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ENGINEERING

An Analysis of the Impact of Chloride-Induced Corrosion on Reinforced Concrete Structures in the Port of Cape Town



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Abstract

Corrosion of steel reinforcement is a main deterioration mechanism for reinforced concrete structures in the marine environment, where chloride ingress causes severe and widespread damage. These structures make up a substantial proportion of infrastructure globally and are costly to construct and require constant maintenance throughout their service life. For infrastructure owners such as the Transnet National Ports Authority, whose infrastructure is mostly civil in nature and is exposed to severe marine exposure conditions, an improved understanding of each port's environment can lead to the design of more durable structures, or the improved maintenance of existing structures, both leading to more effective investment decisions. The aim of this research is therefore to understand the relationship between marine exposure conditions, cover depths, age, and deterioration due to chloride-induced corrosion in the port of Cape Town through the investigation of select structures.

As-built data collection, visual condition assessments, and cover measurements were performed on various components of seventeen reinforced concrete structures in the port of Cape Town. These structures varied in terms of type, age, distance from the sea and marine exposure. The condition assessments focused on deterioration related to corrosion namely rust staining and spalling, and damage ratings were ascribed based on the classification scheme provided in the RILEM technical committee 104-DDC 1994. Further, the predominant wind direction affecting the structural component (either South Easterly or North Westerly) and the marine exposure class were noted. These assessments were supplemented by cover readings on the structural components, with a focus on average and minimum cover depth per component.

Findings from the visual condition assessments confirmed significant reinforcement corrosion damage on twelve of the seventeen structures. This would imply that these structures are in the propagation phase of the two-phase corrosion model, or at the end of the maintenance free period in service life models. In addition, the average cover depth was found to be above 50 mm in most of the structural components (27 out of 38), however, the minimum cover was found to be below 40 mm in 55% of the structural components and below 30mm in 26% of the components.

The distance from the sea was found to significantly affect the damage ratings of structures, particularly for those further than 100 m from the sea, where ratings showed minimal damage. Further, the predominant wind direction was found to have a significant influence, damage ratings being higher for structures exposed to South Easterly winds compared to those exposed to North Westerly winds.

Minimum cover, or localised low cover depth, and average cover were found to have no significant influence on the degree of deterioration as no trend could be identified. In addition, deterioration compared to the age of structures, particularly for structures in the age ranges of 40-50 and 50-60 years showed an increase in damage ratings with an increase in age. Anomalies in the data can be attributed to the limitation of performing single parameter analysis since it is likely that multiple parameters influence the deterioration ratings.

The findings from this study can be used to inform the design and maintenance of structures in severe marine environments while quality controls can be put in place during construction to ensure the design cover is met and that localised low cover areas are prevented as much as possible. In addition, the findings from the visual condition assessment can be used to inform repair strategies for damaged structural components. The incorporation of cover meter readings in condition assessments, as well as other tests such as chloride concentration tests, can be used to inform preventative repair methodologies for existing structures at a high risk of corrosion damage, but with little to no visual damage.

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Table of contents

Declaration	ii
Abstract	iii
Acknowledgements	v
Table of contents	vi
List of tables	viii
List of figures	ix
Acronyms and abbreviations	x
1. Introduction	1-1
1.1 Background to the study	1-1
1.2 Research motivation and significance	1-2
1.3 Research objectives and key research questions	1-2
1.4 Scope and limitations	1-3
1.5 Plan of development	1-3
2. Literature review	2-4
2.1 Introduction	2-4
2.2 Concrete durability in the marine environment	2-4
2.2.1 Design phase	2-5
2.2.2 Construction phase	2-6
2.2.3 Service life	2-7
2.3 Chloride-induced corrosion	2-8
2.3.1 Corrosion initiation and propagation	2-8
2.3.2 Mechanisms of corrosion	2-9
2.3.3 Chloride binding	2-9
2.3.4 Chloride threshold	2-10
2.4 Select factors affecting corrosion rate	2-10
2.4.1 Cover depth	2-11
2.4.2 Concrete quality	2-12
2.4.3 Cracking	2-12
2.5 Marine exposure	2-12
2.5.1 Chloride-ion transport processes	2-13
2.5.2 Combined transport	2-14
2.6 Chloride ingress prediction models	2-15
2.7 Condition assessment	2-17
2.7.1 Visual assessment	2-17
2.7.2 Non-destructive testing	2-18

3.	Condition assessment and data acquisition	3-19
3.1	Study area	3-21
3.1.1	Cape Peninsula climate	3-21
3.1.2	Wind speed and direction	3-22
3.1.3	Effect of weather on structures	3-22
3.2	Location of structures assessed	3-22
3.3	Observed defects	3-24
3.3.1	Effects of reinforcement corrosion	3-24
3.3.2	Pop-outs	3-24
3.3.3	Spalling	3-25
3.4	Cover depth measurements	3-26
3.5	Data analysis	3-29
3.5.1	Effects of proximity from the sea	3-32
3.5.2	Effects of predominant wind direction	3-32
3.5.3	Effects of minimum cover	3-34
3.5.4	Effects of average cover	3-35
3.5.5	Effects of the age of the structure	3-36
4.	Summary, conclusions and recommendations	4-38
4.1	Summary and conclusions	4-38
4.2	Recommendations	4-39
	References	R
	Appendices	A
	Appendix A: Visual condition assessments	A1
	Appendix B: Cover depth readings	A2

List of tables

Table 1	Factors affecting chloride ingress (Otieno, 2014)	2-10
Table 2	Summary of Chloride-ion transport processes (Arito, 2016)	2-14
Table 3	Chloride ingress prediction models (Arito, 2016)	2-16
Table 4	Damage classification of defects (Cement & Concrete SA, 2021)	2-18
Table 5	Summary of assessed structures	3-20
Table 6	Summary of cover measurements	3-27
Table 7	Summary of cover measurements and damage ratings	3-30
Table 8	Predominant wind direction and damage ratings	3-33
Table 9	Average damage rating per minimum cover range	3-34
Table 10	Average damage rating per average cover range	3-35
Table 11	Average damage rating per age of structure range	3-36

List of figures

Figure 1	Lack of correlation between cube strength tests from specimens in the laboratory versus oxygen permeability tests measured on site (Alexander & Beushausen, 2019)	2-5
Figure 2	Two-phase corrosion model (Alexander & Beushausen, 2019)	2-8
Figure 3	Corrosion in reinforced concrete (Beushausen, 2022)	2-9
Figure 4	Influence of cover depth and binder type on corrosion rate on reinforced concrete structures in the marine environment for (a) uncracked concrete and (b) 0.4mm cracked concrete (Otieno, 2014)	2-11
Figure 5	General durability processes of concrete in the marine environment (Li et al., 2022)	2-13
Figure 6	Marine exposure classes and transport mechanisms (Beushausen et al., 2021)	2-15
Figure 7	Location of Port of Cape Town in Western Cape of South Africa (map extracted from Google Earth Pro)	3-21
Figure 8	Wind roses at Cape Town Yacht Club for measured SAWS data (Daniels et al., 2022)	3-22
Figure 9	Location of structures assessed in the Port of Cape Town (map extracted from Google Earth Pro)	3-23
Figure 10	Effects of corrosion at TNPA House building (left) and Eastern Mole Approach Bridge (right)	3-24
Figure 11	Pop outs at the Quay 700 Prefabricated Substation (Left) and the Eastern Mole Pumproom (right)	3-25
Figure 12	Spalling at the Unitie building exterior wall (left) and the B-Berth Shed (right)	3-25
Figure 13	Proceq Profoscope used for cover measurements	3-26
Figure 14	Proximity from the sea compared to damage ratings (condition), where damage ratings are not visible, they are overlaid (25 of 38 components have the same damage ratings per parameter – pop-outs, spalling, and effects of corrosion)	3-32
Figure 15	Predominant wind direction and impact on damage ratings	3-33
Figure 16	Damage rating per minimum cover range	3-34
Figure 17	Damage rating per average cover range	3-35
Figure 18	Damage rating per age of structure range	3-36

Acronyms and abbreviations

DBV	German Concrete and Construction Association
GGBS	Ground Granulated Blast Furnace Slag
GGCS	Ground Granulated Corex Slag
GPR	Ground Penetrating Radar
NDT	Non-Destructive Testing
OPI	Oxygen Permeability Index
RC	Reinforced Concrete
RILEM	The International Union of Laboratories and Experts in Construction Materials, Systems and Structures
SAWS	South African Weather Service
SCM	Supplementary Cementitious Materials
SLM	Service Life Modelling
TNPA	Transnet National Ports Authority

1. Introduction

1.1 Background to the study

The deterioration of reinforced concrete structures may be due to chemical, physical, or mechanical factors (Alexander et al., 2017). The chemical and physical factors are largely influenced by the environment to which the structures are exposed and therefore, environmental exposure has been shown to play an important role with some exposure conditions more aggressive than others. In addition to the environment, structures may perform inadequately due to a poor design, poor construction, or poor material selection or a combination of these factors (Alao, 2015).

Although various deterioration mechanisms exist, corrosion of steel reinforcement is considered the most significant mechanism, particularly in temperate, marine, and industrial environments (Otieno, Beushausen & Alexander, 2016). Reinforcement corrosion may be caused by either carbon dioxide ingress (carbonation) or chloride ingress with the latter causing more severe and widespread damage, potentially resulting in failure or a reduced service life (Otieno, Beushausen & Alexander, 2016).

The marine environment is particularly aggressive for reinforced concrete structures. The exposure of structures in this environment is categorised into those which are 1) submerged, 2) those in the tidal zone, 3) those in the splash and spray zone, and 4) those in the airborne zone (Beushausen et al., 2021). These zones result in varying chloride transport mechanisms and rates of deterioration.

The durability design of structures has become imperative in recent years, factoring in environmental exposure, concrete properties, and construction methods (Beushausen et al., 2021). These methods have improved over time as research resulted in a better understanding of the deterioration processes and mechanisms. While this can be incorporated into new structures, a substantial number of structures already exist. A key consideration for these structures is how they have already been affected by environmental exposure and their concrete properties, and how new methodologies can impact their analysis, to optimally specify and perform maintenance.

Condition assessment is a tool used to assess the condition of existing structures. These assessments include the collection of all relevant existing data on the structure, performing of visual assessments, and if necessary, designing a strategy for in-situ and laboratory testing (Cement & Concrete SA, 2021). These assessments identify deterioration mechanisms and their extents and inform maintenance strategies and refurbishment plans.

To support efficient maintenance of structures exposed to marine conditions in the port of Cape Town, it is deemed useful to perform condition assessments on select reinforced concrete structures with varying marine exposures, and to identify the deterioration mechanisms due to reinforcement corrosion and measure their extent. This information, together with any as-built

concrete and structural data, can be used to understand the relationship between exposure and deterioration, and assist in design and maintenance planning of marine structures in the port of Cape Town.

1.2 Research motivation and significance

Reinforced concrete structures make up a substantial proportion of infrastructure globally, and this is no different in South Africa. This infrastructure is costly to construct and requires constant maintenance throughout its service life, depending largely on the prevailing environmental exposure. Corrosion of steel reinforcement is a main deterioration mechanism, especially in coastal environments, where chloride-induced corrosion is chief in causing severe and widespread damage (Otieno, Beushausen & Alexander, 2016). The South African coastline consists of many of these examples with varying severity.

The Transnet National Ports Authority (TNPA) is a state-owned entity which owns and manages the eight commercial ports across South Africa with a total asset base of R97 billion (Transnet National Ports Authority, 2022), most of which includes civil assets. Assuming an industry norm 2% of replacement cost per annum for maintenance of these assets, the annual maintenance requirement would be R1.9 billion (Thoresen, 2003). This value assumes the infrastructure has been well maintained from inception and therefore actual financial needs may be higher due to any maintenance backlog. This is a significant financial commitment and is indicative of infrastructure organisations of this size. Further, state-owned entities play a role in stimulating the economy and therefore, the more efficient they can be at performing maintenance, the more value they can derive and the more they afford these benefits to the citizens of the country.

The port of Cape Town consists of a range of reinforced concrete structures at varying marine exposures and ages. A representative sample of select structures was analysed and compared to literature to understand the impacts of chloride-induced corrosion. An improved understanding of this environment can lead to the design of more durable structures, or the improved maintenance of existing structures, both leading to more effective investment decisions.

1.3 Research objectives and key research questions

The main objective of this research is to perform as-built data collection, visual condition assessments and cover readings on various reinforced concrete structures in the port of Cape Town, with the aim of understanding the relationship between marine exposure, cover depth, and deterioration due to chloride-induced corrosion.

The research aim above translates to several key research questions which are summarised below:

- i. Which reinforced concrete structures and their location in a port environment would represent a reasonable sample for the study?
- ii. Which as-built information is available for these selected structures?
- iii. What type of condition assessment and testing methods are the most practical for this application?
- iv. What effect do various parameters, including marine exposure class, cover depth, exposure to wind, and age, have on the deterioration of the structures selected?

1.4 Scope and limitations

This study is limited to structures in the port of Cape Town in the Western Cape Province of South Africa. Since this location is in and around the marine environment, the study will focus on deterioration as a result of chloride induced corrosion.

The study is further limited by the number and extents of reinforced concrete structures that will be identified together with any as-built information available. While care was taken to select random structures and components spread across the port area, certain structures may be more well known based on their adverse condition, which could affect the results. Further, data collection considered time constraints for field work, which would affect the data sets obtained. All data to be collected and access to the site will be upon approval and permission being granted by the controlling authority.

The data analysis performed in this study considered the impact of single parameters such as distance from the sea and minimum cover on deterioration. This rarely occurs in practice and must be noted when considering the results.

1.5 Plan of development

The dissertation comprises four chapters, namely: Chapter 1, which provides a brief background to the proposed topic, outlines the research objectives, and summarises the scope and limitations.

Chapter 2 presents a literature review in which relevant literature on deterioration mechanisms in relation to chloride induced corrosion are reviewed, together with exposure conditions and their effects, and condition assessments including visual, non-destructive, and destructive testing.

Chapter 3 presents the data obtained through as-built drawings, visual condition assessments and cover meter readings and this data is analysed to determine trends in deterioration compared to various structural characteristics and exposures. Finally, Chapter 4 provides a summary of the findings, a conclusion of the research conducted, and recommendations for future research.

2. Literature review

2.1 Introduction

Concrete structures are expected to perform their intended function over typically long service lives such as 50 years, or 100 years or more in the case of critical infrastructures. Modern structures are expected to perform with as little maintenance as possible over their service lives. To achieve this, sufficient knowledge of their long-term durability is required in the design and construction phase, and in the management of existing structures, particularly in aggressive environments such as the marine environment. Here structures are exposed to chloride ingress and seawater erosion among other effects which can cause premature deterioration and failure in severe cases. (Li et al., 2022)

2.2 Concrete durability in the marine environment

The durability of reinforced concrete structures is defined as the ability of a structure or component to withstand the design environment over the design life without a loss of serviceability or the need for substantial maintenance (Alexander & Beushausen, 2019).

Durability and deterioration are complex problems, and while there has been a wide range of research on the topic, leading to a better understanding of design requirements, these have not permeated into actual construction. The result is many existing and new structures exhibiting similar deterioration challenges that could have been prevented or delayed.

