figure 4-21)
t = Potential service life (s)
erf = mathematical error function

Table 5-3 - Potential service life of RC structures in "very severe" environment using SANRAL DI limits

Surface Concentration of Chlorides (% binder)	Potential Service Life (Years)
2.5	88
3.0	78
3.5	70
4.0	65
4.5	60
5.0	57
5.5	54
6.0	52

The results in Table 5-3 suggest that the CCI limit value from the SANRAL specifications is nonconservative and may result in a potential service life of only 50 years for a structure designed for 100 years. A simple desktop study could give more conservative target DI values. It is also shown in this study that obtaining better DI results is achievable, with values in the study ranging between 0.48 and 1.41 mS/cm, which are substantially less than the target 2.2 mS/cm.

5.3.4 Improvements to SA Chloride Ingress Model

a) Selection of input parameters

The South African Chloride Ingress Model is dependent on several factors (chloride conductivity, surface concentration of chlorides, exposure environment, and concrete cover). The sensitivity of the calculated service life to these factors was assessed in in Section 4.5.3. The prediction model is generally used at the design stage where these factors will be predicted or assumed. The model allows for a reasonable estimate of surface concentration of chlorides and environmental exposure based on the location of the element and the type of exposure to chlorides.

The most critical factor based on the sensitivity analysis is the concrete cover. The results of concrete cover testing indicate that almost half of the structural elements measure a minimum cover below the target, while one in four bridge elements measured cover to be on average less than specified. . For this reason, it is suggested that the assumed concrete cover be 5 – 10 mm below the specified value in service life predictions without in-situ tests.

b) Case study

There is data from this project of DI results, and potential service life predictions on all elements. With this initial data, the bridges could be used as a case study for the validity of DI results by comparing these with conventional durability tests (e.g. chloride profiling) at 5, 10 and 20 years. A potential service life prediction at these intervals could also be used for validation of the South African Chloride Ingress Model.

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7 APPENDICES

7.1 SANRAL DURABILITY SPECIFICATIONS

These specifications are an addendum to the standard specifications used in the construction of roads and highways in Southern Africa (COLTO, 1998). Items prefixed "B" indicate an adjustment or addition to the standard specifications.

B6404 CONCRETE QUALITY

a) General

Insert the following paragraph after the second paragraph:

"When structural concrete prefixed 'W' is shown on the drawings, it shall, in addition to the strength requirement, comply with the durability requirements specified in sub-clause 6404(h), 'W' class concrete shall not apply to minor structural elements such as side drains and catchpits except in very severe environmental conditions of exposure. Requirements for concrete quality (including any durability requirements) for concrete pavements are found in Section 7100 of the specifications."

b) Strength concrete

Replace the sixth paragraph with the following:

"Where concrete is designated by the prefix "W", e.g. class W30/19, such designations shall denote concrete achieving the durability criteria specified in the relevant tables under sub-clause B6404(h)."

Add the following sub-clauses:

"**h) Concrete durability**

(i) General

Concrete designated by the prefix 'W' shall, in addition to the requirements of sub-clause 6404(b) comply with the durability parameters described below. Durability is influenced by the materials used in the concrete, their mix proportions, transporting, placing, compacting and, in particular, curing of the finished cover concrete (concrete layer between the outermost layer of steel reinforcement and the exposed outer surface of the concrete element). The tests required to prove durability performance of the placed concrete are given under sub-clause B8106(i).

It is the engineer's responsibility to approve the component materials and their mix properties, however it is the contractor's responsibility to utilise acceptable component material and to achieve mix properties complying to the specifications. It is the contractor's responsibility to design and blend materials to produce concrete of the specified quality

(ii) Durability parameters

Water sorptivity:	Sorptivity is sensitive to surface effects and may be used to assess the effectiveness of initial curing.

Oxygen permeability:	Permeability is sensitive to changes in the coarse pore fraction and thus a means of assessing compaction of concrete. It is used to quantify the microstructure of the concrete and sensitive to macro-defects such as voids and cracking.
Chloride conductivity:	Chloride conductivity provides a method of characterisation of concretes in the marine environment and is used to assess the chloride resistance of concrete.
Cover concrete:	Cover concrete is the outer concrete layer that protects reinforcing steel. Concrete cover is a requirement for all concrete whether specified as durability concrete (Class "W") or normal reinforced concrete.
Individual Cover Depth Measurement (CDM):	Individual cover depth measurement determined by an electromagnetic cover meter, complying with BS 1881, Part 204.
Average Cover:	The average of at least 30 individual CDM's per m² determined on a clearly identified area.
Overall Cover:	The mean average cover determined for the scanned area per structure.
Scan Area:	Areas of approximately 1m², randomly distributed over the entire structure, representing at least 5% of total surface area for that structure.
Individual bar reading:	A minimum of 3 linear CDM's, spaced at 100mm intervals, representing a single bar of reinforcement.
Capped CDM:	The value applied to all CDM's in excess of the maximum allowed CDM, determined by the engineer (e.g. 40mm (specified cover) + 15mm (upper limit) = 55mm) or
Capped Value:	A value in mm, assigned to a cover reading where the raw reading exceeds the specified cover, plus a value (mm) specified by the engineer.
Quick/Linear Scan:	For evaluation of cover depth measurements taken perpendicular to closest rebar in a line covering required area to be scanned.
Image/Block/Grid Scan:	Provides an overview of rebar layout. Measurements taken over a square meter clearly indicating position of first and second layer of rebar.

Notes:
1. Water sorptivity and Oxygen Permeability tests are required to assess carbonation resistance
2. Water sorptivity, permeability and chloride conductivity tests are required to assess chloride resistance

Concrete cover:	Concrete cover is a dimensional indicator of cover concrete depth and it varies according to the

requirements of the different environmental exposure classes.

When tested in accordance with the test protocols described in B8106 for each potential durability parameter, the concrete shall meet the limits listed in tables B6404/4 and B6404/6. ***compiler to list only table that is applicable***

(iii) Cement content

In order to meet the durability criteria, the proportions of cementitious binder used shall be determined to suit the fine and coarse aggregate and cement type used in order to achieve the durability limits specified in tables B6404/4 and B6404/6 under the Acceptance Category of "Concrete made, cured and tested in the laboratory."

In order to avoid the possibility of Alkali Silica Reaction (ASR), the following shall be taken into account when designing the mixes:

1. Where the cementitious contents is less than 350kg/m^3, the maximum equivalent sodium monoxide content (calculated as Na_2O) permitted shall be 0.60%, unless a test certificate from the CSIR (Built Environment) is provided stating that the long term testing has proved the aggregate to be non-reactive.
2. Where the cement content exceeds 350kg/m^3, the maximum equivalent sodium monoxide content permitted shall be 2.1kg/m^3 of concrete.
3. Where potentially reactive aggregate is used, the maximum cement content shall be 400kg/m^3 and the equivalent sodium oxide (Na_2O) content permitted shall be 2.4kg/m^3 of concrete.
4. The contractor shall prior to the use of cement provide test certificates from an approved laboratory confirming the equivalent sodium oxide (Na_2O) content of the batch of cement to be used.
5. Special literature should be consulted e.g. Fulton's Concrete Technology.

(i) Environmental Classes of Exposure

For this project, the environmental classes for carbonation and chloride exposure for the different structural elements are as shown below in Table B6404/3.

TABLE B6404/3: ENVIRONMENTAL CLASSES OF EXPOSURE FOR ELEMENTS OF STRUCTURE

Element	Carbonation Environment (OPI)	Chloride Environment (Chloride Conductivity)
Foundations	n/a	XS1
Substructures	XC3	XS1
Superstructures	XC3	XS1

Note to compiler: Insert relevant environmental classes, from Table 600/1 – Concrete Durability Specification Targets (Civil Engineering Structures only) – Update 12 August 2009. It is available on SANRAL's website at www.sanral.co.za with the link to "Service Provider Zone > Manuals, Policies & Technical > Technical Specifications". Consideration needs to be given to each structural element e.g. foundations may be subject to chloride attack such as in estuaries or located in salt pans and rivers, whereas the decks may be only subject to carbonation. In addition, if the structure is not within a chloride environment, delete the last column of Table B6404/3.

