Review of Different Practices to Reduce Reinforcement Corrosion in Concrete Structures Managed in the City of Cape Town

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Research Field:	Repair and Rehabilitation of Concrete Structures
Duration:	2016 -2017

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ABSTRACT

The repair and maintenance of aging reinforced concrete (RC) structures has become increasingly costly, especially in the Cape Peninsula. Protection and maintenance of these reinforced concrete structures against concrete deterioration and rebar corrosion has become far more important for road authorities and asset managers. City Engineers are responsible for the repair and rehabilitation of RC structures in different exposure conditions, by identifying the type of deterioration and then employing the correct concrete repair solutions or corrosion resistance measures.

This dissertation investigates the environmental exposure conditions in the Cape Peninsula that result in chloride-induced and carbonation-induced corrosion of reinforced concrete structures in the region. It includes a literature review on concrete-deterioration mechanisms and the role of aggressive elements in rebar corrosion. The literature review also considered alternative corrosion resistant rebar. There are a number of available alternatives, which include Fiber Reinforced Polymer (FRP), Hot Dip Galvanized (HDG) steel, and Stainless Steel rebar. Each alternative has advantages and disadvantages depending on design applications and durability requirements. The use of corrosion resistant rebar would increase the structure's longevity, thus providing long-term cost saving for road authorities.

In the City of Cape Town, city engineers have standardised the use of HDG rebar for repair solutions and new concrete structures. HDG improves corrosion resistance, thus making it desirable to road authorities. The HDG process has been developed in the construction industry with low production time and cost, proving favourable factors for engineers. In addition, engineers have to improve concrete quality and construction methods to protect the underlying rebar from corrosion.

On a technical level, HDG rebar use in RC structures has benefits which outweigh their cost implications. The exclusive use of HDG rebar without sound engineering judgment based on factors such as the location of the structure, distance from the coast, the structural loading conditions, and construction methods and quality standards, might not ensure better concrete durability and structural longevity. Generally, correct structural rebar design and concrete quality can eliminate the need for the use of corrosion protection methods and materials. The use of HDG is a very attractive solution for structures within 5km from the coast; otherwise normal steel is suited for most applications. Reinforced concrete members such as concrete bollards, bridge handrails and balustrades can be treated as consumables and can be replaced once steel corrosion or concrete deterioration has occurred and becomes unsightly, which would be about 20 years. This approach would be economically advantageous and politically favourable to the road authority as it creates skills and jobs by reducing initial internal and contractual costs.

To illustrate the common forms of rebar deterioration in the Cape Peninsula region, this dissertation has included five repair and rehabilitation projects completed by the City of Cape Town's Road Authority. These rehabilitation projects have been identified for concrete repair and rehabilitation works, and some of these structures have recently undergone extensive concrete rehabilitation.

City engineers are faced with many challenges that hinder service delivery, engineering processes and effectiveness. Among these is lack of staff with experience in concrete repair and asset management, and the lack of proactive maintenance tools. The lack of an adequate Bridge Management System (BMS) contributes to the inefficient allocation of resources for rehabilitation and repair projects. The Supply Chain Management System also delays the appointment of appropriate contractors due to unwieldy management systems and bureaucracy. These systemic problems are discussed to provide a better understanding of the current selection of concrete repair systems.

WITHOUT KNOWLEDGE ACTION IS USELESS. KNOWLEDGE WITHOUT ACTION IS FUTILE.

\- Abu Baker Siddique

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Glossary of Terms

1. RC –Reinforced Concrete
2. BMS –Bridge Management System
3. FRP – Fibre Reinforced Polymers
4. HDG – Hot Dip Galvanised
5. FA – Fly Ash
6. GGBS – Ground Granular Blast Slag
7. SS – Stainless Steel
8. TCT – Transport for Cape Town
9. DER rating – Degree, Extent and Relevance
10. SANS – South African National Standards
11. EuroCode – European Design Codes
12. SCM – Supply Chain Management
13. AMM – Asset Management and Maintenance

1. Introduction

1.1 Background

Concrete has been used as a construction material for centuries. Many Roman concrete structures like the Pantheon are still standing today (figure 1). As civilisation has advanced and progressed, so have the developments in concrete technology. Cities have continued to grow and develop since the development of Portland cement in the 1820s. The use of Portland cement has made modern construction more economical and standardised the world over.