The marine environment is particularly severe. Here the exposed concrete will undergo physical and chemical changes due to environmental exposure and service loadings respectively (Li et al., 2022). Experience has shown that any defect will become evident fairly quickly and therefore, cognisance should be taken by all parties involved in the construction and maintenance process. The following are common and likely reasons for damage in this environment (Thoresen, 2014):

- a. Poor quality of concrete or inadequate design
- b. Insufficient concrete pouring practice
- c. Insufficient concrete cover
- d. Ineffective surface drainage
- e. No maintenance or inspections performed.

The factors above have been grouped into three categories namely design, construction and service life representing the three main lifecycles of a structure and their impacts on durability are discussed in more detail below.

2.2.1 Design phase

The design phase involves the concrete mix design considering the prevailing environmental conditions, the selected material factors and the loading of the structure. Traditionally, the mechanical property of compressive strength was used as an indication of the durability of a structure. This has been shown to be inadequate, partially because various mix designs can be used to arrive at the same compressive strength (Alexander & Beushausen, 2019). The prescriptive type of design in which minimum mechanical targets are set have various limitations and the performance-based design approach is preferred for durability. One of the limitations of prescriptive design is the lack of correlation between test results and the actual condition of the structure. Figure 1 below highlights this lack of correlation in which cube tests of specimens in the lab are plotted against oxygen permeability tests performed on cores removed from the actual structures.

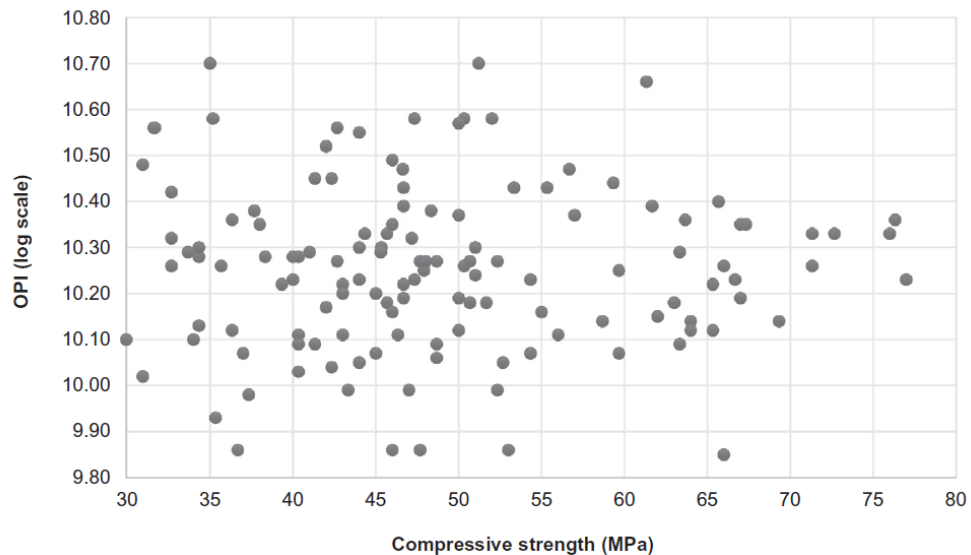


Figure 1 Lack of correlation between cube strength tests from specimens in the laboratory versus oxygen permeability tests measured on core samples obtained on site (Alexander & Beushausen, 2019)

Concrete in the marine environment will be exposed to sea water and will therefore be affected by chloride ingress. Sea water consists of about 3.4% dissolved salts, which are mostly chlorides and sulphates of sodium and magnesium. These would have a negative impact on the hardened state of concrete. (Alexander, Bentur & Mindess, 2017)

Fresh concrete can be influenced by using sea water when mixing and therefore this should not be done. This will lead to accelerated corrosion rates due to chlorides already in the mix, and ultimately a lower strength (Alexander, Bentur & Mindess, 2017). Hardened concrete which is exposed to seawater in various forms is required to withstand erosion and chloride ingress. The following considerations in the mix design have been shown to improve durability (Alexander, Bentur & Mindess, 2017):

- Low water/binder (w/b) ratio
- Blended cement with silica fume
- Low alkali cement
- Use of Supplementary Cementitious Materials (SCMs) (Fly ash, GGBS, GGCS)

The low w/b ratio and use of silica fume restricts oxygen and water at the level of the steel and prevents electron flow within the steel, both delaying chloride ingress. In addition, Supplementary Cementitious Materials (SCMs) such as fly ash, Ground Granulated Blast Furnace Slag (GGBS), and Ground Granulated Corex Slag (GGCS) from the Western Cape has been shown to improve durability in this environment.

The steel cover plays a significant role in delaying chloride-induced corrosion. The marine environment typically has concrete cover in the range of a minimum 50 mm (typically above a berth slab) to 120 mm (in the tidal zone) (Thoresen, 2014). Guidance on cover depths is provided in the design codes locally in Section 8.2 of SANS 10100-2 (2014) and in international codes such as EN 206 2013 for varying degrees of the aggressiveness of the environmental exposure (Cement & Concrete SA, 2021). These minimum cover depths are typically specified for a design life of 50 years but can be adjusted for the prevailing design conditions and increased or decreased accordingly. For example, an increased design life of 100 years could result in an increase of cover by 10mm, and the use of stainless steel for reinforcement or protective coatings could result in a decrease of cover depth (Fulton's Concrete Technology (10th rev. edition), Midrand, Cement and Concrete SA, March 2021).

Finally, the proportioning of the concrete elements should be considered to allow for concrete to be transported, compacted, and finished without segregating and to produce the designed durability and strength (Alexander, Bentur & Mindess, 2017). Trial batches should be made to test the mix design prior to implementation and can support the performance base design in which service life prediction models are used to model the durability of a structure.

2.2.2 Construction phase

Most factors which ultimately impact durability is related to construction practice and therefore it must be managed closely with sufficient quality control procedures at all steps in the process.

The batching process must be carried out accurately to ensure the correct materials and proportions thereof are used. The volume of mixing water added must be monitored closely both at the batching plant and on site as this will play an important role in concrete strength and durability, affecting the w/b ratio (Alexander, Bentur & Mindess, 2017).

The transport, placement, and compaction must be carried out correctly and finished and cured as stipulated. Common challenges such as segregation can be mitigated by placing the wet concrete correctly and entrapped air should be avoided by sufficient compaction (Alexander,

Bentur & Mindess, 2017). In addition, the cover depth must be checked prior to placement by inspecting cover blocks and considering their material.

Once the concrete has been cured, checks can be conducted to confirm cover depth and other factors. This is an important step to ensure what has been designed is in fact, what has been built. In the event of any discrepancies, remedial measures can be specified and implemented such as coating systems.

2.2.3 Service life

Service life of a structure is defined as the period for which a structure or part thereof is used for its intended purpose with planned maintenance being conducted but without the need for major repair. Service life is related to the serviceability limit state condition (SLS), for which the durability limit state condition (DLS) can be considered a sub-limit. Increasingly, asset owners require certainty around extended service life periods with minimal maintenance. This is not simple in the modern world where environmental conditions are changing, modern materials are being developed or recycled, construction quality is uncertain, and there is inadequate knowledge around prediction models. Service life design can be considered in two scenarios namely the design of new structures and the remaining service life of existing structures. These are considered below. (Alexander & Beushausen, 2019)

2.2.3.1 New Structures

Regular inspections and maintenance are generally required to ensure the structures achieve their service life. In new structures, service life prediction models can be used to model ingress of chlorides, and at which point interventions will be required (Bastidas-Arteaga & Schoefs, 2015). Service Life Modelling (SLM) should combine deterioration modelling and damage modelling to be able to assess quantitatively the time it would take for a structure to reach an unacceptable level of damage such as cracking, spalling etc, and to determine the effect such damage would have on the performance of the structure. (Alexander & Beushausen, 2019)

The accuracy of these models requires them to be semi-empirical at present with a combination of laboratory and site data required. Once sufficient data is collected in the initial stages of the service life, this information can be used to constantly assess performance versus design during the structures service life. (Alexander & Beushausen, 2019)

2.2.3.2 Existing structures

In existing structures, once certain parameters are measured such as the chloride diffusion coefficient, and chloride content, a similar approach can be adopted. This can allow for interventions to be implemented timeously prior to structures being completely saturated with chlorides or prior to major damage occurring. Once delamination and spalling are widespread, and patch repairs have started, this will likely continue on a regular basis. The only longer-term

solution which may assist here is cathodic protection, depending on the level of deterioration and the service life requirement.

2.3 Chloride-induced corrosion

2.3.1 Corrosion initiation and propagation

Steel reinforcement in concrete is initially protected by the high alkalinity of the cement matrix ($\text{pH} > 12.5$) which passivates the steel surface. This passive ferric oxide film on the surface of the steel together with the concrete cover, limits the ingress and effects of deleterious substances such as oxygen, moisture, carbon dioxide, and chlorides which are required for corrosion to initiate and continue (Alexander, Bentur & Mindess, 2017).

The ingress of species such as chlorides and carbon dioxide can depassivate the protective layer leading to the initiation of corrosion (Otieno, 2014). This process and the service life of reinforced concrete structures as it relates to corrosion is typically modelled using a distinct phased approach as first proposed by Tuutti. The Tuutti model makes use of two phases namely the initiation phase and the propagation phase. This model can further be modified as indicated in Figure 2, in which the propagation phase has further been modified and enhanced. The corrosion process is described in the two-phase model of Figure 2 below:

- a. Initiation phase, time taken to depassivate the protective ferric oxide film and for which there is no visible damage to the structure,
- b. Propagation phase, time taken to form cover cracks and the time to spalling of the concrete cover. This is followed by an acceleration in corrosion as deleterious substances can more easily penetrate the cover and enhance the corrosion process, eventually leading to a reduction in structural capacity and the time to structural collapse.

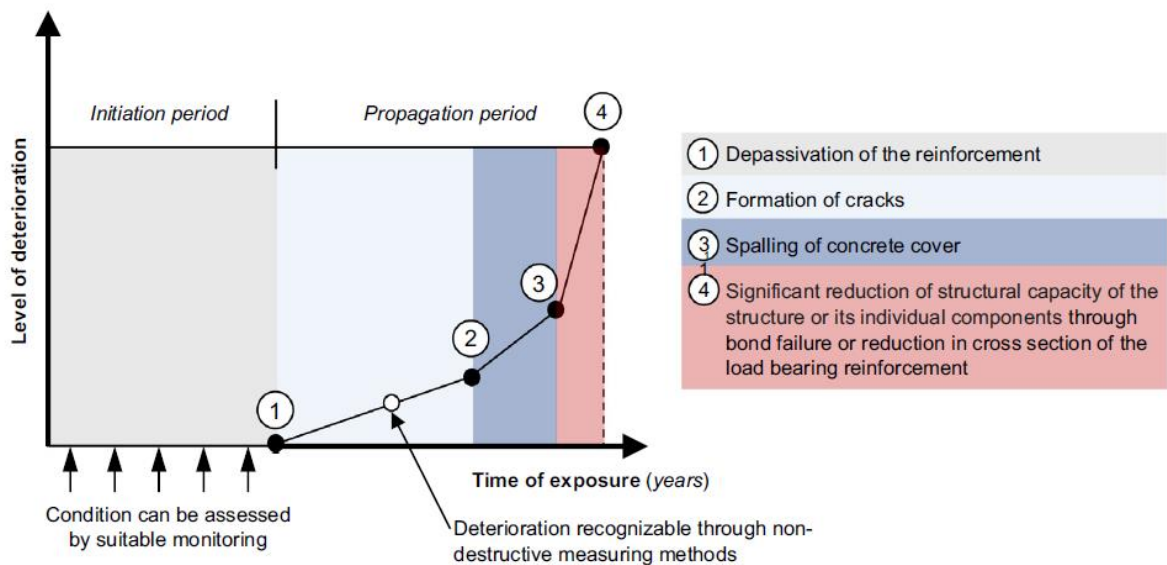


Figure 2 Two-phase corrosion model (Alexander & Beushausen, 2019)

2.3.2 Mechanisms of corrosion

Corrosion is an electrochemical process in which electric current flows between a metallic surface and a conducting environment. It typically consists of an anode, cathode, electrolyte and a link between the anode and cathode. Steel is prone to corrosion, however, the pore solution in concrete has a passivating effect on steel where a protective oxide layer is formed. Corrosion occurs when there is a change in the pore solution which subsequently prevents this protective action and causes a breakdown in the oxide layer. This altering is mainly caused by carbon dioxide ingress (carbonation) or chloride ion ingress. (Alexander & Beushausen, 2019)

Figure 3 shows a schematic of chloride-induced steel corrosion. Here the chloride ions present in sea sprays for example, penetrate the concrete cover by transport mechanisms such as capillary absorption, diffusion, and permeation. Once sufficient chloride ions accumulate at the interface between the concrete and steel beyond a critical chloride threshold, the passive oxide layer is removed, resulting in corrosion. (Arito, 2016)

Chloride induced corrosion typically results in macrocell corrosion in which small distinct anodic sites and large cathodic sites are created which are separated from each other. This results in the more severe local loss of steel reinforcement also known as pitting corrosion which can also be seen in Figure 3.

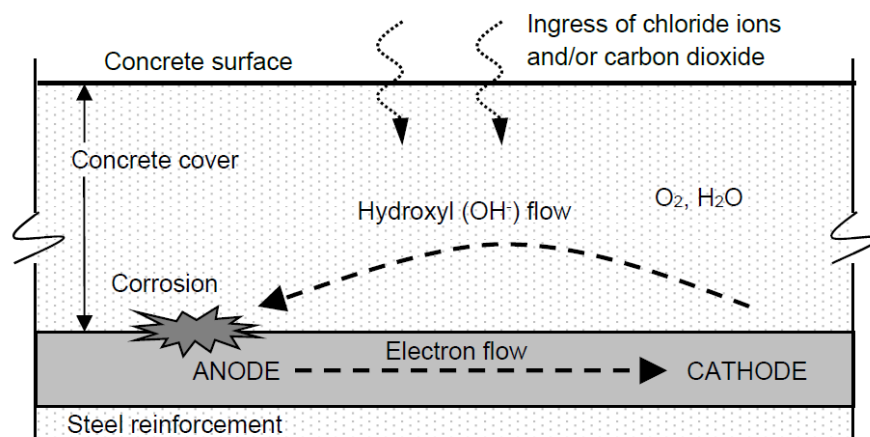


Figure 3 Corrosion mechanisms in reinforced concrete (Beushausen, 2022)

2.3.3 Chloride binding

Chlorides may be free and therefore available for corrosion or bound in the cement matrix. The free chlorides are dissolved in the cement pore solution and become bound or attached either chemically by reacting with the aluminate phases or physically to the hydrate surfaces by absorption. This process reduces the free chlorides from the pore solution and increases the time of reaching the chloride threshold. The type of binder used therefore strongly influences the rate at which chlorides penetrate concrete from an external environment. Blended binders which contain fly ash or slag have been shown to have greater chloride binding properties and can extend the service life of structures. (Alexander, Bentur & Mindess, 2017)

2.3.4 Chloride threshold

The chloride threshold level or critical chloride concentration refers to the concentration of chlorides required for depassivation to occur. This is typically an input into service life prediction models and may vary based on various factors such as cover depth, temperature, relative humidity, chemistry of the binder among others. Since this value may not always be known, it is conservatively taken as 0.4% by mass of the total binder. (Alexander, Bentur & Mindess, 2017)

2.4 Select factors affecting corrosion rate

As with durability considerations of concrete structures in the marine environment, there are several interrelated material and environmental factors which affect chloride induced corrosion and the rate thereof. These factors often occur in conjunction with others and are summarised in the Table 1 below.