(v) Acceptance ranges

TABLE B6404/4: DURABILITY PARAMETERS ACCEPTANCE RANGES

Acceptance Category	Test No./ Description/ Unit				
	Water Sorptivity (mm// h)	Oxygen Permeability (log scale)			
		Parapets	Sub-structures	Super-structure	Etc for other members
Concrete made, cured and tested in the laboratory using Trial Panels	<10.0	>a*	>c*	>e*	> g*
Full acceptance of in-situ using Test Panels	<10.0	>a*	>c*	>e*	>g*
Conditional acceptance of in-situ concrete based on results of Test Panels	Not applicable [2]	b* - a*	d* - c*	f* - e*	h* - g*
Rejection based on results of Test Panels	Not applicable [2]	<b*	<d*	<f*	<h*

Notes:

1. A value has been given, but the value to be adopted shall be based on the results from design mixes.
2. Although no value has been given due to ongoing research, values above 12 are regarded as poor quality concrete.
3. For purposes of interpretation, substructure is deemed to be all supporting elements below the deck (superstructure), including buried lengths of columns, etc, but excluding foundation elements like bases and spread footings.

**** Note to compiler: The limiting values for OPI to be inserted in Table B6404/4 i.e. values a, c, e and g shall be the "Recommended" values and values b, d, f and h shall be the "Minimum" values obtained from Table 6000/1, discussed above. The values to be used are based on the cover requirements and therefore vary for the different cover ranges as well as environmental categories chosen.***

Note to compiler: Only insert the specifications below for chloride conductivity if elements of a structure are within a chloride environment – See Table B6000/1 for requirements for a chloride environment - else delete specifications below up to and including Table B6406/5

Chloride index testing is required where specified by the engineer. The maximum water: binder (only blended binders omitted) content of concrete used in the XS environment classes shall be 0,55.

Chloride conductivity testing shall be used for materials selection and design of mixes in aggressive chloride conditions. It will therefore only be used as a check on mix designs during the initial stages of construction (based on trial panels) and on the test panels constructed as described in clause B6404(i). Testing shall be undertaken during the construction as described in clause B8106(i).

Where a cement blend is used which is different from that given in Table B6404/5, the appropriate limits shall be discussed with the engineer for approval.

TABLE B6404/5: APPROPRIATE LIMITS FOR CHLORIDE CONDUCTIVITY
(mS/cm) (For a range of possible cement blends, with minimum cover of 50mm)

ENV Class	70:30 CEM1 : FA	50:50 CEM1 : GGBS	50 : 50 CEM1 : GGCS	90 : 10 CEM1 : CSF
XS 1	*Provide value*	*Provide value*	*Provide value*	*Provide value*

XS 2a	*Provide value*	*Provide value*	*Provide value*	*Provide value*
XS 2b, XS 3a	*Provide value*	*Provide value*	*Provide value*	*Provide value*
XS 3b	*Provide value*	*Provide value*	*Provide value*	*Provide value*

FA=Fly Ash, GGBS = Ground Granulated Blast furnace Slag, GGCS = Ground Granulated Corex Slag, CSF = Condensed Silica Fume

Note to compiler: Insert values in Table B6404/5 from Table 6000/1 on SANRAL'S website. The values are chosen based on environmental class, cement blend and minimum cover requirements.

Notes:

1. Table B6404/5 shows typical blends of cements only with appropriate chloride conductivity values. These are minimum values to be achieved in the as-built structure. Cement blends used under the contract that differs from these must be tested in the laboratory to obtain appropriate values.
2. Acceptable cement blends shall be as approved by the Design Engineer.

TABLE B6404/6: DURABILITY PARAMETERS ACCEPTANCE RANGES: COVER FOR ALL CONCRETE TYPES

Test No.	Description of Test	Specified Cover (mm)	Acceptance Range	
			Min	Max
			Overall cover	Overall cover
B8106(g) (iv)	Concrete cover to reinforcement (mm)	30 to 80	85% of specified cover – 5mm	Specified cover + 15mm or where member depth is less than 300mm the limit accepted in writing by Design Engineer.

(v) Site Testing

To ensure that the concrete has been placed, compacted and cured correctly, a number of tests shall be carried out on the concrete by an approved laboratory.

(vi) Non-compliance with specified criteria

The Contractor should also note that there is specific provision made for curing of concrete under payment item B64.07 of the project specification. The amount priced under this item will be subject to reduced payments should the durability tests indicated under B8106(h) fail to meet the required targets. Similarly, failure to achieve the required durability test results will be sufficient cause to apply partial payment factors for all the pay items of the elements of the structure under sections 6300 and 6400 of the standard and project specifications or in some cases the removal of the rejected concrete.

Add the following sub-clauses:

"**i) Mix design approval procedures**

(i) General

The compressive strength achieved on 'W' class concrete shall generally exceed the characteristic strength class structurally required. The contractor shall note that the process of finalising 'W' class mix designs could take up to two months. In order to expedite the process, the contractor must submit samples of aggregate and cement to an approved laboratory within seven days of the Commencement Date. Should 'W' class concrete be required

before the mix design is finalised, the engineer will approve a preliminary mix design in consultation with the contractor.

(ii) Laboratory designs and site tests based on Trial Panels

Good mix design practice is essential and the following criteria shall be taken into consideration when pricing and determination of the mix design:

1. Selection of sands and aggregates to achieve a good grading is important if the desirable concrete density and durability have to be achieved.
2. The selection and use of the correct cement grade and type for the environmental conditions (and not based solely on costs) is fundamental
3. Water: cement ratios are critical, dictating both the structural strength and the durability requirements

Mix proportions for the concrete to be used on site need to be determined by an approved laboratory, Cylindrical specimens, 70 ± 2mm in diameter shall be made or cored from a trial panel during the laboratory trial mix for performance of tests B8106(g)(i), (ii) (if required).

Note that concrete cubes are not cored for durability testing during design trial mix stage or during the construction stage.

Testing for approval purposes shall be carried out by an accredited laboratory approved by the engineer, the costs of which are deemed to be included in the contractor's rates for structural concrete. Concrete as designed shall satisfy the limits set out in Table B6404/4 under the heading "Concrete made, cured and tested in the laboratory, using Trial Panels". It is therefore a requirement that the trial panels be cast on the site and the cores extracted and tested in the laboratory as part of the mix design approval process. Where the site is remote from the laboratory, the Trial Panels may be cast at the laboratory in accordance with the requirements of sub-clause B8106(g).

It will be necessary for the contractor to establish a target mean strength with a margin above the minimum requirement so that small fluctuations due to material changes or workmanship can be accommodated. In general, mean target strength = characteristic strength + $1{,}645 \times S_n$.

Once the mix is approved, the target mean compressive strength for quality control purposes for durability class concrete shall be the mean compressive strength obtained from the mix that satisfies the durability requirements."

B6414 QUALITY OF MATERIALS AND WORKMANSHIP

a) Criteria for compliance with the requirements

Add the following paragraphs after the first paragraph:

"The cores shall be taken from the Trial Panels cast using the design mixes made in the laboratory. Where the site is remote from the laboratory, the Trial Panels may be cast at the laboratory in accordance with the requirements of sub-clause B8106(g).

In the event that for 'W' classed concrete strength requirements the actual achieved average cube strengths of an element are less than 85% of the target mean strength needed to meet durability requirements or less than 100% of the target mean strength to meet strength requirements, it may result in the durability parameters not meeting the prescribed targets and the engineer will instruct the taking of cores from

the test panel and structure for additional testing. The cost of these in-situ tests shall be borne by the contractor.

The approved quality control criteria for process control testing for durability concrete shall be coring and testing of test panels. The frequency of manufacture and coring of test panels shall be as ordered by the engineer and indicated in Tables B8106/1 and B8106/2.

Note to compiler:
When assessing the quantity and frequency of the test panels due consideration shall be given to the different concrete suppliers used under the contract, the number of concrete pours taking place in the day and various elements of bridges being cast on the same day. It may mean that initially the minimum requirements are followed and once a trend with respect to the test results are gathered, that the frequency of manufacture and testing of the test panels can be reduced.

Tests B8106(g)(i), (ii) and (iii) (when required), shall be conducted on cores extracted from the test panels when the concrete reaches the age of at least 28 days. To allow for variability in the material potential, the type of chloride conductivity values shall be limited to 90% of the values indicated in table B6404/5. Test no. B8106(g)(iv) shall be conducted to confirm that the specified depth of concrete cover has been achieved. The frequency of these tests shall be as described under item B8106(g). The test results shall be accepted or rejected on the criteria set out in Table B6404/4 and B6404/5 based on the following categories:

(i) Full acceptance

Concrete shall be accepted unconditionally and full payment shall be made.

(ii) Conditional acceptance

Concrete may be accepted, based on the cube strength and durability index results with a warning that construction methods be examined to improve the durability criteria. A reduced payment shall be applied to all the relevant pay items of the specific element under B6300 where the cover requirements are not achieved and B6400 where the oxygen permeability and strength requirements are not achieved for the non-conforming element or concrete pour as set out in Tables B8212/1 and B8212/2. The decision to accept the substandard concrete at reduced payment shall rest solely with the Employer.