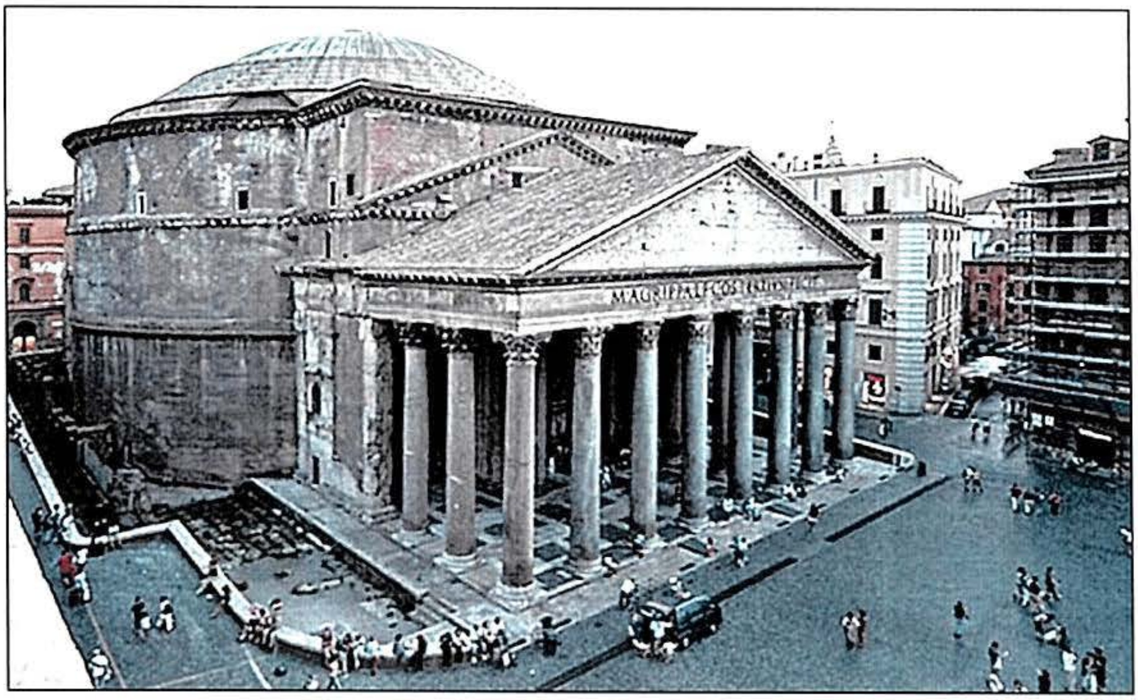


Figure 1 : The Pantheon in Rome; Roman concrete structure 2000 years after its construction. (Anon., 2016)

Concrete is the most commonly used construction material in the world, due to its considerable compressive strength and relatively low cost. However, as concrete is used for bigger structures over time, the limitations of weak tensile strength of concrete became more apparent. The areas of tension were reinforced with steel embedded in the concrete. To overcome this limitation, steel reinforcement (rebar) was introduced during the second half of the 19th century, which led to a boom in the construction of bigger, taller and larger concrete structures (SARCEA, 2011). In South Africa, reinforced concrete is extensively used for the construction of bridges, high-rise buildings, dams and other important civil infrastructure.

Reinforced concrete (RC) structures are more effective due to the interaction between the steel and the concrete, which withstands loads and forces. The embedded steel rebar has a good bond with the concrete and has good tensile strength, while still able to be bent or shaped for almost any construction application. The bond between steel and concrete has no physical or chemical disadvantages and the material properties of thermal expansion of steel and concrete are similar.

During the 1940s, the introduction of ribs in the steel rebar increased the bond strength between the concrete and the steel. The ribs are made during the rolling process which was efficient and saved costs during fabrication.

However, a major drawback of steel rebar is its tendency to revert to its natural state, a metal oxide; commonly known as rust. The exposure of reinforced concrete structures in aggressive environments reduces the service life, as detrimental elements penetrate the concrete cover and reach the surface of the underlying steel – causing varying degrees of deterioration. Carbonation-induced and chloride-induced corrosion of steel are the most common forms of concrete deterioration. This has spurred the engineering industry to improve construction methods and technologies to mitigate corrosion and its effects.