Table 1 Factors affecting chloride ingress (Otieno, 2014)

Factor affecting corrosion rate	Description
Cover depth	Cover depth affects chloride ingress by influencing the ease of ingress of chlorides, moisture, and oxygen.
Concrete quality (binder type and w/b ratio)	The use of supplementary cementitious materials can improve the microstructure while the water-to-binder ratio (w/b) plays an important role in the pore structure. A good quality concrete with less pores would result in less penetrability.
Concrete resistivity	The electrical resistivity of concrete related to penetrability of fluids and diffusivity of ions. Affected by a wide range of factors however the degree of saturation of the pore structure is most important.
Temperature	Higher temperature results in higher corrosion rate up to a temperature of 40 degrees Celsius after which there is an inhibiting effect.
Relative humidity	Corrosion activity is most vigorous at a relative humidity above 80%. Rate of ingress of oxygen to cathodic sites depends on diffusion coefficient.
Oxygen availability	Oxygen availability at the steel level affects corrosion. Affected by relative humidity and concrete quality.
Cracking	Cracking allows for access pathways to steel and therefore can accelerate corrosion.
Environmental exposure	Marine exposure classes can be divided into four namely the submerged zone, the tidal zone, the splash and spray zone, and the airborne zone. These exposure classes have varying impact on the corrosion rate based on factors such as moisture content and oxygen availability among others.

Chloride transport in concrete is more well defined and understood in uncracked concrete than in cracked concrete. Cracked concrete does, however, have an impact on chloride ingress. A select review of some of the factors which affect the corrosion rate as summarised in Table 1 are covered in more detail below.

2.4.1 Cover depth

Insufficient cover depth is likely the most significant cause of premature deterioration in reinforced concrete structures. This can be caused by incorrect specifications or by construction practice in which the specified cover is not achieved. (Alexander, Bentur & Mindess, 2017)

Cover thickness, together with other factors such as concrete quality and condition, play an important role in the ease of ingress of deleterious substances such as chlorides, moisture, and oxygen, which impacts the rate of corrosion.

The Figure 4 shows that for both cracked and uncracked concrete at a given w/b ratio, the corrosion rate increases with a decrease in cover depth for field measured samples. This is attributed mainly to a lack of oxygen availability as cover increases. (Otieno, 2014)

It can therefore be noted that a suitably specified and implemented cover depth, can significantly improve the service life of a reinforced concrete structure by hindering oxygen availability at the level of the steel and thereby reducing the corrosion rate.

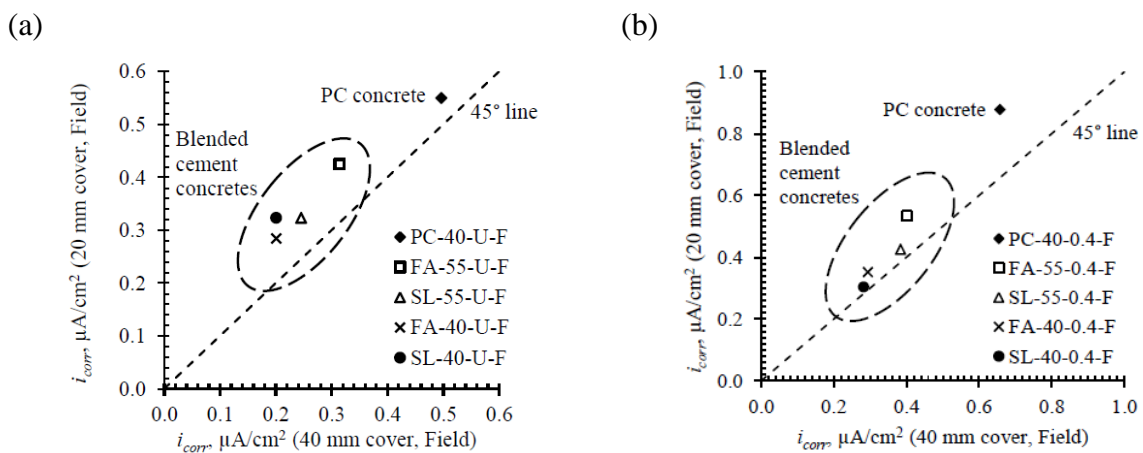


Figure 4 Influence of cover depth and binder type on corrosion rate on reinforced concrete structures in the marine environment for (a) uncracked concrete and (b) 0.4 mm cracked concrete (Otieno, 2014)

2.4.2 Concrete quality

Concrete quality impacts the rate of corrosion by affecting the penetrability of deleterious substances through the pore structure and by influencing the chemical resistance. The mix design should seek to minimize the pore structure through the w/b ratio and use of SCMs and maximize chemical resistance through an appropriate binder. (Alexander, Bentur & Mindess, 2017)

The w/b-ratio affects the corrosion rate by defining the pore structure. A lower w/b ratio results in a smaller pore size, reducing the penetrability and thereby reducing the corrosion rate. Conversely, a high w/b ratio results in a larger pore size, increasing penetrability and corrosion rate. (Otieno, 2014)

The use of SCMs through partial replacement of Portland Cement by fly ash, GGBS or GGCS has shown to improve the microstructure of concrete. GGBS creates a dense matrix with finer pores while fly ash also results in a finer pore with gel pores being higher and capillary pores lower. Fly ash is particularly effective in reducing the corrosion rate in chloride environments. (Otieno, 2014)

2.4.3 Cracking

Cracking can increase the corrosion rate by providing access pathways to steel for the penetration of deleterious species. The corrosion initiation and propagation can be influenced by pre-corrosion cracking, crack width and depth, frequency, orientation and self-healing and activity or dormancy. The rate at which cracks influence corrosion is also dependent on concrete quality and cover depth. (Otieno, 2014)

Design codes such as BS 8007 (1987), SANS 10100-1 (2000) and EN 1992-1-1 (2000) provide guidance on acceptable crack widths at the concrete surface. For example, they typically permit 0.3mm for normal structures and 0.2mm for water retaining structures (Cement & Concrete SA, 2021).

2.5 Marine exposure

The chloride material interactions and transport mechanisms vary depending on the marine exposure zones and whether interactions occur at the surface of the concrete or in the bulk as indicated in the Figure 5 below. These exposure zones impact the durability of concrete and their service life and should therefore be considered in design and maintenance. These processes also play an important role in service life prediction models.

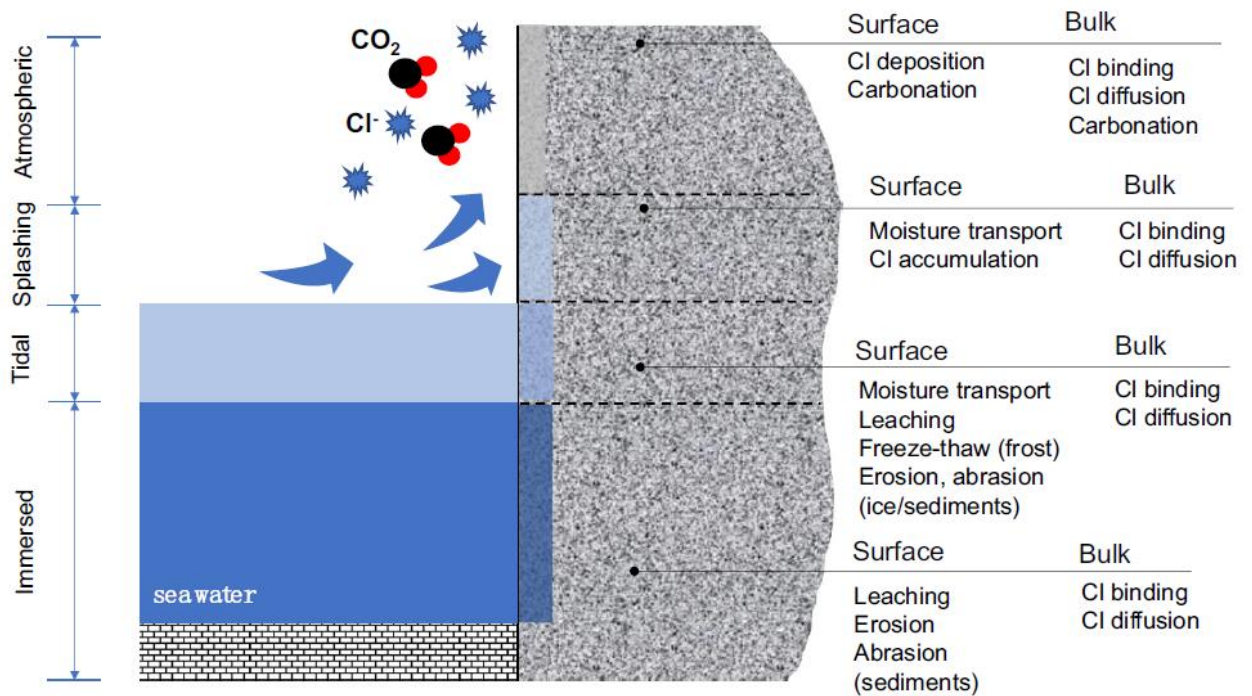


Figure 5 General durability processes of concrete in the marine environment (Li et al., 2022)

2.5.1 Chloride-ion transport processes

The transport of chloride ions plays a critical role in the rate of chloride ingress and therefore service life design. The main transport processes in uncracked concrete include capillary absorption, diffusion, permeation, wick action and migration. In cracked concrete, diffusion and advection play an important role among others (Otieno, 2014). These mechanisms are summarised in the Table 2 below focusing on uncracked concrete.

Table 2 Summary of Chloride-ion transport processes (Arito, 2016)

Transport mechanism	Description
Chloride diffusion	Key transport mechanism. Defined as the movement of fluids and ionic species due to a concentration gradient. Key factors affecting the rate of diffusion include the moisture content, temperature, diffusivity of material and diffusing substance.
Permeation	Fluid movement due to pressure gradients. Occurs within pores in the marine tidal zone and is impacted by permeability and the diffusion coefficient. Wave action can provide the pressure required to move chloride ions through permeation.
Capillary absorption	Suction which takes place where partially saturated concrete absorbs chloride ions through moisture gradients. Typically found in the airborne zone in marine environments.
Convection	Occurs in combination with other transport processes such as diffusion. Chloride ions are transported through wetting and drying cycles in the marine environment.
Migration	Occurs when the movement of ions are influenced by an electric field. Typically occurs in laboratory accelerated chloride tests.
Wick action	Occurs when one side of a thin concrete section is exposed to moisture while the other is kept dry causing moisture and ions to move to the dry surface. Typically found in areas of constant wetting and drying and can occur in combination with diffusion.

2.5.2 Combined transport

Chloride ion transport typically occurs as a combination of the above transport mechanisms. In addition, Figure 6 illustrates how chloride ions are transported in the marine environment based on various exposure classes.

It can be noted that the submerged zone typically results in movement through diffusion. The tidal and splash and spray zone, with constant wetting and drying, can result in transport through a combination of permeation, convection, and diffusion. The airborne zone can be impacted by wick action in combination with diffusion.

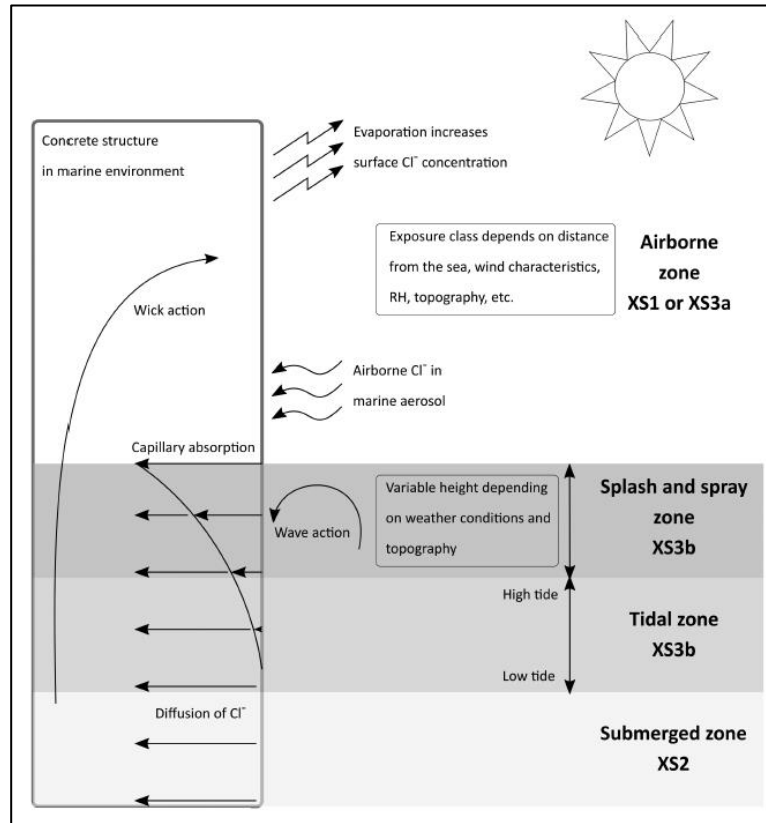


Figure 6 Marine exposure classes and transport mechanisms (Beushausen et al., 2021)

The airborne zone in the Figure 6 is further broken down into two zones based on severity. It has been found that this zone can be severe depending on site conditions and each site should therefore be considered based on their prevailing conditions.

2.6 Chloride ingress prediction models

Service life models for chloride ingress may be either numerical or empirical in nature. They consider the environment and exposure condition in which the structure is situated together with the material characteristics to evaluate the performance of concrete and make use of the diffusion coefficient as an input parameter. (Arito, 2016)

Ingress models typically make use of Fick's laws of diffusion where the first law (steady state) is used in conjunction with the second law (unsteady state). The law of conservation of mass is used in conjunction with the first law to arrive at the following equation:

$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial x} \left(D \frac{\partial c}{\partial x} \right)$$

Where c is the concentration in free chlorides (g/m^3); t , x represents time(s) and position (m) respectively; and D is the diffusion coefficient (or diffusivity) for chloride ingress in concrete (m^2/s)

Several chloride ingress models from the literature have been listed and their salient details summarised in the Table 3 below.