Should the test result(s) indicate conditional acceptance of the element tested, the Contractor shall have the option of carrying out additional tests (on 4 extracted cores) on that element of the structure, at his own expense to confirm or disapprove the original test result(s). These cores shall be extracted within 56 days from the date of the element being cast.

Should the additional test confirm the original test result, then the original test result shall serve to determine payment in accordance with Tables B8212/1 and B8212/2.

Should the additional test show that the structure meets the targets, the penalty shall be halved."

(iii) Rejection

The concrete shall be removed and replaced with fresh concrete at the expense of the contractor, as directed by the engineer.

"**B8106 TESTING THE CONCRETE AND COVER TO STEEL REINFORCEMENT**"

Add the following sub-clauses under B8106:

"**(g) Trial panels for durability concrete (W class concrete**)

As part of the durability class concrete mix design approval process, trial panels shall be constructed on the site (or at the laboratory) before construction of structural elements commences, to ensure that the contractor can successfully achieve the oxygen permeability and sorptivity targets set for the in-situ concrete with method of construction to be adopted. Each trial panel shall be constructed using the same type of concrete mix, shuttering type, placing and curing methods (including application rates of curing compounds if applicable) as to be used on the final structural element to be constructed. The dimensions of such a trial panel shall be 0.40m wide, 0.60m high and 150mm thick. The panel shall be constructed vertically. It is suggested that 2 lifting hooks be cast into the panel to facilitate lifting, moving or disposal of panel. It most likely will be that one trial panel will be required for substructures (piers, abutments, retaining walls, etc) if the same grade concrete is specified for all substructures and another for the decks due to type of casting and curing methods.

The test area for taking of cores (taken in horizontal direction) shall not be less than 100mm from all horizontal and vertical edges. The number of cores to be extracted and tested is described under B8106(i).

(h) Test panels for durability concrete (W class concrete)

During casting of concrete on site, test panels shall be constructed on the site adjacent to where the concrete element is being placed. Each test panel shall be constructed with the same concrete, shutter type, compaction and curing methods being used in the element being cast (including same vibrator frequency and curing compound application rates), and be left to cure for 28 days adjacent to the concrete element. Thereafter it shall either be cored on site or transported to the laboratory for testing of the required durability parameters. The dimensions of the test panels shall be 0,4m wide, 0,6m high and 150mm thick and be cast vertically to simulate vertical casts of the substructures and vertical faces of bridge decks. It is suggested that 2 lifting hooks be installed at both top ends of the test panel to assist with transport. For precast concrete, test panels will not be constructed, as cores will be drilled from the concrete elements at the Precast yard before being placed at its final location. For the horizontal faces of in-situ bridge decks and culverts, test panels will also not be constructed. Instead cores will be extracted from the top surface of the decks.

The frequency of the testing and number of cores to be extracted is described under B8106(i).

The test area for the taking of cores (taken in a horizontal direction) shall not be less than 100mm all horizontal and vertical edges.

The costs for construction of the test panels shall be deemed to be included under rates for pay item 64.01."

(i) Testing for concrete durability

Durability predictions for durability concrete prefixed 'W' will be based on the following tests that shall be carried out by an accredited laboratory approved by the Engineer:

(i) Oxygen permeability
(ii) Water sorptivity
(iii) Chloride conductivity (if specified)

Notes:
The test methods shall be as described below.

For test no's (i) and (ii) (and (iii) when required), cores of 70 ± 2mm diameter shall be extracted from the test panels when the concrete reaches the age of at least 28 days and tested for the durability criteria set out in clause B6404(h) and used to determine the payment as per Table B8212/1. Test No. (iii) may only be required where specified (e.g. within a chloride environment along the coast or where chlorides are present in ground water).

A sample for the purposes of durability testing is as defined in Table B8106/1. The cores for durability testing shall be extracted from the test panels for process and acceptance control (at the frequency as shown in Table B8106/2). Durability testing shall only be required for concrete specified as durability concrete with the prefix "W". The number of samples to be taken shall be as shown in Table B8106/2.

TABLE B8106/1: NUMBER OF CORE RESULTS REQUIRED FOR A SINGLE SAMPLE FOR DURABILITY TESTING

Durability Parameter	No. of Core Results
a. Sorptivity	2
b. Oxygen Permeability	4
c. Chloride Conductivity	4

* Test undertaken only if specified and within a chloride environment.

TABLE B8106/2 : NUMBER OF TEST PANELS REQUIRED FOR DURABILITY TESTING

Element	No. of Test Panels to be taken (see Table B8106/1 for number of core results required for a single sample)
In-situ Bridge Decks	1 (per pour)[1]
Bridge Piers/Abutments	1 (per element)[2]
Precast Elements	1 (per element)[2], [3]
Bridge / Culvert Parapets	1 (per element)[2]
Culvert walls / wingwalls / slabs	1 (per wall section)[1], [2]
Retaining walls	1 (per wall section)[2]
All bases	1 (per element/pour)[2]

Note:
1. Test panels required to be cast vertically. Additional cores required to be extracted from top of deck / major culvert slabs, i.e. in-situ cores.
2. Note that where group of elements are cast on the same day, only one test panel will be required, but only if the same grade concrete is used.
3. Sample required to be taken from Precast element in casting yard. For edge beams, inner face to be cored.

For cores to be extracted from precast elements and top of bridge decks, the Engineer will indicate the positions at which the cores will be extracted. Filling of the holes left by the drilling of the cores shall be the responsibility of the contractor and shall be carried out using an approved proprietary non-shrink repair mortar so as to restore structural integrity and durability of the structural element tested.

If the test results indicate that the durability requirement has not been achieved, then the structural element shall be cored and tested for the durability criteria. The Engineer will indicate the positions at which the cores will be extracted. The costs for testing of the structure shall be borne by the contractor. Filling of the holes left by the drilling of the cores shall be the responsibility of the contractor and shall be carried out with material as described in the paragraph above.

Note that if testing has to be undertaken on sides of decks and walls, the cores shall be taken on the exposed faces of the concrete i.e. the sidewall face taking care not to cut the reinforcing bars. Where the cores do contain pieces of reinforcing steel, they shall not be used for the tests. The cores shall be extracted through the cover concrete from the Test Panels or constructed concrete element as applicable. The outer 5mm of the exposed surface of the core shall be cut off and then a slice (30 ± 2mm thick) shall then be cut and prepared for testing. The Engineer will indicate the positions at which the cores will be extracted.

The methodology and latest revisions for the durability index tests are available at the University of Cape Town's web address at www.civil.uct.ac.za. In addition, the results of all the durability testing shall be submitted at least once a month in the required format to the University of Cape Town, where the present contact person is:
Dr. H Beushausen - email: hans.beushausen@uct.ac.za.

7.2 SANRAL TABLE 6000/1: CONCRETE DURABILITY SPECIFICATION TARGETS

Carbonation-Induced Corrosion (from Atmospheric & Industrial)