To combat reinforcement corrosion, one needs to understand concrete and steel, the process of corrosion, and investigate viable corrosion-resistant materials. The City of Cape Town's Road Authority is responsible for the maintenance and rehabilitation of RC structures. The City allocates considerable resources to repair and maintain aging RC structures.

Hot Dipped Galvanizing (HDG) is a common corrosion resistant solution used to mitigate corrosion of rebar. In Cape Town, HDG is often used in a prescriptive specification, regardless of environmental exposure conditions and durability requirements. It is becoming increasingly important for engineers to have a more holistic approach to corrosion mitigation, thus reducing lifecycle costs, increasing concrete durability and increasing knowledge of alternative methods for repair and rehabilitation.

1.2 Research motivation

This dissertation focuses on rebar corrosion of RC structures particularly in the Cape Peninsula area. It highlights the issues associated with concrete durability and deterioration, with an investigation of the current solutions and strategies.

This dissertation highlights the local authority's high expenditure on the repair and maintenance of structures due to reinforcement corrosion. City engineers are required to have a more specific standard, an appropriate repair solution and improved technical evaluation for improved selection of a cost effective and practical solution.

This dissertation also explores the general environmental exposure conditions experienced in the Cape Peninsula. The study highlights factors that influence corrosion, as well as design criteria to consider for improved concrete durability. Furthermore, the scope of this dissertation includes the process of steel rebar corrosion and the conditions which favour the corrosion process.

Engineers must make use of existing standards and practices to design concrete structures.Repair and rehabilitation of deteriorated concrete structures require proper repair strategies, and strict adherence to quality construction practices. This research identifies the role of proper design, good construction practices and the use of alternative reinforcement.

Furthermore, common failures of RC structural design and construction are examined. A lack of proper quality control and poor construction practices result in poor quality of the concrete, which significantly increases the occurrence of corrosion. City engineers who are responsible for the maintenance of concrete structures are forced to work within the confines of existing guidelines and construction practices, which are mostly outdated and require a serious review to improve current repair strategies.

The City's bridge authority must employ practical solutions and proper design standards which are specific to Southern Africa, to achieve durable, corrosion-resistant reinforced concrete structures.

1.3. Scope and Limitations

The scope of this study is limited to the Cape Peninsula region. This dissertation was approached as a non-empirical study, excluding testing of rebar corrosion and concrete condition. It is limited to existing data and standards used in the city. In addition, this investigation identifies new and old concrete bridges, retaining walls and concrete culverts, which require repairs, and are presented as case studies. These case studies required condition assessments and minor onsite measurements to illustrate the deterioration of these concrete structures.

This dissertation only focuses on concrete deterioration caused by chloride-induced corrosion and carbonation-induced corrosion, the most common deterioration processes of reinforced concrete structures in the City of Cape Town. It will exclude any in-depth explanation of sulphate attack, AAR of concrete and other deterioration that does not contribute to corrosion of the embedded rebar.

Non-destructive Test (NDT) methods and technologies that measure and detect concrete deterioration and steel corrosion will not be included. However, their use will be referenced. Furthermore, the discussion of corrosion mitigation methods and cathodic protection will be limited as these methods are not common practice in the City of Cape Town. Generally, road authorities avoid these methods, as they are heavily dependent on maintenance and highly susceptible to vandalism. The performance based concrete durability index testing approach will be referenced but the specifics of the tests will not be expounded upon.

1.4 Objectives

The objectives of this dissertation are:

i) To highlight the environmental exposure conditions in the Cape Peninsula, specifically the effects of chloride and carbonation exposure.
ii) Provide literature of alternative material selection for rebar, and review other construction factors affecting concrete quality and rate of concrete deterioration.
iii) Provide a brief literature study of chloride- and carbonation-induced steel corrosion and concrete deterioration mechanisms
iv) To investigate the durability standards and corrosion resistant solutions available to city engineers, focusing on the use of Hot Dipped Galvanised (HDG) rebar.
v) To investigate other corrosion resistant rebar alternatives and compare them to HDG used in the Cape Peninsula. This will include the construction practices that increase the quality of the concrete.
vi) To investigate specifications for good quality concrete and construction methods and the use of concrete coating systems to ensure concrete durability.

vii) To compare the economic viability of alternative corrosion resistant rebar

This dissertation identified 5 RC structures in the Cape Peninsula located in different environmental exposure conditions. These structures display varying degrees of concrete deterioration, which illustrate the effects of rebar corrosion and repair solutions.