Table 3 Chloride ingress prediction models (summarized by Arito, 2016)

Chloride ingress model	Description	Equation	Key parameters	Comment
Colleparidi (ERFC)	Mathematical Error Function Complement based on Ficks 2 nd law of diffusion	$C(x,t) = C_i + (C_s - C_i) \cdot \operatorname{erfc}\left(\frac{x}{2\sqrt{D_a \cdot t}}\right)$	Surface chloride concentration and diffusion coefficient – determined by curve fitting chloride ingress profiles from specimens	Only describes chloride ingress in the short term
ClinConc	Chlorides in concrete is a numerical model for the submerged environment but can be modified for structures above water. Uses Ficks 2 nd law of diffusion followed by mass balance and non-linear chloride binding.	$D_0 = \frac{(0.8\alpha_1^2 - 2\alpha_1 + 2.5)(1 + 0.59K_b) \times k_{TP}}{1 + k_{OH} \times K_b \times k_{TS} \times f_b \times \beta_b \left(\frac{c_s}{35.45}\right)^{\beta_b - 1}} \times D$	Temperature, curing age, concrete cover and the effects on diffusivity	Takes into account convection and ionic migration in addition to diffusion
Life-365	Developed by American Concrete Institute. Based on Ficks 2 nd law of diffusion focusing on total chloride content.	$D(t,T) = D_{ref} \times \left(\frac{t_{ref}}{t}\right)^m \times e^{\frac{t(t-1)}{n \times 365 \times t}}$	Temperature effect on diffusion coefficient	Simplifications and assumptions must be taken into account. Limited use outside of North America
DuraCrete	Probabilistic service life prediction. Uses 28-day diffusion coefficient measured using tests based on Ficks 2 nd law. Time dependent variation of diffusion coefficient can also be used.	$D_0(t) = D_0(t_{ex}) \times \left(\frac{t_{ex}}{t}\right)^n$	Chloride concentration at exposed surface, measure chloride diffusion coefficient and exposure time	Relies on binder type and exposure environment
South African chloride prediction	UCT Service Life model based on environmental conditions, concrete quality and cover. Characterises chloride resistance using early age (28 to 35 days) chloride conduction properties.		Binder type, initial curing, marine exposure, and concrete maturity.	Can be used to predict long term performance to chloride resistance

Time dependency of the diffusion coefficient in the models can be addressed by using an empirical relation. This can be taken into account by substituting into Ficks 2nd law or into the various models such as the error function solution above.

2.7 Condition assessment

Condition assessments are typically carried out as a precursor to modifications or repairs of structures or routinely as may be required. In this case, a condition assessment strategy was developed, focusing on the effects of corrosion, and with the intention of determining the following (Cement & Concrete SA, 2021):

1. The cause of damage
2. The extent of the damage
3. The grade of the damage

The assessment began by gathering all existing information of the structure including as-built information and any previous maintenance or refurbishment projects. Due to time constraints, and permissions required, destructive tests were not carried out and therefore, this has not been covered in this review.

2.7.1 Visual assessment

The visual assessment is key in this study and will inform any subsequent non-destructive testing. Once all existing structural data is obtained, a systematic visual assessment will be carried out.

The identification of defects, focusing on corrosion effects, will focus on the presence and extents of delamination, spalling, pitting corrosion, rust staining, and any other concrete quality issues such as honeycombing, blow holes, and segregation.

Once the damage is identified and the extents are measured, the damage is to be classified in terms of an industry recognised grade. Table 4 below will be used to classify the respective damage and is based the guidelines developed by the RILEM technical committee 104-DDC 1994.

Table 4 Damage classification of defects (Cement & Concrete SA, 2021)

Damage	Damage rating				
	1 (very slight)	2 (slight)	3 (moderate)	4 (severe)	5 (very severe)
Cracks in prestressed concrete due to overloading	Width < 0.05 mm	Width 0.05-0.1 mm	Width 0.1-0.3 mm	Width 0.3-1 mm	Width 1-3 mm with some spalling
Cracks in reinforced concrete due to overloading	Width < 0.1 mm	Width 0.1-0.3 mm	Width 0.3-1 mm	Width 1-3 mm with some spalling	Width > 5 mm with widespread spalling
Cracks in unreinforced concrete	Width < 1 mm	Width 1-10 mm	Width 10-20 mm	Width 20-25 mm	Width > 25 mm with widespread spalling
Shrinkage or settlement cracks	Single small crack	Several small cracks	Many small cracks	Few large cracks	Many large cracks
Effects of reinforcement corrosion	Barely noticeable	Light rust stains	Heavy rust stains	Heavy rust stains and cracking along line of bars	Heavy rust stains and spalling along line of bars
Pop-outs	Barely noticeable	Clearly noticeable	Holes up to 10 mm diameter	Holes between 10 and 50 mm diameter	Holes > 50 mm diameter
Spalling	Barely noticeable	Clearly noticeable	Larger than coarse aggregate	Areas up to 150 mm across	Areas larger than 150 mm

Equipment to be used will include a crack width ruler, a tape measure, a camera and a small hammer for a simple delamination survey.

2.7.2 Non-destructive testing

Non-Destructive Testing (NDT) was planned to follow the visual inspection. These tests are related to corrosion and was carried out factoring in instrument availability, access to the structures and allowable time. It was envisaged that cover will be measured using a cover meter. Since no other tests are considered at this stage, only the cover meter is discussed below.

2.7.2.1 Cover measurements

Cover surveys are performed to determine the position and depth of reinforcement and are typically carried out using an alternating magnetic field to locate steel. These types of measurements can be unreliable when cover is greater than 80 mm or where rebar is closely spaced. In these cases, Ground Penetrating Radar (GPR) or X-ray equipment can be used. Cover surveys are an important consideration in cases where the ingress of aggressive agents may be expected, such as chloride ingress. They can be related and compared to other elements of an investigation such as defect mapping. (Cement & Concrete SA, 2021)

3. Condition assessment and data acquisition

A visual condition assessment was performed on seventeen reinforced concrete structures within the port of Cape Town in the Western Cape Province of South Africa. These structures varied in terms of type, marine exposure, distance from the sea and age. The as-built drawings were used to acquire data related to the age, design concrete strength, and cover depth, and actual cover depth readings were taken of various components of these structures. Furthermore, the predominant wind direction affecting the structural components was recorded. It must be noted that in some cases, the design cover depth and concrete strength information were unable to be found in the archived drawings within the port drawing office.

The visual condition assessment focused on deterioration because of chloride-induced corrosion as it relates to three defects namely:

- i. Effects of reinforcement corrosion (rust staining),
- ii. Pop outs, and
- iii. Spalling.

Pop outs typically relate to defects resulting from unsound aggregate but are defined in this study as small areas of spalling, with extents as described in Table 4.

The extent of this damage was measured and rated on a severity scale based on Table 4, and was captured in a condition assessment document for each structure which can be found in App. A.

Table 5 summarises the details of the assessed structures. Here the structure reference number quoted is kept consistent throughout this document and the age is determined based on the assessment date.

Locations of the structures are recorded using coordinates in the condition assessments and are illustrated in Figure 9. Locations are colour-coded based on the structure type and are referenced based on the structure reference number.

The cover readings of various components of the structures were taken to relate the actual cover to the condition. The measurements and findings are analysed and discussed further in this chapter.

Table 5 Summary of assessed structures

Structure reference number	Name	Type	Age (y)	Distance from sea (m)	Exposure class	Assessment date
1	TNPA House	Building	51	80	XS3a	12/09/2023
2	B-Berth Shed	Building	31	25	XS3a	12/09/2023
3	Electrical Workshop	Building	56	140	XS3a	12/09/2023
4	Lighthouses Building	Building	62	220	XS3a	12/09/2023
5	Unitie Building	Building	46	6	XS3a	13/09/2023
6	Sturrock Altar	Stairway Altar	78	50	XS3b	13/09/2023
7	EM1 Access Bridge	Bridge	58	0	XS3a	13/09/2023
8	EM2 Catwalk Bridge	Bridge	58	0	XS3a	13/09/2023
9	EM Pump Room	Building	23	0	XS3a	14/09/2023
10	TB1 Dolphin Access Bridge	Bridge	58	0	XS3a	13/09/2023
11	Quay 700 Prefab Substation	Building	25	1.5	XS3a	12/09/2023
12	Workshop 17 Substation	Building	46	320	XS3a	14/09/2023
13	T-Bridge	Bridge	47	220	XS3a	13/09/2023
14	Marine Drive Portal Frame	Portal Frame	45	590	XS3a	12/09/2023
15	Container Terminal Seawall	Seawall	43	0	XS3a	12/09/2023
16	Harbour Signal Substation	Building	48	50	XS3a	14/09/2023
17	Salt River Seawall	Seawall	43	0	XS3a	14/09/2023

3.1 Study area

The study area is the port of Cape Town in the Western Cape Province of South Africa. The port is situated in the Table Bay, which is a westward facing embayment located at the northern end of the Cape Peninsula (Daniels et al., 2022). The Cape Peninsula in turn, is situated on the south-west coast of South Africa. The location of the port is indicated in Figure 7 below.

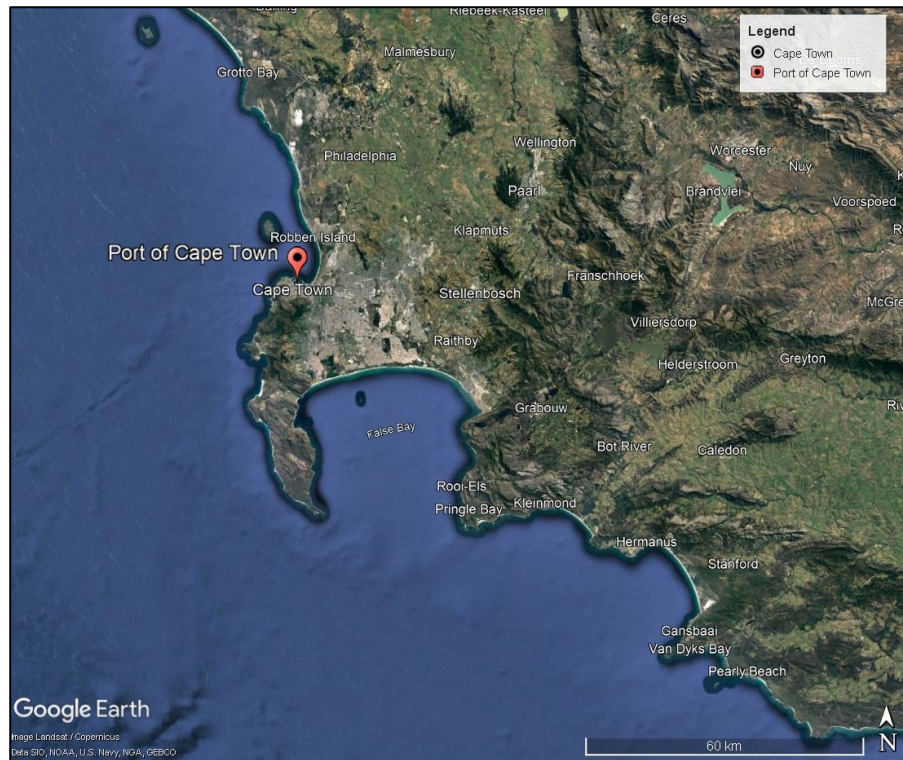


Figure 7 Location of port of Cape Town in Western Cape of South Africa (map extracted from Google Earth Pro)

The port is situated along a busy maritime trade route and is one of South Africa's busiest ports exporting various commodities such as fresh fruit and fish. In addition, it is the second largest container port in South Africa (Troch, Terblanche & Henning, 2021).

The port has a land area of 253 hectares and a water area of 9163 hectares (Transnet National Ports Authority, n.d.). It consists of various reinforced concrete infrastructures such as buildings, bridges, culverts, quay walls, breakwaters and sea walls with varying exposures and distances to and from the sea.

3.1.1 Cape Peninsula climate

The climate in the Western Cape is typically Mediterranean, with warm dry summers and cold, wet winters (Daniels et al., 2022). The average air temperatures range from 15 to 27°C in summer and from 7 to 20°C in winter. In addition, the Table Bay is adjacent to the Atlantic Ocean and experiences sea temperatures which range from 12 to 15°C. (Beushausen, 2022)

3.1.2 Wind speed and direction

The Cape Peninsula wind patterns are determined, at a high level, by the South Atlantic high-pressure cell. In summer, the pressure cell shifts southward, resulting in more dominant South Easterly winds. In winter, the pressure cell shifts northward, resulting in more dominant North Westerly winds. (Daniels et al., 2022)

While there are dominant wind patterns in the Cape Peninsula, the microclimate wind patterns may vary significantly based on the site and the influence of surrounding mountains and other structures. The wind roses shown in Figure 8 below illustrate the wind speed and direction at the Cape Town Yacht Club (within the port of Cape Town) from data obtained from the South African Weather Service (SAWS) over the period 1 October 2008 to 30 September 2009 (Daniels et al., 2022). This information confirms the predominant wind direction in winter being North Westerly and South Easterly in Autumn, Spring and Summer.

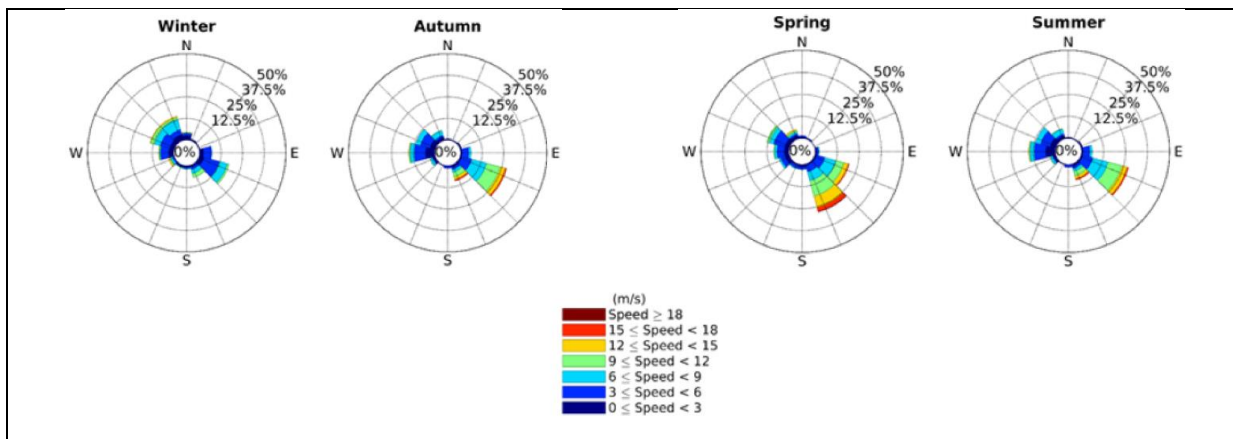


Figure 8 Wind roses at Cape Town Yacht Club for measured SAWS data (Daniels et al., 2022)