Design-ation	Description	Condition of Exposure	Description of Exposure	Typical Examples where applicable	Recommended Minimum Cover (mm)	In-situ Durability Index for various Cover Depths within Exposure Condition - 100 Year Life				
							OPI (log scale)		Sorptivity (mm/h)	
						Cover Depth (mm)	Recommended value	Minimum value	Recommended value	Maximum value
XC1a	Low hum. (<50%); exter. conc. sheltered from moisture, arid areas; interior concrete	Mild	Inland dry areas - arid to semi-arid, Karoo etc. Very low (<40%) to low humidity (40% - 50 %). Concrete surfaces not in contact with ground, protected against wetting.	Arid areas, infrequent rain: all exposed members; sides of decks & beams; deck soffits; enclosed surfaces (e.g. interior of box girders); surfaces protected by waterproof cover or permanent formwork not likely to be subjected to weathering; interior members in buildings;	40	40 mm min. cover	N/A	N/A	10.0	12.0
XC1b	Permanently wet or damp		All areas with access to external or environmental moisture Saturated conditions (RH >95%). Concrete surfaces above ground level kept permanently moist by exposure to water; concrete that never appreciably dries. Concrete surfaces below ground such as piles and buried foundations or abutments kept permanently damp.	Partially submerged and hydraulic structures kept permanently damp; drainage & other elements kept moist; surfaces in contact with permanently damp soil; surfaces kept damp by condensation or moisture; piles (both dry cast and against casings)	40	40	9.20	9.00	10.0	12.0
						50	9.00	9.00	10.0	12.0
						60	n/a	n/a	n/a	n/a
XC2	Wet, rarely dry	Moderate	All areas with access to external or environmental moisture Concrete surfaces above ground level kept mostly in moist condition by exposure to water; concrete may occasionally dry for appreciable periods such as when tanks are emptied	Partially submerged and hydraulic or drainage structures kept mostly damp; surfaces in contact with mostly damp soil; surfaces kept mostly damp by condensation or moisture; all wet or mostly damp surfaces which may occasionally dry for limited periods	40	40	9.40	9.00	10.0	12.0
						50	9.10	9.00	10.0	12.0
						60*	9.00	9.00	10.0	12.0
						70*	n/a	n/a	n/a	n/a
XC3	Moderate Hum. (50-80%). Ext. conc. sheltered from rain in non-arid areas		Near-coastal areas with no chlorides; moist inland areas; adjacent to dams, lakes, major rivers Moderate humidity (50% to 80%), moist climate. Exterior concrete surfaces in moist areas or adjacent to major water bodies, permanently sheltered from rain or direct surface moisture	Moist areas: sides of beams protected from direct rain; deck soffits; enclosed surfaces (e.g. interior of box girders); surfaces protected by waterproof cover or permanent formwork not likely to be subjected to weathering. Consider additional cover at edges of deck at expansion joints, soffits of cantilevers and parapets.	40	40	9.40	9.00	10.0	11.0
						50	9.10	9.00	10.0	11.0
						60*	9.00	9.00	10.0	11.0
						70*	n/a	n/a	n/a	n/a
XC4	Cyclic wet and dry	Severe	All areas with access to external or environmental moisture; arid areas excluded Moderate humidity (50% to 80%), moist climate. Concrete surfaces exposed to rain or alternately wet and dry conditions	All exterior surfaces exposed to rain; surfaces where heavy condensation takes place; surfaces alternately wetted and dried by drainage or environmental moisture, such that moisture may penetrate concrete member.	45	40	9.60	9.20	10.0	10.0
						50	9.30	9.00	10.0	10.0
						60*	9.10	9.00	10.0	10.0
						70*	9.00	9.00	10.0	10.0

WARNING: Covers shown with a asterisk (*) should be avoided so as to (i) limit crack widths, and (ii) ensure durability concrete is being specified and must be discussed with the client before being specified.
NOTE: Heavily Polluted Industrial Areas : Increase Cover for any exosure Condition above by 10mm

Chloride-Induced Corrosion (from Groundwater, Seawater & Sea spray)

Designation	Description	Condition of Exposure	Description of Exposure	Typical Examples where applicable	Recommended Minimum Cover (mm)	In-situ Durability Index for various Cover Depths within Exposure Condition - 100 Year Life						
							Chloride Conductivity (mS/cm)				Sorptivity (mm/h)	
							Typical Binder Blends					
						Cover Depth (mm)	70:30 CEM1:FA	50:50 CEM1:GGBS	50:50 CEM1:GGCS	90:10 CEM1 : CSF	Recommended value	Maximum value
XS1	Exposed to airborne salt but not in direct contact with seawater or inland saline waters	Very Severe	Proven presence of chlorides; generally < 1km from sea, and coastal river valleys (where chlorides are present) and estuaries, or the presence of chlorides proven by experience or testing. This will include inland salt pans or groundwater carrying slats, etc	All exposed and external surfaces subject to significant airborne salt; any surface on which salt can deposit from the air.	50	40	1.50	1.60	2.10	0.40	10.0	12.0
						50	2.10	2.20	2.80	0.50	10.0	12.0
						60	2.60	2.70	3.40	0.65	10.0	12.0
XS2a	Permanently submerged in sea (or saline waters)	Severe	Permanently (or substantially) submerged: in the sea (without heavy wave action); in coastal saline estuaries & rivers; in any aggressive saline waters Concrete surfaces exposed to heavily polluted industrial waters; permanently or substantially submerged or permanantly wet saline conditions (Generally oxygen starved area approximately 1-1,5m below spring type level)	Coastal or other structures permanently submerged in seawater or other aggressive saline waters, including industrially polluted water; surfaces of structures in contact with marshy conditions	50	40	1.00	1.10	1.40	0.30	10.0	11.0
						60	1.40	1.60	2.00	0.40	10.0	11.0
						60	1.80	2.10	2.50	0.50	10.0	11.0
XS2b	XS2a + exposed to abrasion	Extreme	As above, but with heavy wave action; in any aggressive saline waters where abrasion occurs	As above + exposed to abrasion	60 (Mandatory)	60	1.45	1.70	2.00	0.40	10.0	11.0
XS3a	Tidal, splash & spray zones	Extreme	Sea or saline estuaries and rivers, but not permanently submerged; tidal zone; and in a spray or splash zone. surfaces exposed to aggressive saline waters, including heavily polluted industrial waters, without being permanently wet.	Coastal or other structures exposed to intertidal, splash, or spray zones, or exposed to other aggressive saline waters, including industrially polluted waters, without being permanently wet; members subject to burying by aeolian sands near coast	50	40	0.65	0.85	1.00	0.25	10.0	10.0
						50	1.10	1.35	1.45	0.35	10.0	10.0
						60	1.45	1.70	2.00	0.40	10.0	10.0
XS3b	XS3a + exposed to abrasion		As above, but with heavy wave action or where abrasion or erosion can occur	As above + exposed to abrasion	60 (Mandatory)	60	1.10	1.30	1.55	0.30	10.0	10.0

Notes:

1. Exposure Classes

i). Exposure classes are only best estimates at this stage and considerably more work is needed on this.
ii) The key to interpreting the exposure classes is that the steel should 'feel' the impact of the exposure. E.g. wetting and drying should really influence the concrete at the level of the steel, rather than being a fleeting surface wetting.
iii) Various bridge elements will experience the same exposure class in different ways. E.g. interior columns and deck undersides will generally remain dry, while deck edges, exposed abutments, and balustrades will experience the full climatic effects.

2. Cover:

i) Minimum cover for bridge structures is taken as 40 mm, i.e. civil engineering structures are contemplated.

ii) In-situ piles shall in general have cover not less than 75mm due to tolerance variation

iii) Pre-cast piles shall not be lesser than than 55mm
iv) Variable cover should be considered for bridge design:
- Cantilevers and balustrades
- Soffits and interior columns
- Pile caps and tops of piles

3. OPI

i) Values are based on UCT spreadsheets.

ii) Most values are based on a blended binder, not a pure OPC binder.

iii) UCT's spreadsheet tends not to differentiate between OPC and Slag mixes, but does show more conservative values for FA mixes. The values in the spreadsheet tend towards the FA mix values, since a great deal of concrete in South Africa, particularly the interior regions, contains FA.

iv) The justification for the above is that it is not possible to always know what binders will be used in construction concretes, and therefore a conservative approach is justified.

4. Chloride Conductivity

i) Values are based on UCT spreadsheets.
ii) In this case, allowance is made for the different binder types.
iii) Interpolation or extrapolation of the CC values taken from UCT spreadsheets for the different exposure classes

5. Sorptivity

i) Values are based on research undertaken at UCT/Wits.

ii) Final value to be used during construction to be based on laboratory mix design testing done for project i.e. value specific to location of project but within limits specified

7.3 SOUTH AFRICAN CHLORIDE INGRESS MODEL

The relevant sections of the South African Chloride Ingress Model are extracted for reference purposes. This consists of pages 9-14 on "designing for durability" from Research Monograph 1, published by the University of Cape Town (Mackechnie, 2001), which was used as the service life prediction model in Chapter 4.

DESIGNING FOR DURABILITY

In order to design a marine concrete structure for durability, the resistance of the cover concrete needs to be quantified. This monograph provide empirical means for assessing this resistance in terms of the rate of chloride ingress through concrete, and therefore the time to corrosion activation of the reinforcement. The empirical model used for predicting chloride levels in concrete was developed from data gathered from case studies of concrete structures, and marine exposure testing of a range of concrete specimens at several sites in the Cape Peninsula.[4] Details of concrete characterization, marine exposure and case studies of concrete structures are given in Appendix 1-3. Chloride contents are predicted using the solution of Fick's second law of diffusion given below:

$$C_x = C_s\left(1\text{-erf}\left(\frac{x}{2\sqrt{D_c t}}\right)\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where C_x is the chloride concentration at depth x and time t, C_s is the surface concentration, D_c is the diffusion coefficient and erf is the mathematical error function. The time-integrated nature of diffusion coefficients allows chloride contents to be determined from long-term parameters using equation 1 but some allowance needs to be made when predicting chloride levels using short-term D_c values. This can be achieved by introducing an effective D_c reduction factor into equation 1 that allows for reducing D_c values with time.[5]

The design technique consists of several steps that are given below.