2. Literature Review

2.1 Concrete Properties

Concrete consists of a mixture of cement, water, fine aggregate, coarse aggregate and admixtures. Each of these constituents has a certain characteristic and purpose in the concrete mix; each contributes to the strength and durability of concrete. Depending on the function of the concrete, the concrete-mix design can be modified to increase certain properties to the benefit of the concrete design. Furthermore, the science of the interaction between cement products and the hydration process must be highlighted to fully understand the strengths and the shortcomings of concrete. Normal concrete has high compressive strength and a low tensile strength. It also has a low thermal expansion, with good bond interaction with reinforced steel.

The developments of high performance concrete allows for the reduction of water results in a decrease in the amount of pores in the concrete, making it denser and thus increasing the concrete durability (Alexander , et al., 2009). Modifications and adjustments to the w/c ratios, type of cement and other modifications to the concrete mix design can increase the concrete quality.

2.2 Concrete Microstructure

The aggregate-cement paste interface, named the ITZ, is generally the weakest area in hardened concrete. This area is influenced by the w/c ratio, the amount of bleeding and the amount of fines in a concrete mix. This region is more porous and has an influence on the concrete durability.

The cement microstructure has an influence on the concrete strength and durability. The bigger the pore sizes, the weaker a concrete mix. The pores increase due to air bubbles in the cement matrix which forces an uneven distribution of cement products in the microclimates, hence creating a weaker cement lattice. The crystalline solids in a concrete mix include calcium hydroxide, this has no influence on strength but reacts with pozzolanic extenders such as fly ash. This pozzolanic reaction forms calcium silicate hydrates. The unhydrated cement particles present in the hardened cement paste (HCP) act as fine aggregates, they are inherently strong and have good bonds with the surrounding water. These particles increase the concrete strength and refine its microstructure (Perrie, 2009).

The durability of concrete is closely linked to the transport mechanics of harsh elements into and through the concrete cover. The ingress of aggressive agents from the environment increases the rate of concrete deterioration. The voids in the cement paste are about 25% of the total volume. These become interlinked and provide pathways into the concrete and eventually to the surface of the embedded steel (Rendell , et al., 2002).

2.3 Hardened Cement Paste

The hardened cement paste (HCP) consists of a ratio between hardened concrete, pore gels and capillary pores. The capillary pores are a function of the water/cement (w/c) ratio and the degree of reaction during cement hydration. The w/c ratio is used as a parameter for specifying concrete strength dependent on the cement crystal growth and affected by the degree of cement hydration. This cement reaction rate (hydration) depends on the age of the cement paste, the available water, the type of cement and curing temperature. The pores are mainly filled with water or air, which increases the porosity of the concrete (Perrie, 2009). Furthermore, these pores in hardened concrete and the fine micro cracks may cause plastic shrinkage or tension cracking which serve as pathways to the embedded steel.

2.4 Concrete Deterioration

Concrete cover protects the embedded steel and reduces rebar exposure to harsh environment. However, it is common for the concrete to have imperfections due to many external and internal factors which reduce the concrete's reliability to protect the steel. There has been a lot of research in the field of concrete deterioration in order to investigate, describe, identify and reduce these factors that influence the rate of concrete deterioration.

Generally, the presence of water surrounding the surface of the concrete and the water within the concrete substrate contributes and assists to all forms of concrete deterioration. The water serves as a transport mechanism through the concrete material matrix through cracks and the naturally occurring pores (Alexander , et al., 2009).

The effect of water in concrete deterioration is evident, especially when investigations of reinforced concrete structures deterioration in dry environments. These structures have lower rates of deterioration compared to structures in wet environments. The environment has a major influence on the rate of deterioration of reinforced concrete structures. Finally, inadequate concrete design and poor construction workmanship also has a negative effect on concrete deterioration. Coupled with the exposure conditions, defects in construction contribute to accelerated deterioration rates and eventually failure (Alexander , et al., 2009).