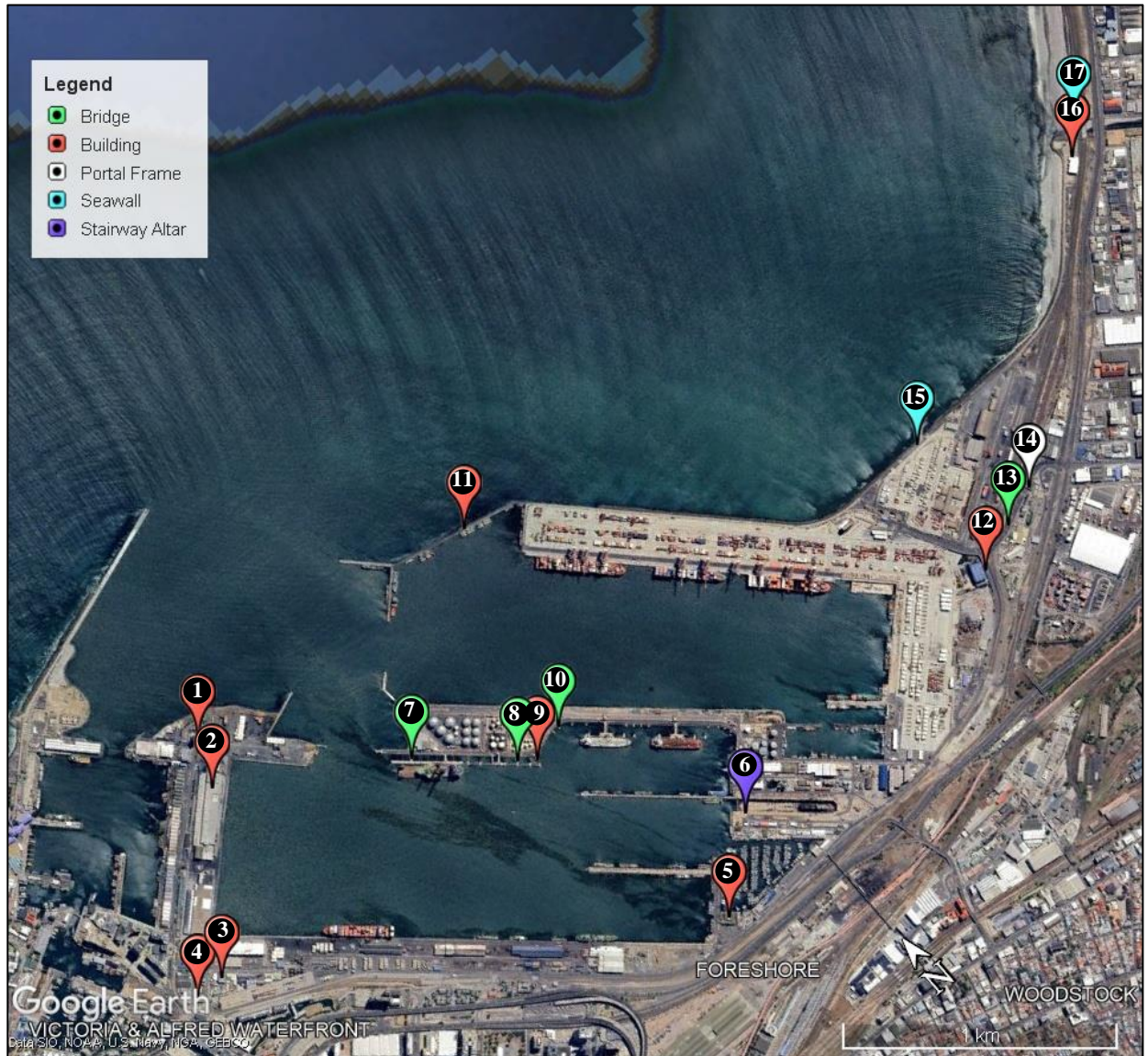
3.1.3 Effect of weather on structures

The exposure of structures to airborne chlorides will be impacted by their position in relation to the sea and the impact of wind and rain. In summer, the strong South Easterly wind will impact structural elements which face this direction. Similarly, in winter, the North Westerly wind will impact structural elements on that side of structures. In the port of Cape Town, it can be seen from the wind roses in Figure 8 above, that wind speeds are stronger in summer, potentially having a more significant impact on structures.

In winter months, the increased rainfall will result in more saturated structures, which can also accelerate chloride induced corrosion. This will depend on the chloride concentration at the level of the steel reinforcement.

3.2 Location of structures assessed

The locations of the structures are shown in Figure 9 below.



1: TNPA House	2: B-Berth Shed	3: Electrical Workshop	4: Lighthouses Building
5: Unitie Building	6: Sturrock Altar	7: EM1 Access Bridge	8: EM2 Catwalk Bridge
9: EM Pump Room	10: TB1 Dolphin Access Bridge	11: Quay 700 Prefab Substation	12: Workshop 17 Substation
13: T-Bridge	14: Marine Drive Portal Frame	15: Container Terminal Seawall	16: Harbour Signal Substation
17: Salt River Seawall			

Figure 9 Location of structures assessed in the port of Cape Town (map extracted from Google Earth Pro)

3.3 Observed defects

Various components of the selected seventeen structures were assessed in terms of effects of reinforcement corrosion, pop outs, and spalling. The damage ratings were found to vary widely between structures; however, typical signs of corrosion were found in most structures. It was also found that different components of the same structure could have varying damage ratings.

Examples of the recorded defects together with their extent are illustrated in the following sections.

3.3.1 Effects of reinforcement corrosion

Reinforcement corrosion is a defect characterised by surface discolouration because of rust staining, or in severe and very severe cases, heavy rust staining accompanied by cracking and/or spalling. This was found at varying rates on the structures inspected.

Figure 10 below illustrates two examples, one relating to rust staining and spalling at the TNPA House building with a damage rating of 5 (left) and the other relating to localised rust staining at the Eastern Mole Approach Bridge with a damage rating of 2 (right).



Figure 10 Effects of corrosion at TNPA House building (left) and Eastern Mole Approach Bridge (right)

3.3.2 Pop-outs

Pop-outs are surface defects which are characterized by holes of varying sizes, the larger holes being more severe. This study defines pop-outs as smaller areas of spalling. The Figure 11 below shows two examples from the inspections namely a pop-out at the Quay 700 Substation building

(left) and at the Eastern Mole Pumproom building (right), both with a damage rating of 5 (greater than 50 mm diameter).



Figure 11 Pop-outs at the Quay 700 Prefabricated Substation (Left) and the Eastern Mole Pumproom (right)

3.3.3 Spalling

Spalling is a surface defect characterized by more widespread loss of cover and exposure of reinforcement, typically caused by expansion of the reinforcement because of reinforcement corrosion. Figure 12 below illustrates very severe spalling on the Unitie building exterior wall (left) and the B-Berth shed floor slab soffit (right), both with a damage rating of 5 as spalling affected areas are greater than 150 mm.



Figure 12 Spalling at the Unitie building exterior wall (left) and the B-Berth Shed (right)

3.4 Cover depth measurements

Cover measurements were performed on various components of the structures investigated to determine the actual depth of reinforcement both as an average and a minimum (the shallowest depths). This was completed using a Proceq Profoscope as shown in the Figure below. A minimum of 25 readings were taken on each component surface based on the guideline by the German Concrete and Construction Association (DBV), which requires a minimum of 20 readings per measured surface.

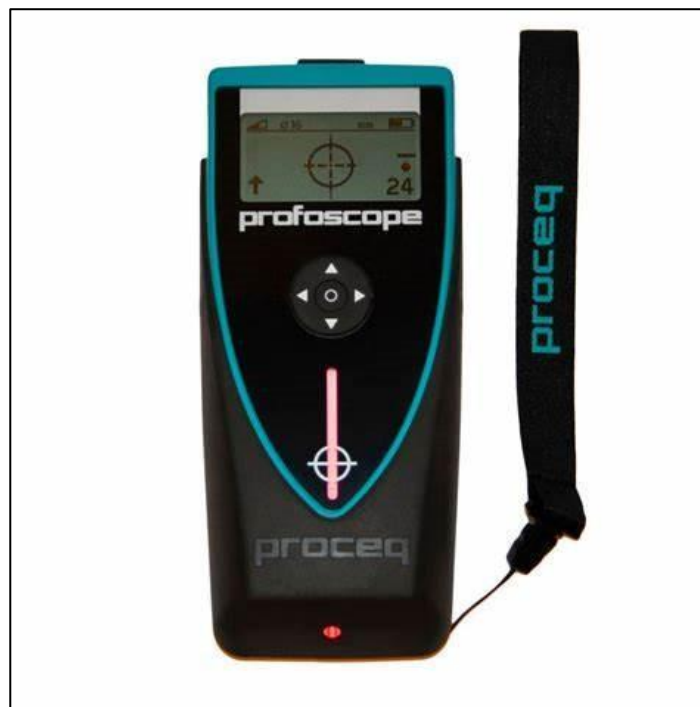


Figure 13 Proceq Profoscope used for cover measurements

A summary of the cover readings is shown in Table 6 with the detailed readings indicated in App. B. In areas where cracking was observed or steel was exposed, these areas were avoided for measurement purposes, with readings taken adjacent to these defects. Components were selected based on ease of access and safety requirements of the controlling authority.

Table 6 Summary of cover measurements

Structure reference number	Name	Structural component	Minimum (mm)	Average (mm)	Standard deviation (mm)
1	TNPA House	Exterior stairway columns West	74	82	5.2
		Exterior stairway columns East	61	84	16.7
		Exterior wall	48	70	10.7
		Roof beam soffit	34	51	6.5
2	B-Berth Shed	Internal column	49	63	8.7
		Internal column	32	71	19.7
		Slab soffit	36	38	1.5
		Slab soffit	25	30	3.5
3	Electrical Workshop	Column North	25	46	13.2
		Column East	43	56	7.6
4	Lighthouses Building	Exterior column	22	28	3.2
		Exterior column	30	37	4.5
5	Unitie Building	Column East	33	51	9.5
		Column West	22	35	12.4
		Wall column	15	25	5.3
6	Sturrock Altar	Altar soffit 1	37	49	8.3
		Altar soffit 2	36	58	8.0
7	EM1 Access Bridge	Beam soffit	44	64	9.8
		Handrail foundation	65	75	5.2
8	EM2 Catwalk Bridge	Beam soffit	27	43	17.7
9	EM Pump Room	Column East	15	35	9.2
		Column South	48	56	3.5

Table 6, continued

Structure reference number	Name	Structural component	Minimum (mm)	Average (mm)	Standard deviation (mm)
10	TB1 Dolphin Access Bridge	Beam soffit	52	64	10.7
11	Quay 700 Prefab Substation	Wall South	27	45	9.1
		Wall North	18	33	8.5
12	Workshop 17 Substation	Column East	45	50	4.4
		Column South	16	36	9.3
13	T-Bridge	Abutment	37	74	15.9
		Column	37	61	15.4
		Column	42	62	8.8
14	Marine Drive Portal Frame	Column	42	51	3.2
		Column	53	58	3.0
15	Container Terminal Seawall	Seawall Interior	54	69	9.1
		Seawall Exterior	60	78	10.1
16	Harbour Signal Substation	Column East	49	61	6.7
		Column South	50	55	4.8
17	Salt River Seawall	Seawall Interior	30	36	5.1
		Seawall Exterior	32	53	9.0

3.5 Data analysis

Seventeen reinforced concrete structures located in the port of Cape Town were assessed in terms of their visual condition and their reinforcement cover based on visual condition assessments and cover readings taken on various components. The structures varied in age, marine exposure, and distance from the sea and consisted of nine buildings, four bridges, one portal frame, one stairway altar, and two seawalls. These structures were selected in consultation with port technical staff to obtain a spread of structure types and locations within the port area. The components of these structures were further selected based on accessibility for cover measurement readings, and to obtain a varied sample of structural component types and locations. It was not possible, given the time constraints, to measure every component of every structure and where components were selected, they were done as random as possible. It can be noted that while care was taken to select random structures and components, bias may have been present from port staff who helped select the structures or the researcher in selecting the components. Subsequent studies of this nature could limit this by increasing the sample size and measuring more components. Table 7 below summarises the condition ratings and cover measurements for the structures and their components.

The data analysis in this chapter is based on analysing one parameter and the influence on deterioration at a time. While this can provide some insights into the effects of these parameters, they rarely occur in isolation of each other, since deterioration occurs through interactions with the various parameters of the concrete system and environmental exposure.

The visual condition assessments confirm significant reinforcement corrosion damage (rust staining, pop-outs, and spalling) in locations on twelve of the seventeen structures. This would have occurred once the concentration of chlorides at the level of the reinforcement had reached the chloride threshold (conservatively estimated to be 0.4% of the mass of cement). In the two-phase corrosion model, this would occur in the propagation phase, which would further imply the end of the maintenance free period in service life prediction models.

In structures or components with no visual signs of corrosion, the risk of corrosion damage can be assessed by additional testing such as chloride testing on core samples. This fell outside of the scope of this study.

Average cover was found to be above 50 mm in twenty seven of the thirty-eight structural components (seven out of the seventeen structures). The scatter in measurements varied per component indicating localised minimum cover below 50 mm. Twenty eight of the thirty-eight components had a minimum cover below 50 mm and only two structures of the seventeen had a minimum cover above 50 mm for all their respective components. In all structures, the minimum cover was found to be below the design cover in localised areas.

Table 7 Summary of cover measurements and damage ratings

Structure reference number	Name	Structural component	Cover measurements (mm)			Damage rating		
			Minimum	Average	Standard Deviation	Effects of corrosion	Pop outs	Spalling
1	TNPA House	Exterior stairway columns West	74	82	5.2	5	5	5
		Exterior stairway columns East	61	84	16.7	1	1	1
		Exterior wall	48	70	10.7	1	5	5
		Roof beam soffit	34	51	6.5	4	1	1
2	B-Berth Shed	Internal column	49	63	8.7	1	1	1
		Internal column	32	71	19.7	1	1	1
		Slab soffit	36	38	1.5	5	5	5
		Slab soffit	25	30	3.5	5	5	5
3	Electrical Workshop	Column North	25	46	13.2	1	1	1
		Column East	43	56	7.6	1	1	1
4	Lighthouses Building	Exterior column	22	28	3.2	1	1	1
		Exterior column	30	37	4.5	1	1	1
5	Unitie Building	Column East	33	51	9.5	1	1	1
		Column West	22	35	12.4	1	5	5
		Wall column	15	25	5.3	5	5	5
6	Sturrock Altar	Altar soffit 1	37	49	8.3	4	5	5
		Altar soffit 2	36	58	8.0	4	3	2
7	EM1 Access Bridge	Beam soffit	44	64	9.8	2	5	4
		Handrail foundation	65	75	5.2	5	5	5

Table 7, continued

Structure reference number	Name	Structural component	Cover measurements (mm)			Damage rating		
			Minimum	Average	Standard Deviation	Effects of corrosion	Pop outs	Spalling
8	EM2 Catwalk Bridge	Beam soffit	27	43	17.7	2	5	5
9	EM Pump Room	Column East	15	35	9.2	5	5	5
		Column South	48	56	3.5	1	1	1
10	TB1 Dolphin Access Bridge	Beam soffit	52	64	10.7	5	5	5
11	Quay 700 Prefab Substation	Wall South	27	45	9.1	2	5	4
		Wall North	18	33	8.5	2	1	2
12	Workshop 17 Substation	Column East	45	50	4.4	1	1	1
		Column South	16	36	9.3	1	1	1
13	T-Bridge	Abutment	37	74	15.9	1	1	1
		Column	37	61	15.4	1	1	1
		Column	42	62	8.8	1	1	1
14	Marine Drive Portal Frame	Column	42	51	3.2	1	1	1
		Column	53	58	3.0	1	1	1
15	Container Terminal Seawall	Seawall Interior	54	69	9.1	5	5	5
		Seawall Exterior	60	78	10.1	2	2	2
16	Harbour Signal Substation	Column East	49	61	6.7	1	4	3
		Column South	50	55	4.8	2	5	5
17	Salt River Seawall	Seawall Interior	30	36	5.1	5	5	5
		Seawall Exterior	32	53	9.0	5	5	4

3.5.1 Effects of proximity from the sea

The damage ratings from the visual inspection are plotted against the distance from the sea for all the respective structures and are presented in Figure 14 below.