1. Marine exposure categories

Accurate assessments of the severity of exposure are vital in order that materials of sufficient durability are specified for the exposure conditions. Table 1 provides a basic framework for classifying the severity of marine exposure, developed from guidelines given in BS 8110.[6]

Table 1: Classification of marine exposure categories

Marine exposure category	*Marine tidal and splash zones*	*Marine spray zone*
Extreme	Structure exposed directly to sea water with heavy wave action and/or abrasion	N/A
Very severe	Structure exposed directly to sea water under sheltered conditions with little wave action	Structure within 500m of shore exposed to heavy wave action and onshore wind
Severe	N/A	Structure located near shore (>500m) in an exposed marine location
Moderate	N/A	Structure in a sheltered location within 1km of shore or anywhere within 30km of coast

Whilst an attempt has been made to define exposure categories more rigorously, engineering judgement and experience are required to accurately quantify exposure conditions. The durability performance of surrounding infrastructure often provides useful guidance in this regard.

2. Service requirements

Factors relating to the service function and structural requirements need to be defined before detailed design can commence. Service functions

include the service life required for the structure, likely future maintenance and logistics of repairs. Structural considerations include minimum concrete grade, cover depth and crack width limitations, dimensional stability (i.e. shrinkage and creep potential) and long-term risk of cracking.

3. Selection of concrete materials

Two approaches may be used when selecting concrete materials for marine structures: laboratory-based investigations and desk-top studies.

(a) Laboratory-based investigations

The potential durability of concrete may be defined by early-age characterization tests that measure the resistance of cover concrete to ingress of fluids and ions causing deterioration (Appendix 1). Movement of fluid and ions through concrete occurs by three main transport mechanisms: absorption, diffusion and permeation. Chloride-induced corrosion is primarily affected by diffusion of chlorides through concrete and may be assessed using the chloride conductivity test developed at UCT.[7]

The chloride conductivity test uses an applied voltage to accelerate chloride movement such that virtually instantaneous readings are possible on preconditioned core specimens. The results may be used for comparative purposes or to predict long-term diffusion coefficients for marine concrete. Figure 2 shows a nomogram developed from synthesis of marine exposure tests and case studies of marine structures. The nomogram shows the relationship between 28 day chloride conductivity measurements and 50 year diffusion coefficients for different concrete types. Concrete containing 100% portland cement is denoted PC, 30% fly ash is denoted FA, 50% slag is denoted SL and 10% condensed silica fume is denoted SF.

It should be noted that the nomogram was developed for Western Cape materials and environmental conditions. The modified chloride conductivity value referred to in the nomogram allows for long-term effects such as chloride binding and continued cementing reactions.

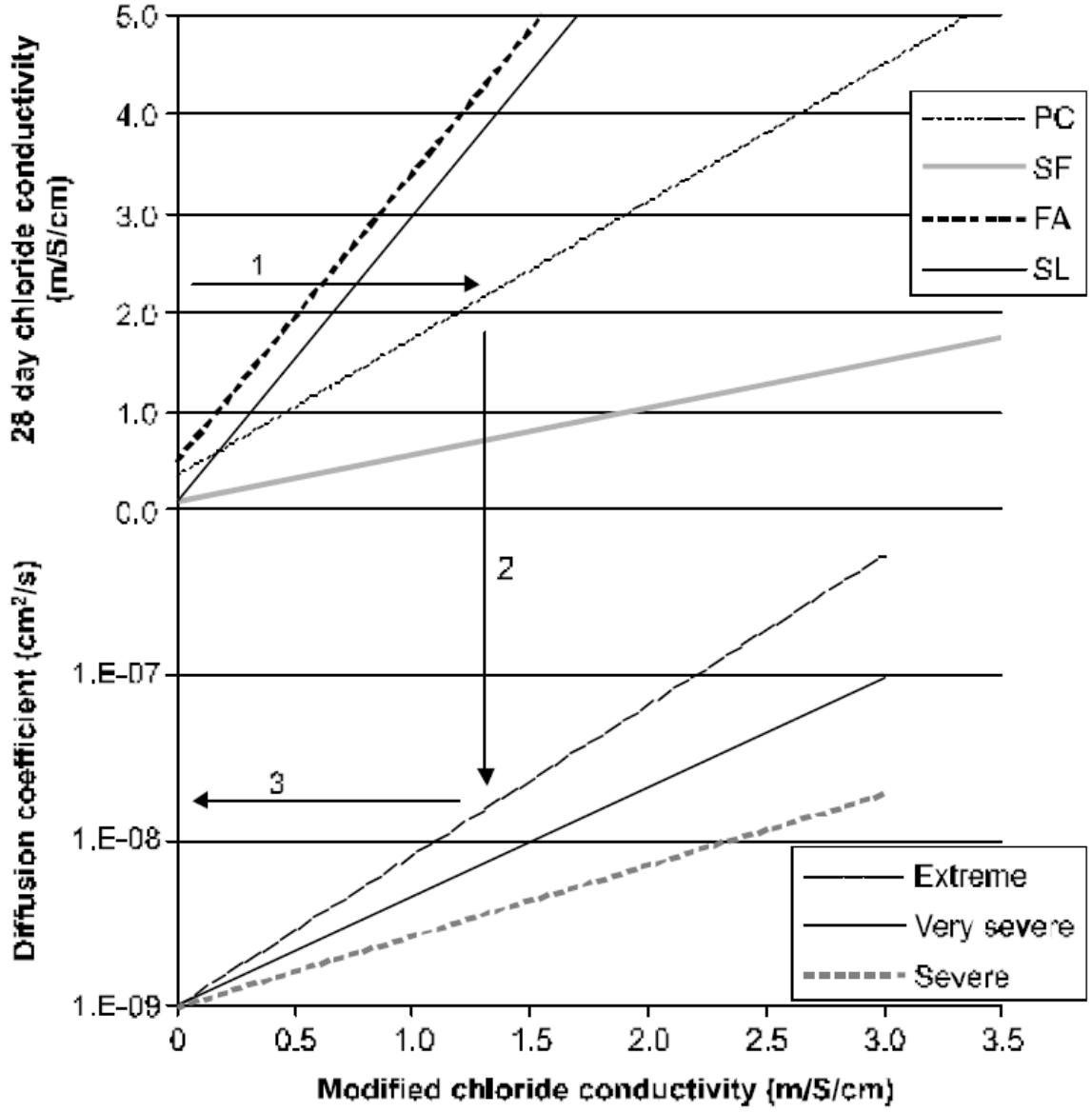


Figure 2: Predicting 50-year diffusion coefficients for marine concrete

(b) Desk-top study

Since most preliminary designs of concrete structures are done without laboratory trials, it is often necessary to assess materials in advance using approximate methods. An estimation of long-term diffusion coefficients can be made using general design charts summarized in Table 2. The data has been generated from Western Cape concrete having 3 days initial moist curing and standard compaction.

Table 2: Predicted diffusion coefficients (x10^{-8} cm^2/s)

Concrete type	*Grade (MPa)*	*Extreme exposure*			*Very severe exposure*		
		10 years	*25 years*	*50 years*	*10 years*	*25 years*	*50 years*
100% PC	30	13.8	10.6	8.65	4.58	3.51	2.87
	40	4.70	3.61	2.95	2.63	2.02	1.65
	50	3.14	2.40	1.97	1.88	1.44	1.18
	60	2.51	1.92	1.57	1.44	1.11	0.90
10% SF	30	6.02	4.79	4.03	4.01	3.19	2.68
	40	4.01	3.19	2.68	2.68	2.13	1.79
	50	2.68	2.13	1.79	1.67	1.33	1.12
	60	1.00	0.80	0.67	0.64	0.51	0.42
30% FA	30	1.74	0.93	0.58	1.07	0.57	0.36
	40	1.14	0.61	0.38	0.80	0.43	0.27
	50	0.90	0.48	0.30	0.67	0.36	0.22
	60	0.67	0.36	0.22	0.57	0.31	0.19
50% SL	30	1.34	0.72	0.45	0.87	0.47	0.29
	40	0.87	0.47	0.29	0.60	0.32	0.20
	50	0.67	0.36	0.22	0.50	0.27	0.17
	60	0.54	0.29	0.18	0.40	0.22	0.13

The desk-top study approach does not allow for local factors such as material quality and site practice. Some adjustment is recommended for exceptional conditions, particularly with regard to climatic conditions. A more detailed laboratory investigation (such as given in section 3a) is necessary to confirm information derived from the desk-top study.