The visible signs of corrosion that appear on a structure's surface indicate that corrosion has been active for some time. One of the first signs of corrosion is the appearance of rust staining, coupled with minor cracks on the surface, as the embedded reinforcement steel corrodes and occupies a volume twice that of the original steel (Newman & Choo, 2003), creating even larger cracks. The expansive nature of rust creates internal pressure forces which exceed the tensile strength of the concrete resulting in cracking and spalling of the concrete cover, this type of concrete deterioration commonly affects bridge balustrades (see figure 2) due to their low concrete cover. The concrete deterioration exposes the embedded steel to the atmosphere and increases the rate of corrosion. The increased environmental exposure of the underlying reinforcement steel to chloride ions, moisture and oxygen, increases the rate of corrosion and are factors that contribute to corrosion.

Concrete deterioration can be attributed to several factors such as harsh environmental exposure conditions, incorrect concrete design and rebar arrangement, the inherent problems with cement and concrete material constitutes and concrete mix design, and finally problems with construction practices that do not meet the standard and poor quality control. These factors contribute to deterioration, either individually or together, accelerating the concrete deterioration. The concrete deterioration directly impacts on the corrosion of the underlying steel rebar contributing to accelerated deterioration mechanisms.

A major problem caused by corrosion process is the build-up of reaction products which cause the cracking, spalling and dislodging of the concrete cover. Corrosion reduces the tensile strength of rebar and a reduction of its cross sectional area. The exposed steel rebar continues to corrode and the process continues depending on the degree of environmental exposure. In pre-stressed steel, the corrosion has a more detrimental impact to the structure as a minor loss of cross sectional area would result in major failures, which occur suddenly (Alexander & Mackechnie, 2001). It is interesting to note, that once the steel is protected from exposure to moisture, air and chloride ions, the corrosion process cease.

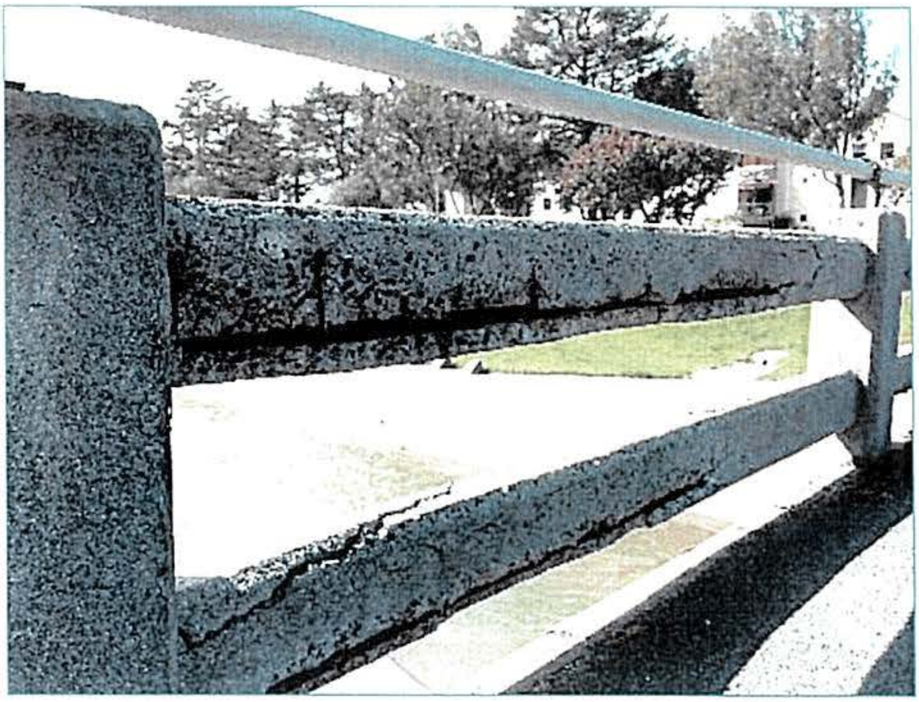


Figure 2 : Concrete deterioration and damage takes place in the form of spalling of the concrete cover and steel corrosion in Pinelands, Cape Town. The cracks normally develop above the steel and as the corrosion progresses the cover concrete is detached. In this case the deterioration was caused by low concrete cover and porosity concrete.