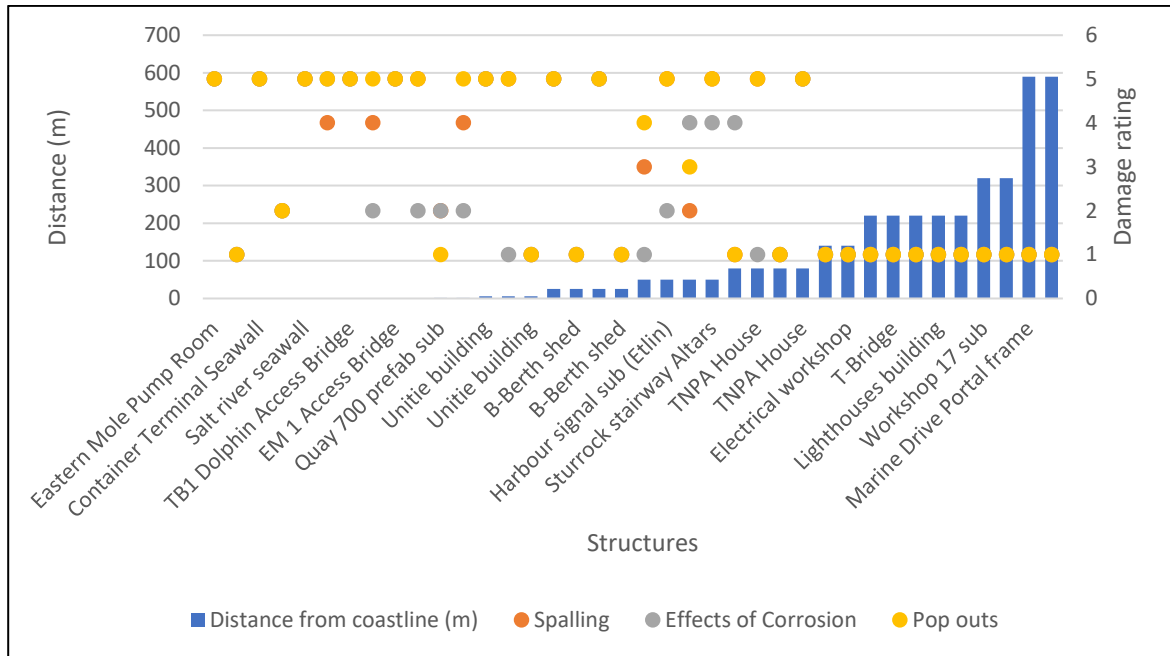


Figure 14 Proximity from the sea compared to damage ratings (condition), where damage ratings are not visible, they are overlaid (25 of 38 components have the same damage ratings per parameter – pop-outs, spalling, and effects of corrosion)

It can be noted that for structures within 100 m of the coastline, there appears to be no obvious trend in damage ratings. For structures further than 100 m however, irrespective of age and other factors, the damage ratings are all one or below, indicating very slight to no damage. For this sample, this highlights the criticality of this parameter on the influence of deterioration of RC structures.

3.5.2 Effects of predominant wind direction

The structural components were grouped in terms of the impact of and exposure to the predominant wind being either the winter North Westerly or the summer South Easterly. It is important to note that a structure could have components exposed to different predominant wind directions, hence the grouping of components and not structures.

Out of a total of thirty-eight components, fifteen were exposed to the South Easterly wind and twenty-three to the North Westerly wind. The average and minimum cover were similar for both groupings. For the South Easterly it was 56 mm and 18 mm respectively and for the North Westerly it was 51 mm and 15 mm respectively. In addition, the average age for each grouping was similar. For the South Easterly it was 48 years (with a standard deviation of 18 years) and

for the North Westerly it was 46 years (with a standard deviation of 8 years). The Table 8 below indicates the average damage rating and standard deviation for each damage type and is further represented by Figure 15.

Table 8 Predominant wind direction and damage ratings

Type of Damage	South Easterly (SE)		North Westerly (NW)	
	Average	Standard Deviation	Average	Standard Deviation
Effects of corrosion	3.5	1.7	1.8	1.8
Pop outs	4.3	2.0	2.0	2.0
Spalling	4.1	1.9	2.0	1.9

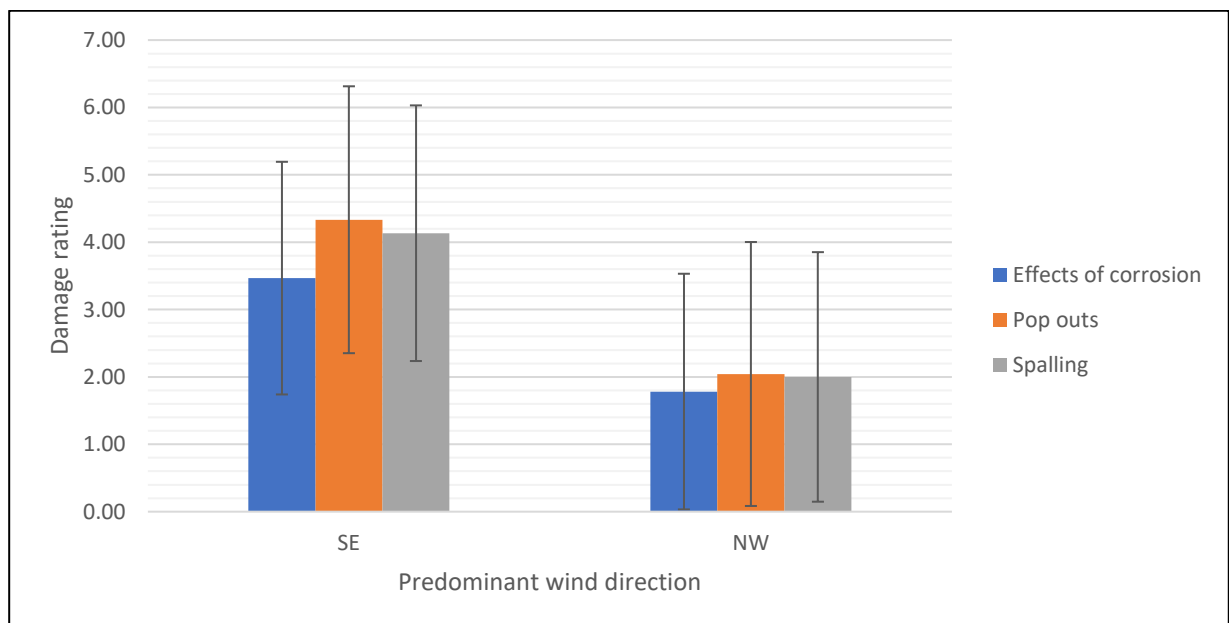


Figure 15 Predominant wind direction and impact on damage ratings

Table 8 and Figure 15 illustrate that in all damage types, the damage ratings were higher for those components exposed to the South Easterly compared to the North Westerly wind. This is expected since the wind roses in the port in Figure 8 highlight a stronger wind speed in Autumn, Spring and Summer for the South Easterly direction compared to the winter North Westerly direction. These winds carry airborne chlorides to the structures and can assist the process of chloride penetration.

3.5.3 Effects of minimum cover

The minimum cover is grouped in ranges of 10 mm starting at 10 mm up to 80 mm. The frequency of the components in each group and the average ratings for the respective damage types are shown in Table 9. This data is illustrated visually in Figure 16.

Table 9 Average damage rating per minimum cover range

Minimum cover range (mm)	Frequency	Effects of corrosion	SD	Pop outs	SD	Spalling	SD
10-20	4	3.3	2.1	3.0	2.3	3.3	2.1
20-30	6	2.0	1.6	3.7	2.1	3.5	2.0
30-40	11	2.9	1.9	2.6	2.0	2.5	1.9
40-50	9	1.1	0.3	2.2	1.9	2.0	1.6
50-60	4	3.3	2.1	4.0	2.0	4.0	2.0
60-70	3	2.7	2.1	2.7	2.1	2.7	2.1
70-80	1	5.0	0.0	5.0	0.0	5.0	0.0

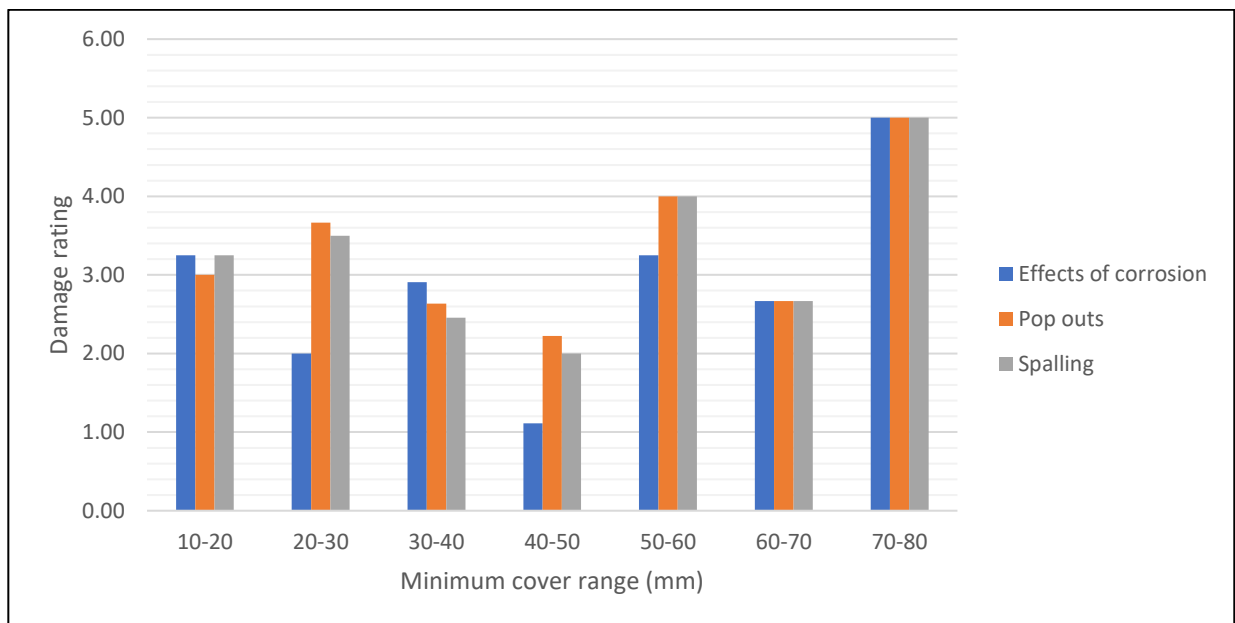


Figure 16 Damage rating per minimum cover range

It is expected that deterioration would decrease with increasing minimum cover and that localised deterioration would be found in areas of shallow cover. Table 9 and Figure 16 appear to show a decreasing apparent trend of damage ratings for a minimum cover from 10 to 50 mm. These samples account for 30 of the 38 component ratings. The apparent trend then appears to indicate an increase in ratings thereafter for all the damage types. Since this is not consistent throughout, the trend cannot be considered and therefore minimum cover is shown to not have an influence on damage ratings.

This highlights the limitations of single parameter analysis. There are likely multiple interacting factors which impact the damage, some of which may be external to this study. The 50 to 80mm range consists of structures such as the TB1 and EM1 access bridges, the Harbour Signal substation, the Container Terminal Seawall and the TNPA House building. All of these structures have severe chloride and wind exposure, in components which have high minimum cover but high deterioration ratings.

3.5.4 Effects of average cover

The average cover is grouped in ranges of 10 mm starting at 20 mm up to 90 mm. The frequency of the components in each group and the average ratings for the respective damage types are shown in Table 10. This data is illustrated visually in Figure 17.

Table 10 Average damage rating per average cover range

Minimum cover range (mm)	Frequency	Effects of corrosion	SD	Pop outs	SD	Spalling	SD
20-30	2	3.0	2.8	3.0	2.8	3.0	2.8
30-40	8	3.1	2.0	3.5	2.1	3.6	1.9
40-50	4	2.3	1.3	4.0	2.0	3.8	1.9
50-60	10	2.1	1.6	2.0	1.7	1.8	1.5
60-70	7	2.3	1.9	3.1	2.0	2.9	1.9
70-80	2	2.0	1.7	2.8	2.0	2.8	2.0
80-90	2	3.0	2.8	3.0	2.8	3.0	2.8

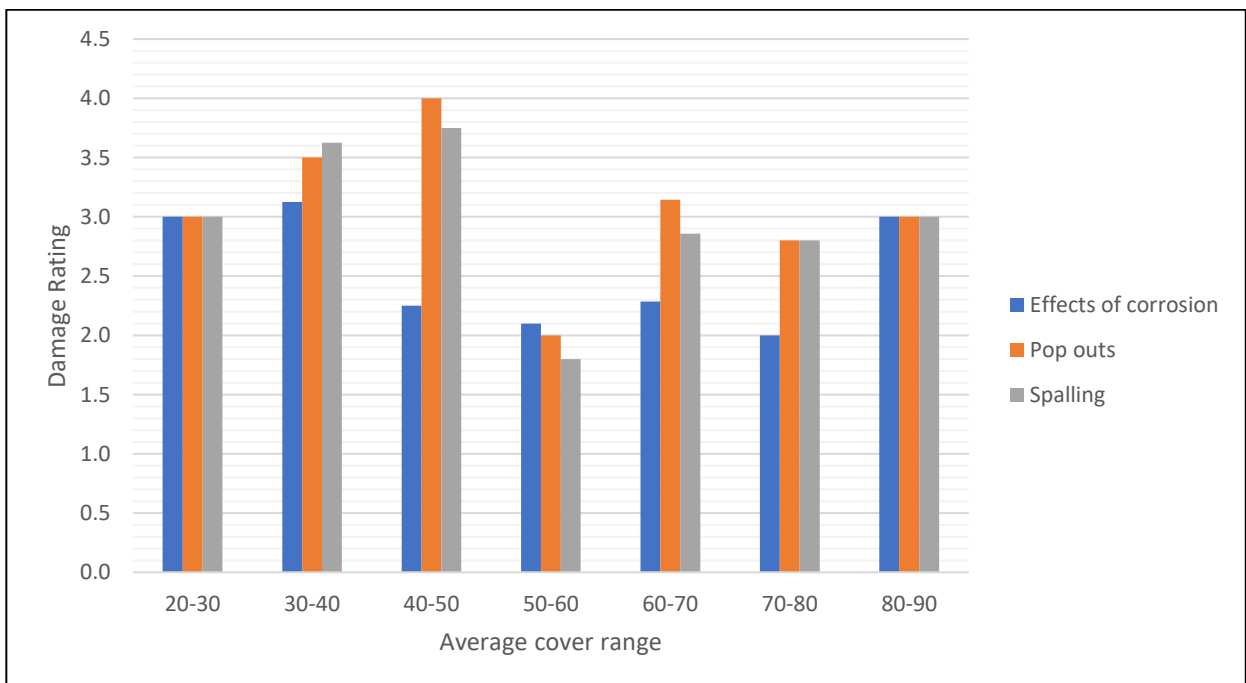


Figure 17 Damage rating per average cover range

Similar to the minimum cover, it is expected that the deterioration would decrease with increasing average cover. There again appears to be an apparent trend between 20 and 60mm average cover however, this is not consistent throughout. As with the minimum cover, average cover does not have an influence on damage ratings in isolation from other parameters.