4. Prediction of chloride levels in concrete

Before chloride levels are predicted using equation 1, the surface concentration must be determined. Data given in Table 3 is proposed from measurements of marine concrete structures and field exposure specimens under Western Cape conditions.

Table 3: Chloride surface concentrations (% by mass of binder)

Concrete type	*Tidal/splash zone*	*Spray zone*
100% PC	3.0 – 4.0	1.5 – 2.0
10% SF	2.5 – 3.0	1.3 – 1.5
30% FA	4.5 – 5.0	2.3 – 2.5
50% SL	5.0 – 6.0	2.5 – 3.0

The higher surface concentrations given for fly ash or slag concrete

are due to their superior chloride binding characteristics (which increase their capacity to hold chlorides) when compared with PC concrete. The lower surface concentration in the spray zone is due to equilibrium established between deposition of wind-borne chlorides and regular flushing of surface deposits by rainfall and mist. Concrete in arid conditions exposed to marine spray may establish higher surface concentrations that could eventually cause salt crystallization damage.

Once the diffusion coefficient and surface concentration have been determined, chloride levels may be predicted using equation 1. A graphical solution of the equation using a nomogram is shown in Figure 3. An example is shown in the nomogram where the chloride concentration at 60 mm is determined for a 50-year old concrete with a diffusion coefficient of 1.0 E-8 cm^2/s.

5. Estimation of time to corrosion activation

An estimate of the time to corrosion activation can be made once chloride levels at the reinforcement have been predicted. Activation of corrosion has been found to occur at chloride levels of 0.4 – 0.5% by mass of cement, while high corrosion rates generally occur at higher chloride levels.[8] It may therefore be prudent to define the maintenance-free design life as that period where chloride levels remain below the corrosion threshold level. Using this premise, typical times to corrosion activation for moderately cured concrete (3 days moist curing) with reinforcement cover of 60 mm are shown in Figure 4. The advantage of using slag, fly ash and silica fume concrete in preference to portland cement concrete is evident, particularly at higher grades.

7.4 CONDITION OF TEST SPECIMENS

Specimen No.	From Bridge	From Element	Test	Panel Type	Condition
1	Swakop Bridge	Deck	OPI	Missing	
2	TR2/2	Deck	OPI	Missing	
3	TR2/2	Deck	OPI	Missing	
4	TR2/2	Deck	OPI	Missing	
5	TR2/2	Deck	CC	deck	good
6	TR2/2	Deck	CC	deck	good
7	Swakop Bridge	Deck	CC	deck	good
8	TR2/2	Deck	CC	deck	One cracked edge
9	TR2/2	Deck	CC	Missing	
10	TR2/2	Deck	CC	deck	good
11	TR2/2	Deck	CC	deck	good
12	TR2/2	Deck	CC	deck	good
13	TR2/2	Deck	CC	deck	good
14	TR2/2	Deck	CC	deck	slightly chipped
15	TR2/2	Deck	CC	deck	good
16	Swakop Bridge	Deck	CC	deck	good
17	TR2/2	Deck	OPI	deck	slightly chipped
18	TR2/2	Deck	OPI	deck	good
19	Swakop Bridge	Deck	OPI	deck	minor blowholes
20	TR2/2	Deck	OPI	deck	good
21	TR2/2	Deck	CC	deck	slightly chipped
22	TR2/2	Deck	CC	deck	One cracked edge
23	TR2/2	Deck	CC	deck	cracked
24	Swakop Bridge	Deck	CC	deck	good
25	TR2/2	Deck	OPI	deck	good
26	Swakop Bridge	Deck	OPI	deck	good
27	TR2/2	Deck	OPI	deck	good
28	Swakop Bridge	Deck	OPI	deck	good
29	TR2/2	Deck	CC	deck	good
30	TR2/2	Deck	CC	deck	good
31	Swakop Bridge	Deck	CC	deck	cracked
32	Swakop Bridge	Deck	CC	deck	cracked
33	Swakop Bridge	Deck	OPI	deck	minor blowholes
34	TR2/2	Deck	OPI	deck	cracked
35	TR2/2	Deck	OPI	deck	One cracked edge
36	Swakop Bridge	Deck	OPI	deck	One cracked edge
37	TR2/2	Deck	CC	deck	slightly chipped
38	Swakop Bridge	Deck	CC	deck	minor blowholes
39	Swakop Bridge	Deck	CC	deck	slightly chipped
40	Swakop Bridge	Pier 2B	CC	Pier	good
41	TR2/2	Deck	OPI	deck	good

42	TR2/2	Deck	OPI	deck	good
43	Swakop Bridge	Deck	OPI	deck	One cracked edge
44	TR2/2	Deck	OPI	deck	slightly chipped
45	TR2/2	Deck	CC	deck	slightly chipped
46	TR2/2	Deck	CC	deck	good
47	TR2/2	Deck	CC	deck	good
48	Swakop Bridge	Deck	CC	deck	slightly chipped
49	Swakop Bridge	Pier 3B	OPI	Pier	good
50	Swakop Bridge	Pier 2B	OPI	Pier	slightly chipped
51	Swakop Bridge	Pier 2B	OPI	Pier	good
52	Swakop Bridge	Pier 3B	OPI	Pier	good
53	Swakop Bridge	Pier 3B	CC	Pier	good
54	Swakop Bridge	Pier 2B	CC	Pier	blowholes
55	Swakop Bridge	Pier 3B	CC	Pier	good
56	Swakop Bridge	Pier 2B	CC	Pier	good
57	Swakop Bridge	Pier 2B	OPI	Pier	good
58	Swakop Bridge	Abutment 1A	OPI	Pier	One cracked edge
59	Swakop Bridge	Abutment 1A	OPI	Pier	good
60	Swakop Bridge	Pier 2B	OPI	Pier	good
61	Swakop Bridge	Abutment 1A	CC	Pier	good
62	Swakop Bridge	Pier 2B	CC	Pier	One cracked edge
63	Swakop Bridge	Pier 3B	CC	Pier	slightly chipped
64	Swakop Bridge	Pier 3B	CC	Pier	good
65	Swakop Bridge	Pier 2B	OPI	Pier	good
66	Swakop Bridge	Pier 3B	OPI	deck	slightly chipped
67	Swakop Bridge	Pier 3B	OPI	deck	good
68	Swakop Bridge	Abutment 1A	OPI	Pier	good
69	Swakop Bridge	Pier 2B	CC	deck	One cracked edge
70	Swakop Bridge	Abutment 1A	CC	Pier	cracked
71	Swakop Bridge	Pier 3B	CC	Pier	blowholes
72	Swakop Bridge	Pier 2B	CC	deck	good
73	Swakop Bridge	Pier 2B	OPI	Pier	slightly chipped
74	Swakop Bridge	Pier 2B	OPI	deck	good
75	Swakop Bridge	Pier 2B	OPI	Pier	blowholes
76	Swakop Bridge	Pier 2B	OPI	Pier	good
77	Swakop Bridge	Pier 2B	CC	Pier	slightly chipped
78	Swakop Bridge	Pier 3B	CC	Pier	slightly chipped
79	Swakop Bridge	Pier 2B	CC	Pier	blowholes
80	Swakop Bridge	Abutment 1A	CC	Pier	slightly chipped
81	Swakop Bridge	Pier 3B	OPI	deck	blowholes
82	Swakop Bridge	Pier 3B	OPI	deck	good
83	Swakop Bridge	Pier 2B	OPI	Pier	One cracked edge
84	Swakop Bridge	Pier 2B	OPI	Pier	slightly chipped
85	Swakop Bridge	Pier 2B	CC	deck	slightly chipped
86	Swakop Bridge	Pier 2B	CC	Pier	One cracked edge