2.5 Environmental Influences

The Cape Peninsula climate and marine environment provide perfect conditions for chloride-induced and carbonation-induced corrosion and concrete deterioration. The rates of rebar corrosion and concrete deterioration may vary depending on location of the structure. The different elements on the structure may be exposed to different conditions. These are called microclimates or local environments.

These microclimate exposure conditions have an effect on the rate of deterioration on different parts of the same structure, (Alexander , et al., 2009). For example, the concrete elements exposed to the sun will have less saturated water whereas elements exposed to the rain with no direct sunlight will be more saturated. The elements in direct sunlight may dry out faster and the elements protected from sunlight will hold water for longer periods. Another example of the effect of microclimates can be found at the coast, where structures located in the tidal zones experience cyclic wetting and drying with higher concentrations of chlorides.

Precipitation

During winter months in the Cape, concrete structures are exposed to increase amounts of rain days; winter months are wetter for longer periods of time compared to the summer months (Ralfe, 2012). The higher rain days during winter has an associated increase of chloride ingress, thus an increase in the rate of corrosion potential. Rainfall with higher intensity and duration increases the moisture available on the surface of reinforced concrete structure and causes the concrete pores to be saturated. This water facilitates transport of CO_2, chloride ions and other detrimental elements that reach the surface of the rebar.

Temperature

Temperature has an influence on the rate of corrosion. An increase in temperature is usually accompanied with an accelerated corrosion rate, which causes rapid concrete deterioration (Alexander , et al., 2009). As the temperature rises the movement of dissolved chloride ions and salts into the concrete increases thus accelerating the rate of steel corrosion. The carbonation of concrete is also affected by temperature changes, an increase in rate of carbonation directly relates to an increase in temperature. Therefore, there is a close relation between temperature changes and the rate of corrosion.

Relative Humidity

The moisture content and relative humidity (RH) has a greater influence on concrete deterioration than temperature. The RH increases, the rate of the movement of gases and ions through the pore structure of the concrete increases and influences the water content of the pore structure. (Alexander , et al., 2009). This is evident in carbonation-induced corrosion, is dependent on the relative humidity. RH of between 50-70% is ideal for the corrosion reactions to occur (Alexander , et al., 2009). Conversely, if the concrete pore structure is completely dry, carbonation will not take place. Furthermore, a high relative humidity causes the first stage of corrosion to occur as all the corrosion promoting elements are readily available.

Chloride ions penetrate the concrete through the process of diffusion. High moisture content in the pores increases the rate of chloride penetration. The maximum penetration occurs when the pores are fully saturated. The oxygen availability also increases once concrete dries after a relative humidity at and above 70% (Alexander , et al., March 1999), creating pathways for oxygen through the concrete.

Wind

Wind provides a mechanism for the interaction of the salts in the sea water and the surface of structures. The wind carries chloride filled air onto the concrete surface. In Cape Town, the winds blow from the surrounding seas – this would normally be accompanied by precipitation. Concrete structures which are not in close proximity to coastal sea are less likely to experience chloride-induced corrosion. It is evident that an increase in wind velocity would display an increase of chlorides ions and ingress into concrete pore structures. It was reported that with an increase of wind speed of between 3.0 m/s to 7.1 m/s there was an increase of salt concentration around RC structures (Ralfe, 2012).

Carbon Dioxide and Other Air Pollutants

Most concrete structures in the Cape Peninsula are located in urban environments. Urban areas are exposed to a high level of air pollutants, these pollutants has high concentrations of CO_2. The data collected in 2007 showed that the air quality in the City of Cape Town is poor (City of Cape Town , 2007) with an increase of carbon dioxide ppm per year during a period of 2007 and 2012 as shown in Figure 3. The lowest air quality is experienced in winter, due to colder air trapped by the warmer air layer above the city. The winter months also do not experience the strong and frequent winds that remove the low air quality from the city, thus there are more pollutants available in high precipitation periods.