3.5.5 Effects of the age of the structure

The age of the structures is grouped into 10-year periods, ranging from 20 years to 80 years. The average damage ratings for all the damage types in these periods are shown in Table 11 with the data visually represented in Figure 18.

Table 11 Average damage rating per age of structure range

Age (years)	Frequency	Effects of corrosion	SD	Pop outs	SD	Spalling	SD
20-30	4	2.5	1.7	3.0	2.3	3.0	1.8
30-40	4	3.0	2.3	3.0	2.3	3.0	2.3
40-50	16	2.1	1.8	2.8	2.0	2.6	1.9
50-60	10	2.7	1.8	3.4	2.1	3.3	2.0
60-70	2	1.0	0.0	1.0	0.0	1.0	0.0
70-80	2	4.0	0.0	4.0	1.4	3.5	2.1

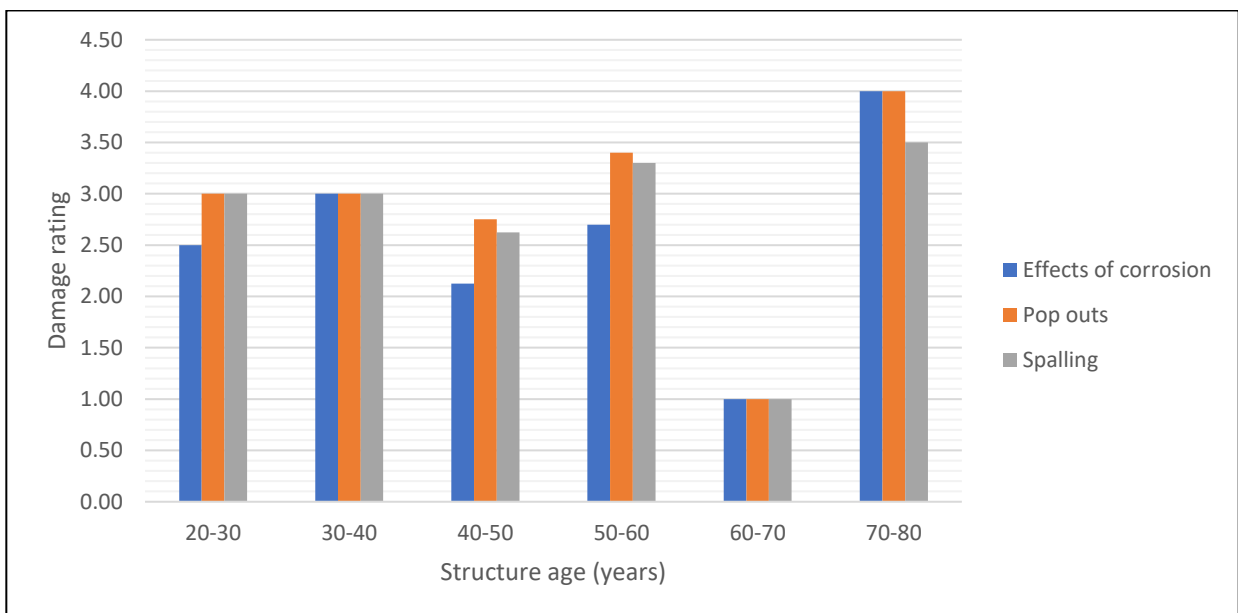


Figure 18 Damage rating per age of structure range

It is expected that deterioration would increase with age. However, overall, this was not found to be the case. There appears to be a decreasing trend followed by an increase in the oldest structure. There does, however, appear to be an increase in damage ratings between the 40-to-50-year range

and the 50-to-60-year range. This range accounts for 26 of the 38 components (68%), making up most of the structural components.

It must be noted that the youngest 8 structural components (3 structures namely Eastern Mole Pump Room, Quay 700 substation, and B-Berth Shed) either have severe chloride and wind exposure or very low minimum cover or both, potentially resulting in premature deterioration. In addition, the second oldest two components (both part of the Lighthouses building), is 220 m from the coastline, and shows no visible deterioration. The comparably sheltered location could account for the relatively good condition compared to the other structures. This again, highlights the limitations of single parameter analysis.

4. Summary, conclusions and recommendations

4.1 Summary and conclusions

The objective of this research was to perform as-built data collection, visual condition assessments and cover measurements on various reinforced concrete structures in the port of Cape Town, with the aim of understanding the relationship between the parameters of marine exposure, cover depth, and age of structure to the damage rating related to chloride-induced corrosion. The study included a literature review of corrosion in the marine environment and condition assessments including destructive and non-destructive testing. This was followed by the selection, as-built data collection, and visual condition assessment of seventeen structures of varying type, age, proximity from the sea, and marine exposure. The condition assessment focused on three defects related to corrosion namely rust staining, pop-outs (defined as small areas of spalling), and spalling. In addition, cover readings were taken of various components of the seventeen structures using a Proceq Profoscope.

Findings from the visual condition assessments confirmed significant reinforcement corrosion damage in locations on twelve of the seventeen structures. This would imply that these locations are in the propagation phase of the two-phase corrosion model, or at the end of the maintenance free period in service life models.

Average cover depths were found to be above 50 mm in most of the structural components (twenty seven out of thirty-eight), however, the minimum localised cover was found to be below 50 mm in most structural components (twenty eight out of thirty-eight). In all structures, the minimum cover was found to be below the design cover.

The following main trends were identified for structures in the port of Cape Town based on the as-built data, visual condition assessments, and cover readings:

- Distance from the sea appears to significantly affect the damage ratings of structures, particularly for those greater than 100 m from the sea, for which the ratings were low. For structures within 100 m from the sea, no trend could be established.
- Structures exposed to the South Easterly winds were found to generally have a higher damage rating, compared to those exposed to the North Westerly winds.
- Deterioration was expected to decrease with increasing minimum and average cover. No trend could be established with these parameters in isolation of others.
- Deterioration was expected to increase with age, however, overall, this trend could not be confirmed. For most of the structural components (26 of 38), particularly for structures between the age ranges of 40-to-50 and 50-to-60 an increase in damage rating was found with an increasing age. Anomalies in the data can be attributed to low cover and severe marine exposure for younger structures, and relatively mild marine exposure in older structures.

The findings from this study can be used to inform design of structures in this severe marine environment while quality controls can be put in place during construction to ensure the design cover is met, and that localised low cover areas are prevented as much as possible. In addition, the findings from the visual condition assessment can be used to inform repairs to damaged structural components. The incorporation of cover meter readings in condition assessments, and other tests such as chloride concentration tests, can be used to inform preventative repair methodologies for existing structures at a high risk of corrosion damage, but with little to no visual damage.

4.2 Recommendations

The following could not be covered within the scope of this research and have been identified as potential future opportunities for research:

- Increase in sample size of structures and components and extension to other types of structures for enhanced trend analysis and precision.
- Improvement in data analysis to compare the impacts of multiple parameters on the deterioration ratings.
- The extension of defects that were assessed in the visual inspection to include for example cracking and those defects relate to concrete quality.
- The addition of destructive testing such as core sampling and the associated chloride concentration testing, compressive strength to further confirm chloride penetration among others.
- The addition of other coastal sites around the coast of South Africa, such as the ports of Durban, Saldanha etc. where local weather conditions may vary.

The findings from this study can be used by TNPA in the port of Cape Town to inform design, construction and repairs and maintenance as follows:

- Consider the distance from the sea and predominant wind direction in the design of new structures. Here structures within 100 m from the sea and those components exposed to the predominant South Easterly wind, should consider additional design measures such as increased cover depth.
- Concrete repair should be implemented in corrosion damaged concrete to return the structures to sound condition. This maintenance should continue through regular inspections and subsequent repairs.
- In areas where no reinforcement corrosion damage is evident, and where the risk of corrosion initiation is low, the use of a surface protection system such as a coating or sealant can extend the service life of the structure.

- Make use of cover meters and other measuring devices during construction to ensure design cover is being achieved, or to timeously implement interventions if it is not.
- Perform additional testing during condition assessments such as cover readings, carbonation depth tests, and compressive strength tests to inform more comprehensive repair methodologies.

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Appendices

Appendix A: Visual condition assessments

Table A. 1 TNPA House Building

Structure name	TNPA House	Structure type	Building		
Structure Location	33°54'16.07"S 18°25'48.61"E	Built	1972		
Assessment date	12/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Exterior stairway column - West			Rust staining, pop-outs and spalling evident. Some pop outs found on the ground during the inspection.	Effects of corrosion	5
				Pop Outs	5
				Spalling	5
Exterior stairway column - East			No visual signs of rust staining, pop outs or spalling.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1



Type of member	Pictures	Description	Ratings	
Exterior wall		Pop outs and spalling evident.	Effects of corrosion	1
			Pop Outs	5
			Spalling	5
Roof beam soffit		Large cracks found on soffit of roof beam along reinforcement (4.5 mm).	Effects of corrosion	4
			Pop Outs	1
			Spalling	1

Table A. 2 B-Berth Shed




Structure name	B-Berth Shed	Structure type	Building		
Structure Location	33°54'21.68"S 18°25'45.58"E	Built	1992		
Assessment date	12/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Internal Column			Generally in a good condition. No visual evidence of spalling and rust staining.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1
Slab soffit	 		In a poor condition. Widespread spalling and severe reinforcement corrosion.	Effects of corrosion	5
				Pop Outs	5
				Spalling	5

Table A. 3 Electrical Workshop




Structure name	Electrical Workshop		Structure type	Building		
Structure Location	33°54'39.46"S 18°25'29.02"E		Built	1967		
Assessment date	12/09/2023		Exposure	XS3a		
Type of member		Pictures		Description	Ratings	
Column North				Generally in a good condition with no visible rust staining and spalling.	Effects of corrosion	1
					Pop Outs	1
					Spalling	1
Column East				Generally in a good condition with no visible rust staining and spalling.	Effects of corrosion	1
					Pop Outs	1
					Spalling	1

Table A. 4 Lighthouses Building

Structure name	Lighthouses Office Building	Structure type	Building		
Structure Location	33°54'39.38"S 18°25'24.58"E	Built	1961		
Assessment date	12/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Exterior Column			Generally in a good condition with no visual evidence of spalling or rust staining.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1
Exterior Column			Generally in a good condition with no visual evidence of spalling or rust staining.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1

Table A. 5 Unitie Building

Structure name	Unitie Building	Structure type	Building		
Structure Location	33°55'12.89"S 18°26'29.29"E	Built	1977		
Assessment date	13/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Column East			Generally in a good condition with no visible rust staining or spalling.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1
Column West			Localised areas of very severe spalling in excess of 150 mm.	Effects of corrosion	1
				Pop Outs	5
				Spalling	5




				
Type of member	Pictures	Description	Ratings	
Wall Column		Very severe spalling and corrosion of rebar reducing structural capacity of column.	Effects of corrosion	5
			Pop Outs	5
	Spalling		5	

Table A. 6 Sturrock Stairway Altars


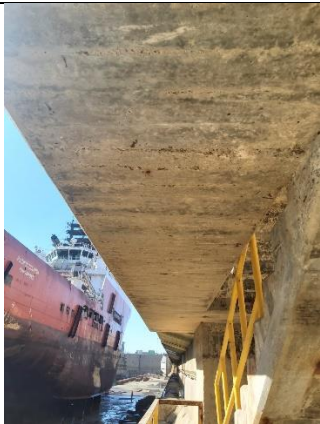
Structure name	Sturrock Dock Stairway Altars	Structure type	Stairway Altar		
Structure Location	33°55'4.73"S 18°26'40.76"E	Built	1945		
Assessment date	13/09/2023	Exposure	XS3b		
Type of member	Pictures		Description	Ratings	
Altar soffit 1			Some areas have been patch repaired however, evidence exists of very severe spalling and heavy rust staining.	Effects of corrosion	4
				Pop Outs	5
				Spalling	5
Altar soffit 2			Some areas have been patch repaired. Heavy rust staining is evident and some minor spalling and pop outs up to 10 mm.	Effects of corrosion	4
				Pop Outs	3
				Spalling	2

Table A. 7 Eastern Mole 1 Access Bridge

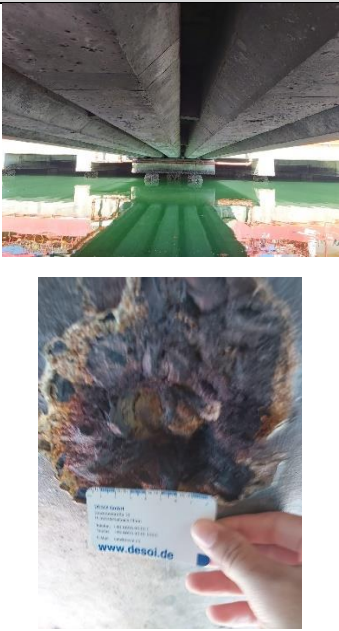

Structure name	Eastern Mole 1 Access Bridge	Structure type	Bridge		
Structure Location	33°54'34.27"S 18°26'9.82"E	Built	1965		
Assessment date	13/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Beam soffit			Generally in a good condition with some localised spalling and light rust staining.	Effects of corrosion	2
				Pop Outs	5
				Spalling	4
Handrail foundation			Very severe spalling and corroded rebar. Large area of rebar diameter corroded reducing structural capacity.	Effects of corrosion	5
				Pop Outs	5
				Spalling	5

Table A. 8 Eastern Mole 2 Catwalk Bridge


Structure name	Eastern Mole 2 Catwalk Bridge	Structure type	Bridge		
Structure Location	33°54'42.59"S 18°26'20.98"E	Built	1965		
Assessment date	13/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Beam soffit			Localised spalling and rust staining.	Effects of corrosion	2
				Pop Outs	5
				Spalling	5

Table A. 9 Eastern Mole Pump Room



Structure name	Eastern Mole Pump Room	Structure type	Building		
Structure Location	33°54'44.00"S 18°26'23.24"E	Built	2000		
Assessment date	14/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Column - East			Spalling, pop-outs, and rust staining evident. Areas larger than 150 mm.	Effects of corrosion	5
				Pop Outs	5
				Spalling	5
Column - South			Generally in a good condition. No visual evidence of rust staining and spalling.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1

Table A. 10 Tanker Basin 1 Dolphin Access Bridge


Structure name	Tanker Basin 1 Dolphin Access Bridge	Structure type	Bridge		
Structure Location	33°54'42.72"S 18°26'28.47"E	Built	1965		
Assessment date	13/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Beam soffit			<p>Very severe spalling and pop outs larger than 150 mm. Rebar exposed indicating severe corrosion and heavy rust staining.</p>	Effects of corrosion	5
				Pop Outs	5
				Spalling	5