87	Swakop Bridge	Pier 3B	CC	Pier	slightly chipped
88	Swakop Bridge	Pier 3B	CC	Pier	One cracked edge
89	Swakop Bridge	Abutment 1A	OPI	Pier	cracked
90	Swakop Bridge	Pier 2B	OPI	Pier	good
91	Swakop Bridge	Abutment 1A	OPI	Pier	slightly chipped
92	Swakop Bridge	Pier 2B	OPI	Pier	slightly chipped
93	Swakop Bridge	Pier 2B	CC	Pier	good
94	Swakop Bridge	Deck	CC	deck	slightly chipped
95	Swakop Bridge	Pier 2B	CC	deck	One cracked edge
96	Swakop Bridge	Pier 2B	CC	Pier	good
97	Swakop Bridge	Pier 3B	OPI	Pier	slightly chipped
98	Swakop Bridge	Pier 3B	OPI	deck	blowholes
99	Swakop Bridge	Pier 2B	OPI	Pier	blowholes
100	Swakop Bridge	Pier 2B	OPI	deck	blowholes
101	Swakop Bridge	Pier 2B	CC	Pier	minor blowholes
102	Swakop Bridge	Pier 2B	CC	Pier	good
103	Swakop Bridge	Pier 2B	CC	Pier	minor blowholes
104	Swakop Bridge	Pier 2B	CC	deck	good
105	Swakop Bridge	Pier 2B	OPI	deck	One cracked edge
106	Swakop Bridge	Pier 2B	OPI	deck	blowholes
107	Swakop Bridge	Pier 2B	OPI	deck	minor blowholes
108	Swakop Bridge	Pier 2B	OPI	deck	minor blowholes
109	Swakop Bridge	Pier 2B	CC	Pier	slightly chipped
110	Swakop Bridge	Pier 3B	CC	Pier	good
111	Swakop Bridge	Pier 2B	CC	Pier	good
112	Swakop Bridge	Pier 2B	CC	discarded	discarded
113	Swakop Bridge	Pier 2B	OPI	Pier	One cracked edge
114			OPI	discarded	Discarded
115	Swakop Bridge	Pier 3B	OPI	Pier	good
116	Swakop Bridge	Pier 3B	OPI	Pier	slightly chipped
117	Swakop Bridge	Pier 2B	CC	Pier	minor blowholes
118	Swakop Bridge	Pier 2B	CC	Pier	One cracked edge
119	Swakop Bridge	Pier 2B	CC	deck	good
120	Swakop Bridge	Pier 2B	CC	Pier	blowholes
121	Swakop Bridge	Pier 2B	CC	deck	blowholes
122	Swakop Bridge	Pier 3B	CC	Pier	slightly chipped
123	Swakop Bridge	Pier 2B	CC	Pier	slightly chipped
124	Swakop Bridge	Pier 3B	CC	Pier	good
125	Swakop Bridge	Pier 2B	OPI	deck	minor blowholes
126	Swakop Bridge	Pier 3B	OPI	deck	cracked
127	Swakop Bridge	Pier 2B	OPI	Pier	One cracked edge
128	Swakop Bridge	Pier 2B	OPI	Pier	minor blowholes
129	Test Cubes		OPI	Test Cube	good
130	Test Cubes		OPI	Test Cube	good
131	Test Cubes		OPI	Test Cube	good

132	Test Cubes		OPI	Test Cube	good
133	TR2/2	Abutment 3	OPI	Pier	good
134	TR2/2	Abutment 3	OPI	Pier	good
135	TR2/2	Abutment 3	OPI	Pier	good
136	TR2/2	Abutment 3	OPI	Pier	good
137	TR2/2	Abutment 1	OPI	Pier	good
138	TR2/2	Abutment 1	OPI	Pier	good
139	TR2/2	Abutment 1	OPI	Pier	good
140	TR2/2	Abutment 1	OPI	Pier	good
141	MR52 Bridge	Deck	OPI	deck	good
142	MR52 Bridge	Deck	OPI	deck	good
143	MR52 Bridge	Deck	OPI	deck	good
144	MR52 Bridge	Deck	OPI	deck	good
145	MR52 Bridge	Deck	OPI	deck	good
146	MR52 Bridge	Deck	OPI	deck	good
147	MR52 Bridge	Deck	OPI	deck	good
148	MR52 Bridge	Deck	OPI	deck	good
149	Test Cubes		CC	discarded	
150	Test Cubes		CC	discarded	
151	Test Cubes		CC	discarded	
152	Test Cubes		CC	discarded	
153	TR2/2	Abutment 3	CC	discarded	
154	TR2/2	Abutment 3	CC	discarded	
155	TR2/2	Abutment 3	CC	discarded	
156	TR2/2	Abutment 3	CC	discarded	
157	TR2/2	Abutment 1	CC	discarded	
158	TR2/2	Abutment 1	CC	discarded	
159	TR2/2	Abutment 1	CC	discarded	
160	TR2/2	Abutment 1	CC	discarded	
161	MR52 Bridge	Deck	CC	deck	good
162	MR52 Bridge	Deck	CC	deck	good
163	MR52 Bridge	Deck	CC	deck	good
164	MR52 Bridge	Deck	CC	deck	good
165	MR52 Bridge	Deck	CC	deck	good
166	MR52 Bridge	Deck	CC	deck	good
167	MR52 Bridge	Deck	CC	deck	good
168	MR52 Bridge	Deck	CC	deck	good

7.5 COMPLETE TEST RESULTS

Cover Test Results

A more detailed summary of cover test results is given in this appendix. The summary is given per test area ($1m^2$ each).

MR52 Interchange Bridge:

Element	Test Area Number	No. readings	Average cover	Standard Deviation	COV
Abutment 1	1	20	43.6	3.1	7.1%
	2	20	43.6	3.6	8.3%
	3	20	42.7	4.8	11.2%
Pier 2	1	25	49.2	2.8	5.7%
	2	25	49.5	1.9	3.8%
	3	25	49.9	2.8	5.6%
	4	25	48.0	2.2	4.6%
Pier 3	1	25	48.5	3.2	6.6%
	2	25	46.6	2.8	6.0%
	3	25	46.7	2.3	4.9%
	4	25	45.3	4.4	9.7%
Pier 4	1	25	46.2	2.7	5.8%
	2	25	49.2	2.2	4.5%
	3	25	45.8	3.7	8.1%
	4	25	50.3	2.5	5.0%
Abutment 5	1	20	40.3	2.7	6.7%
	2	20	40.5	2.4	5.9%
	3	20	43.0	4.5	10.5%

Swakop River Bridge A:

Element	Test Area Number	No. readings	Average cover	Standard Deviation	COV
Abutment 1	1	15	62.4	7.5	12.0%
	2	15	50.9	4.4	8.6%
	3	15	50.6	3.6	7.1%
Pier 2	1	15	68.2	4.1	6.0%
	2	15	58.1	1.6	2.8%
	3	15	56.5	2.6	4.6%
	4	15	50.1	5.4	10.8%
Pier 3	1	15	52.9	5.7	10.8%
	2	15	60.4	7.2	11.9%
	3	15	60.5	4.4	7.3%
	4	15	65.6	3.2	4.9%
Pier 4	1	15	54.3	3.5	6.4%
	2	15	59.4	1.9	3.2%
	3	15	57.2	8.8	15.4%
	4	15	61.9	3.3	5.3%
Abutment 5	1	15	51.2	2.9	5.7%
	2	15	48.9	8.4	17.2%
	3	15	59.5	8.2	13.8%

Swakop River Bridge B;

Element	Test Area Number	No. readings	Average cover	Standard Deviation	COV
Abutment 1	1	15	69.0	5	7.2%
	2	15	47.6	1.6	3.4%
	3	15	48.6	7.7	15.8%
Pier 2	1	15	61.3	7.8	12.7%
	2	15	59.7	1.2	2.0%
	3	15	49.6	7	14.1%

	4	15	58.2	3.4	5.8%
Pier 3	1	15	57.3	1.6	2.8%
	2	15	58.9	3.3	5.6%
	3	15	49.9	6.7	13.4%
	4	15	48.4	5.9	12.2%
Pier 4	1	15	55.6	4.5	8.1%
	2	15	59.2	1.7	2.9%
	3	15	57.3	9.9	17.3%
	4	15	56.7	6.2	10.9%
Abutment 5	1	15	44.5	3.6	8.1%
	2	15	46.7	4.7	10.1%
	3	15	60.5	6.8	11.2%

TR2/2 Interchange Bridge

Element	Test Area Number	No. readings	Average cover	Standard Deviation	COV
Abutment 1	1	15	49.9	3.8	7.6%
	2	15	51.2	2.9	5.7%
	3	15	54.6	1.2	2.2%
	4	15	51.2	2.2	4.3%
Pier 2	1	15	39.2	2.3	5.9%
	2	15	39.0	3.5	9.0%
	3	15	43.3	2.6	6.0%
	4	15	41.7	4.6	11.0%
	5	15	43.4	2.3	5.3%
	6	15	43.1	3.5	8.1%
Abutment 3	1	15	60.4	3.1	5.1%
	2	15	55.7	4.1	7.4%
	3	15	56.1	6.3	11.2%
	4	15	62.5	5.5	8.8%

Durability Index Test Results

Oxygen Permeability Index (OPI)