Table A. 11 Quay 700 Prefab Substation



Structure name	Quay 700 Prefabricated Substation	Structure type	Building		
Structure Location	33°54'17.66"S 18°26'36.64"E	Built	1998		
Assessment date	12/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Exterior wall - South			Spalling up to 150 mm with some light rust staining evident. Generally in a fair condition.	Effects of corrosion	2
				Pop Outs	5
				Spalling	4
Exterior wall - North			Minor spalling with some light rust stains.	Effects of corrosion	2
				Pop Outs	1
				Spalling	2

Table A. 12 Workshop 17 Substation



Structure name	Workshop 17 Substation	Structure type	Building		
Structure Location	33°55'1.33"S 18°27'29.12"E	Built	1977		
Assessment date	14/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Column East			Generally in a good condition. No visible signs of spalling or rust staining.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1
Column South			Generally in a good condition. No visible signs of spalling or rust staining.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1

Table A. 13 T-Bridge



Structure name	T-Bridge	Structure type	Bridge		
Structure Location	33°54'59.05"S 18°27'35.70"E	Built	1976		
Assessment date	13/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Abutment			Generally in a good condition with no visible spalling and rust staining.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1
Column			Generally in a good condition with no visible spalling and rust staining.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1

Table A. 14 Marine Drive Portal Frame


Structure name	Marine Drive Portal Frame	Structure type	Portal Frame		
Structure Location	33°54'57.26"S 18°27'41.46"E	Built	1978		
Assessment date	12/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Column			Generally in a good condition with no visible signs of spalling and rust staining.	Effects of corrosion	1
				Pop Outs	1
				Spalling	1
Column			Generally in a good condition with no visible signs of spalling and rust staining.	Effects of corrosion	1
	Pop Outs	1			
	Spalling	1			

Table A. 15 Container Terminal Seawall

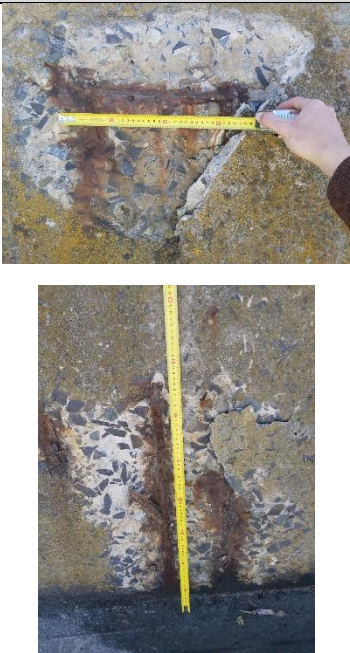
Structure name	Container Terminal Seawall	Structure type	Seawall		
Structure Location	33°54'44.82"S 18°27'33.33"E	Built	1980		
Assessment date	12/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Seawall interior			Very severe spalling and rebar corrosion with spalling in excess of 150 mm in multiple areas.	Effects of corrosion	5
				Pop Outs	5
				Spalling	5
Seawall exterior			Generally in a good condition with slight spalling and rust staining.	Effects of corrosion	2
				Pop Outs	2
				Spalling	2

Table A. 16 Harbour Signal Substation





Structure name	Harbour Signal Substation	Structure type	Building		
Structure Location	33°54'30.90"S 18°28'16.64"E	Built	1975		
Assessment date	14/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Column East			Some moderate spalling and severe pop outs evident.	Effects of corrosion	1
				Pop Outs	4
				Spalling	3
Column South			Very severe spalling evident with minor rust staining.	Effects of corrosion	2
				Pop Outs	5
				Spalling	5

Table A. 17 Salt River Seawall

Structure name	Salt River Seawall	Structure type	Seawall		
Structure Location	33°54'27.68"S 18°28'20.27"E	Built	1980		
Assessment date	14/09/2023	Exposure	XS3a		
Type of member	Pictures		Description	Ratings	
Seawall interior			Very severe spalling in excess of 150 mm.	Effects of corrosion	5
				Pop Outs	5
				Spalling	5
Seawall exterior			Spalling up to 150 mm with heavy rust staining.	Effects of corrosion	5
				Pop Outs	5
				Spalling	4

Appendix B: Cover depth readings

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Table B. 1 Cover measurements for TNPA House Building

Component:	Stairway West	Stairway East	Roof Beam East	East Wall (Roof)
Rebar Diameter:	40	40	40	32
No. Reading	Cover	Cover	Cover	Cover
1	89	62	51	57
2	89	88	54	48
3	85	99	55	51
4	79	65	53	59
5	78	62	46	59
6	74	61	53	68
7	76	93	54	68
8	83	61	52	83
9	86	92	52	81
10	79	105	56	80
11	91	101	54	72
12	90	96	52	62
13	87	88	42	71
14	79	101	55	83
15	80	103	46	68
16	84	61	57	77
17	75	79	55	72
18	85	104	53	81
19	74	103	55	88
20	82	99	63	80
21	81	80	37	81
22	77	79	56	77
23	79	67	34	69
24	76	85	43	61
25	84	62	52	66
Average	82	84	51	70
SD	5.2	16.7	6.5	10.7
Min	74	61	34	48
Max	91	105	63	88

Table B. 2 Cover measurements for B-Berth Shed

Component:	Internal Column	Internal Column	Slab Soffit	Slab Soffit
Rebar Diameter:	40	40	28	22
No. Reading	Cover	Cover	Cover	Cover
1	68	60	40	29
2	70	66	39	30
3	70	68	37	27
4	65	49	38	25
5	61	86	38	27
6	78	85	36	31
7	57	82	37	40
8	50	82	39	28
9	59	96	37	34
10	68	84	37	27
11	63	86	39	34
12	65	85	37	35
13	53	95	39	31
14	63	66	42	27
15	49	49	39	28
16	52	32	38	29
17	76	70	37	30
18	75	42	37	30
19	78	32	40	28
20	60	42	38	25
21	60	91	38	30
22	53	80	39	28
23	51	70	38	30
24	66	89	38	35
25	62	85	42	30
Average	63	71	38	30
SD	8.7	19.7	1.5	3.5
Min	49	32	36	25
Max	78	96	42	40

Table B. 3 Cover measurements for Electrical Workshop

Component:	Column North	Column East
Rebar Diameter:	14	18
No. Reading	Cover	Cover
1	63	58
2	64	43
3	63	56
4	67	63
5	70	66
6	60	65
7	55	63
8	53	58
9	36	43
10	31	57
11	52	44
12	34	60
13	30	60
14	40	43
15	41	60
16	25	44
17	40	58
18	43	61
19	38	60
20	37	60
21	36	43
22	37	53
23	51	59
24	29	58
25	50	59
Average	46	56
SD	13.2	7.6
Min	25	43
Max	70	66

Table B. 4 Cover measurements for Lighthouses Building

Component:	Exterior Column	Exterior Column
Rebar Diameter:	7	7
No. Reading	Cover	Cover
1	29	40
2	23	38
3	30	43
4	25	41
5	29	38
6	30	32
7	25	33
8	32	36
9	27	36
10	31	35
11	34	36
12	32	41
13	31	44
14	33	43
15	30	36
16	26	35
17	27	31
18	29	33
19	27	31
20	22	33
21	28	32
22	23	30
23	27	42
24	27	44
25	31	34
Average	28	37
SD	3.2	4.5
Min	22	30
Max	34	44

Table B. 5 Cover measurements for Unitie Building

Component:	East Column	West Column	Exterior wall (South)
Rebar Diameter:	25	25	18
No. Reading	Cover	Cover	Cover
1	54	49	24
2	59	29	33
3	56	37	26
4	54	34	26
5	61	49	25
6	54	40	29
7	69	44	23
8	69	52	19
9	47	28	17
10	50	55	23
11	46	24	24
12	36	28	15
13	33	28	21
14	45	24	24
15	43	27	15
16	52	62	29
17	64	22	23
18	49	22	25
19	54	32	36
20	52	30	26
21	45	22	35
22	49	30	23
23	45	59	22
24	44	29	23
25	33	25	29
Average	51	35	25
SD	9.5	12.4	5.3
Min	33	22	15
Max	69	62	36

Table B. 6 Cover measurements for Sturrock Stairway Altars

Component:	Altar 1	Altar 2
Rebar Diameter:	32	32
No. Reading	Cover	Cover
1	60	79
2	59	58
3	57	60
4	45	61
5	62	64
6	39	56
7	38	57
8	45	36
9	50	59
10	50	63
11	49	61
12	44	56
13	45	56
14	38	60
15	37	55
16	38	53
17	37	54
18	56	53
19	55	49
20	53	77
21	51	58
22	52	58
23	58	60
24	58	59
25	58	58
Average	49	58
SD	8.3	8.0
Min	37	36
Max	62	79

Table B. 7 Cover measurements for Eastern Mole 1 Access Bridge

Component:	Beam soffit	Handrail foundation
Rebar Diameter:	28	28
No. Reading	Cover	Cover
1	47	71
2	63	71
3	45	88
4	62	74
5	60	71
6	64	73
7	53	85
8	61	76
9	69	77
10	75	69
11	75	76
12	74	77
13	73	81
14	82	81
15	44	72
16	63	72
17	62	77
18	70	78
19	60	70
20	54	75
21	71	72
22	69	76
23	71	71
24	63	65
25	72	70
Average	64	75
SD	9.8	5.2
Min	44	65
Max	82	88

Table B. 8 Cover measurements for Eastern Mole 2 Catwalk Bridge

Component:	Beam Soffit
Rebar Diameter:	28
No. Reading	Cover
1	37
2	75
3	32
4	31
5	59
6	28
7	29
8	56
9	42
10	28
11	34
12	40
13	27
14	84
15	30
16	54
17	53
18	32
19	30
20	84
21	58
22	30
23	37
24	28
25	32
Average	43
SD	17.7
Min	27
Max	84

Table B. 9 Cover measurements for Eastern Mole Pump Room

Component:	Column East	Column South
Rebar Diameter:	20	32
No. Reading	Cover	Cover
1	34	52
2	31	58
3	35	55
4	31	58
5	33	50
6	28	58
7	18	57
8	24	58
9	15	48
10	42	52
11	47	49
12	43	58
13	46	59
14	45	57
15	49	58
16	48	56
17	51	60
18	32	62
19	32	59
20	31	54
21	35	55
22	32	57
23	36	58
24	35	54
25	31	54
Average	35	56
SD	9.2	3.5
Min	15	48
Max	51	62

Table B. 10 Cover measurements for Tanker Basin Dolphin Access Bridge

Component:	Beam Soffit
Rebar Diameter:	18
No. Reading	Cover
1	58
2	57
3	59
4	84
5	58
6	79
7	83
8	62
9	63
10	63
11	92
12	56
13	61
14	60
15	52
16	71
17	66
18	52
19	69
20	52
21	58
22	74
23	54
24	64
25	60
Average	64
SD	10.7
Min	52
Max	92

Table B. 11 Cover measurements for Quay 700 Prefab Substation

Component:	Wall (South)	Wall (North)
Rebar Diameter:	22	20
No. Reading	Cover	Cover
1	40	45
2	37	41
3	45	25
4	30	41
5	49	33
6	43	42
7	44	24
8	40	36
9	56	25
10	51	33
11	45	30
12	35	42
13	56	39
14	41	41
15	51	37
16	50	23
17	52	39
18	42	19
19	45	18
20	28	37
21	27	42
22	55	42
23	61	27
24	42	22
25	56	24
Average	45	33
SD	9.1	8.5
Min	27	18
Max	61	45

Table B. 12 Cover measurements for Workshop 17 Substation

Component:	Column East	Column South
Rebar Diameter:	20	20
No. Reading	Cover	Cover
1	46	36
2	49	28
3	50	29
4	54	48
5	53	35
6	62	47
7	49	41
8	45	16
9	51	45
10	49	39
11	50	23
12	46	25
13	51	47
14	50	40
15	50	32
16	47	48
17	47	32
18	50	42
19	49	41
20	55	41
21	62	48
22	45	21
23	48	35
24	49	39
25	54	25
Average	50	36
SD	4.4	9.3
Min	45	16
Max	62	48

Table B. 13 Cover measurements for T-Bridge

Component:	Column	Column	Abutment
Rebar Diameter:	40	32	25
No. Reading	Cover	Cover	Cover
1	81	61	82
2	66	65	89
3	68	62	95
4	40	60	119
5	46	62	56
6	48	57	58
7	37	60	37
8	45	62	59
9	63	69	57
10	39	42	68
11	52	55	58
12	54	57	62
13	50	62	80
14	44	75	82
15	75	76	74
16	75	73	77
17	79	55	68
18	83	53	75
19	82	69	77
20	77	63	75
21	79	71	74
22	70	77	75
23	74	51	83
24	55	52	83
25	55	72	83
Average	61	62	74
SD	15.4	8.8	15.9
Min	37	42	37
Max	83	77	119

Table B. 14 Cover measurements for Marine Drive Portal Frame

Component:	Column	Column
Rebar Diameter:	22	22
No. Reading	Cover	Cover
1	53	59
2	54	54
3	52	53
4	49	54
5	51	54
6	48	56
7	51	60
8	49	58
9	54	61
10	52	55
11	51	58
12	50	62
13	50	59
14	56	61
15	56	54
16	55	59
17	54	61
18	49	61
19	48	57
20	47	61
21	48	64
22	50	59
23	42	56
24	53	57
25	53	57
Average	51	58
SD	3.2	3.0
Min	42	53
Max	56	64

Table B. 15 Cover measurements for Container Terminal Seawall

Component:	Interior	Exterior
Rebar Diameter:	14	14
No. Reading	Cover	Cover
1	73	68
2	84	70
3	72	77
4	54	69
5	64	82
6	64	78
7	60	79
8	66	67
9	62	65
10	62	83
11	70	77
12	61	75
13	77	95
14	78	73
15	93	90
16	65	92
17	82	95
18	65	96
19	68	88
20	80	79
21	74	76
22	61	60
23	63	75
24	62	73
25	72	67
Average	69	78
SD	9.1	10.1
Min	54	60
Max	93	96

Table B. 16 Cover measurements for Harbour Signal Substation

Component:	Column East	Column South
Rebar Diameter:	36	18
No. Reading	Cover	Cover
1	56	53
2	59	53
3	60	53
4	64	55
5	54	55
6	55	55
7	52	65
8	75	50
9	63	52
10	68	53
11	59	52
12	55	50
13	59	53
14	65	55
15	59	56
16	56	68
17	56	53
18	49	52
19	58	51
20	68	51
21	78	66
22	59	50
23	65	54
24	63	54
25	62	58
Average	61	55
SD	6.7	4.8
Min	49	50
Max	78	68

Table B. 17 Cover measurements for Salt River Seawall

Component:	Interior	Exterior
Rebar Diameter:	18	32
No. Reading	Cover	Cover
1	36	45
2	33	45
3	33	46
4	30	53
5	36	41
6	34	68
7	35	55
8	32	61
9	32	52
10	30	57
11	40	46
12	36	39
13	48	57
14	44	59
15	47	66
16	45	53
17	34	51
18	33	32
19	31	46
20	33	53
21	34	68
22	35	49
23	34	52
24	33	63
25	34	58
Average	36	53
SD	5.1	9.0
Min	30	32
Max	48	68