Test Number	Age at Testing	Element Type	OPI 1	OPI 2	OPI 3	OPI 4	OPI Ave	RSQ 1	RSQ 2	RSQ 3	RSQ 4	st.dev	COV
1	67	Horizontal	10.09	10.25	10.16	-	**10.17**	0.999	0.999	0.999	-	0.065	0.8%
2	88	Horizontal	9.69	9.78	9.80	9.56	**9.71**	0.998	0.999	0.999	0.998	0.094	1.1%
3	132	Horizontal	9.85	10.22	10.30	10.55	**10.23**	0.999	0.999	0.999	0.997	0.250	2.8%
4	132	Horizontal	9.97	10.00	10.07	10.21	**10.06**	0.999	0.999	0.999	0.999	0.092	1.1%
5	52	Horizontal	10.20	9.81	10.60	9.02	**9.91**	0.999	0.999	0.999	0.995	0.583	6.8%
6	66	Horizontal	10.38	10.24	10.22	-	**10.28**	0.999	0.997	0.999	-	0.071	0.8%
7	67	Horizontal	10.03	9.78	10.10	-	**9.97**	0.999	0.999	0.999	-	0.137	1.7%
8	60	Horizontal	9.97	10.09	10.03	9.89	**10.00**	0.999	0.999	0.999	0.999	0.074	0.9%
9	91	Vertical	10.50	10.44	10.34	10.03	**10.33**	0.999	1.000	0.999	0.999	0.181	2.0%
10	60	Vertical	9.99	10.13	9.93	10.22	**10.07**	0.999	0.999	0.999	0.999	0.114	1.3%
11	64	Vertical	9.66	10.06	10.00	9.82	**9.89**	0.998	0.999	0.999	0.999	0.157	1.8%
12	63	Vertical	9.95	10.03	10.08	10.46	**10.13**	0.998	0.999	0.999	0.999	0.196	2.2%
13	62	Vertical	9.72	10.25	10.10	10.00	**10.02**	0.999	0.999	0.999	0.999	0.193	2.2%
14	61	Vertical	9.85	9.39	9.31	9.89	**9.61**	0.999	0.997	0.996	0.999	0.261	3.1%
15	59	Vertical	10.04	10.33	10.03	9.94	**10.09**	0.999	0.999	0.999	0.999	0.146	1.7%
16	63	Vertical	10.87	10.66	10.84	10.43	**10.70**	0.995	0.999	0.992	0.998	0.175	1.9%
17	69	Vertical	10.56	10.55	10.55	9.58	**10.31**	0.999	0.999	0.999	0.999	0.421	4.7%
18	76	Test Cube	10.96	10.79	10.40	10.58	**10.68**	0.995	0.997	0.989	0.999	0.211	2.3%

Water Sorptivity Index (WSI) and Porosity:

Test Number	Water Sorptivity											Porosity						
	WS1	WS2	WS3	WS4	WS Ave	RSQ 1	RSQ 2	RSQ 3	RSQ 4	st.dev	COV	P1	P2	P3	P4	P ave	st.dev	COV
1		6.09	4.52	7.49	**6.03**		0.9961	0.9942	0.9974	1.2132	24.6%		7.3%	9.4%	9.1%	**8.6%**	0.0093	13.2%
2	5.6	7.13	8.3	9.1	**7.53**	0.9970	0.9977	0.9925	0.9983	1.3174	20.2%	8.9%	9.4%	8.6%	11.1%	**9.5%**	0.0097	11.8%
3	6.82	6.85	5.71	4.63	**6.00**	0.9615	0.9356	0.9640	0.9226	0.9160	17.6%	11.8%	10.1%	9.7%	10.9%	**10.6%**	0.0080	8.7%
4	6.19	8.22	7.13	6.37	**6.98**	0.9888	0.9944	0.9864	0.9856	0.7994	13.2%	11.7%	13.8%	11.5%	11.7%	**12.2%**	0.0094	8.9%
5	6.55	7.17	5.05	8.16	**6.73**	0.9995	0.9968	0.9972	0.9992	1.1284	19.4%	8.7%	10.2%	8.8%	12.1%	**10.0%**	0.0138	16.0%
6	5.36	5.63	5.62		**5.54**	0.9913	0.9952	0.9783		0.1250	2.8%	9.1%	10.3%	10.1%		**9.8%**	0.0052	6.5%
7	5.82	8.37	6.9		**7.03**	0.9967	0.9944	0.9895		1.0451	18.2%	9.3%	9.5%	10.9%		**9.9%**	0.0071	8.8%
8	6.36	5.5	9.43	7.72	**7.25**	0.9972	0.9987	0.9981	0.9964	1.4856	23.7%	12.5%	11.2%	11.4%	11.7%	**11.7%**	0.0049	4.9%
9	6.11	7.79	7.21	6.73	**6.96**	0.9706	0.9821	0.9772	0.9933	0.6178	10.2%	6.7%	6.8%	7.4%	8.2%	**7.3%**	0.0060	9.5%
10	7.88	5.47	6.38	6.35	**6.52**	0.9984	0.9923	0.9883	0.9773	0.8661	15.3%	8.3%	10.0%	10.5%	8.9%	**9.4%**	0.0087	10.7%
11	10.07	5.59	6.07	6.8	**7.13**	0.9948	0.9945	0.9879	0.9901	1.7498	28.3%	10.0%	11.1%	10.8%	9.8%	**10.4%**	0.0054	6.0%
12	6.09	5.56	5.49	6.25	**5.85**	0.9892	0.9946	0.9970	0.9792	0.3284	6.5%	9.2%	11.5%	10.7%	8.1%	**9.9%**	0.0132	15.4%
13	8.24	7.25	4.82	6.15	**6.62**	0.9992	0.9954	0.9912	0.9964	1.2730	22.2%	10.9%	8.8%	11.4%	12.0%	**10.8%**	0.0120	12.9%
14	6.54	6.62	5.23	5.99	**6.10**	0.9941	0.9974	0.9979	0.9936	0.5552	10.5%	10.3%	12.0%	11.6%	12.1%	**11.5%**	0.0072	7.2%
15	6.52	5.74	5.55	6.38	**6.05**	0.9962	0.9848	0.9931	0.9878	0.4111	7.8%	10.0%	9.6%	11.0%	11.0%	**10.4%**	0.0062	6.8%
16	9.2	5.59	6.65	6.49	**6.98**	0.9486	0.9479	0.9154	0.9006	1.3425	22.2%	7.6%	9.3%	8.3%	7.9%	**8.3%**	0.0064	9.0%
17	9.62	7.21	5.89	7.62	**7.59**	0.9787	0.9650	0.9664	0.9868	1.3375	20.4%	7.8%	7.5%	7.2%	8.6%	**7.8%**	0.0052	7.7%
18	4.8	3.86	6.28	4.85	**4.95**	0.9903	0.9936	0.9927	0.9816	0.8645	20.2%	6.6%	7.3%	7.2%	7.7%	**7.2%**	0.0039	6.3%

Chloride Conductivity Index (CCI):

Test	Age at Testing	Element Type	CC1	CC2	CC3	CC4	CC Ave	St. dev	COV
1	90	deck	0.46	0.46	0.48	0.61	**0.50**	**0.063**	14.4%
2	90	deck	0.95	0.65	0.62	0.97	**0.80**	**0.163**	23.6%
3	89	deck	0.48	0.48	0.53	0.44	**0.48**	**0.032**	7.6%
4	90	deck	0.58	0.56	0.64		**0.59**	**0.034**	7.0%
5	132	deck	0.52	0.48	0.57	0.59	**0.54**	**0.043**	9.2%
6	69	deck	0.77	0.83	0.81	0.69	**0.78**	**0.054**	8.0%
7	66	deck	1.24	1.63	1.35		**1.41**	**0.164**	14.3%
8	59	deck	0.65	0.68	0.78	0.80	**0.73**	**0.064**	10.1%
9	58	deck	0.81	0.83	0.83	0.85	**0.83**	**0.014**	2.0%
10	91	Pier	0.55	0.75	0.61	0.65	**0.64**	**0.073**	13.1%
11	50	Pier	1.26	1.23	1.12		**1.20**	**0.060**	6.1%
12	60	Pier	0.98	1.05	1.14	1.21	**1.10**	**0.087**	9.2%
13	59	Pier	0.90	0.82	0.82		**0.85**	**0.038**	5.5%
14	61	Pier	0.53	0.58	0.59	0.60	**0.58**	**0.027**	5.4%
15	64	Pier	0.84	0.88	0.89	0.90	**0.88**	**0.023**	3.0%
16	62	Pier	0.93	1.05	1.02	1.04	**1.01**	**0.047**	5.4%
17	58	Pier	1.00	1.05	1.10	1.14	**1.07**	**0.053**	5.7%
18	132	Test Cube	0.32	0.34	0.39	0.36	**0.35**	**0.026**	8.5%