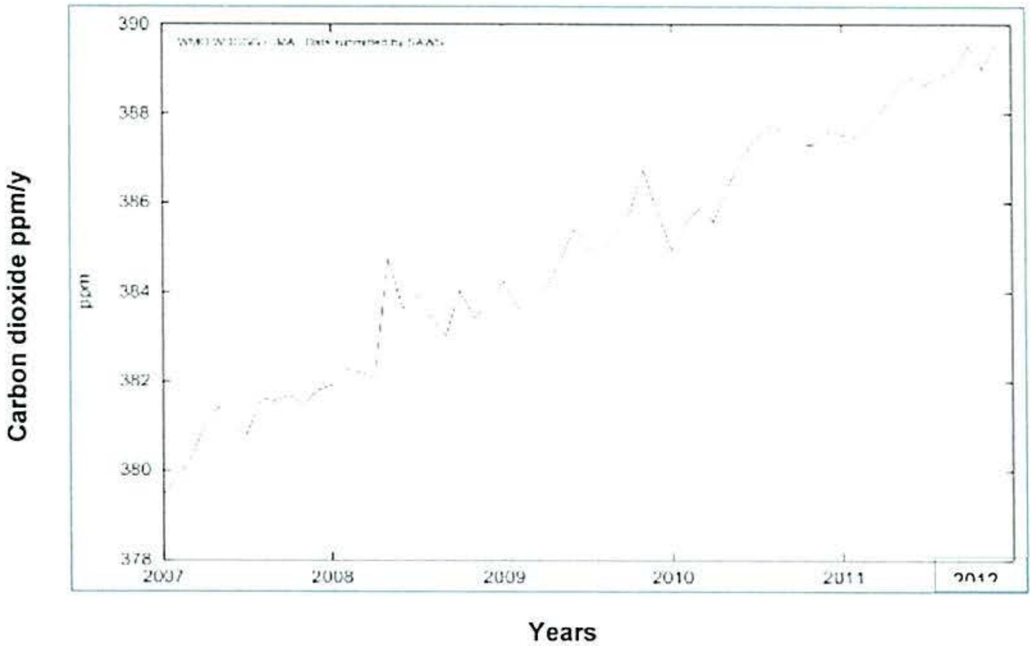


Figure 3: Carbon dioxide levels measured at Cape Point between 2007 and 2012 (World Meteorological Organisation, 2012)

Marine Environments

Concrete structures located in close proximity to the sea, are constantly exposed to moisture, wind and salts. These coastal-exposure conditions are the most severe a reinforced concrete structure may experience. Reinforced concrete structures which are located in tidal and splash zones have the added mechanism of wave action and tidal phases, exponentially increasing the hazardous mechanisms on the concrete surface and increasing the rate of concrete deterioration.

The chloride ions from the ocean are left on the concrete surface by the moisture taken up by the wind above the waves. The chloride ions are then transported through the concrete cover to the underlying rebar. The chloride ions diffuse into the concrete causing corrosion of the rebar. The corrosion products that form on the parent steel cause an increase in volume and pressure within the concrete cover.

The frequency of wetting and drying affects the moisture conditions of the concrete surface and has a direct effect on the rate of deterioration. Wetting and drying cycles of the cover concrete experience the additional factors of stress caused by soluble salts, introduced in the pores during these wetting cycles. The salt crystalizes during the dry tidal cycle and expands, causing cracking and spalling of the concrete cover (Alexander , et al., 2009). Marine environments have the additional factor of abrasion, by the sediments and biological material found in the sea water. The decay of biological materials on the concrete surfaces produces a by-product which normally contains cement hungry hydrogen sulphide (Alexander , et al., 2009). This increases the rate of concrete deterioration.

Engineers in the Cape Peninsula must understand the effects of marine environments. The extent of concrete deterioration in certain exposure zones can determine the type and degree of concrete deterioration, as shown in Figure 4 – a concrete bollard in environmental A spray zone located in Sea Point, on the Atlantic Seaboard. Marine RC structures are exposed to the following environmental exposure conditions:

A. **Spray Zone.** This is the zone which is exposed to the spray of the sea water caused by the wave and wind action. Rough sea conditions and high winds.

B. **Tidal Zones.** These zones are intermediately submerged or completely exposed to the atmosphere according to the tidal periods

C. **Atmospheric Zone**. The zone is exposed to the effect of airborne chlorides by the sea winds.

D. **Submerged zones.** These concrete regions are completely submerged in sea water the entire structure's life.

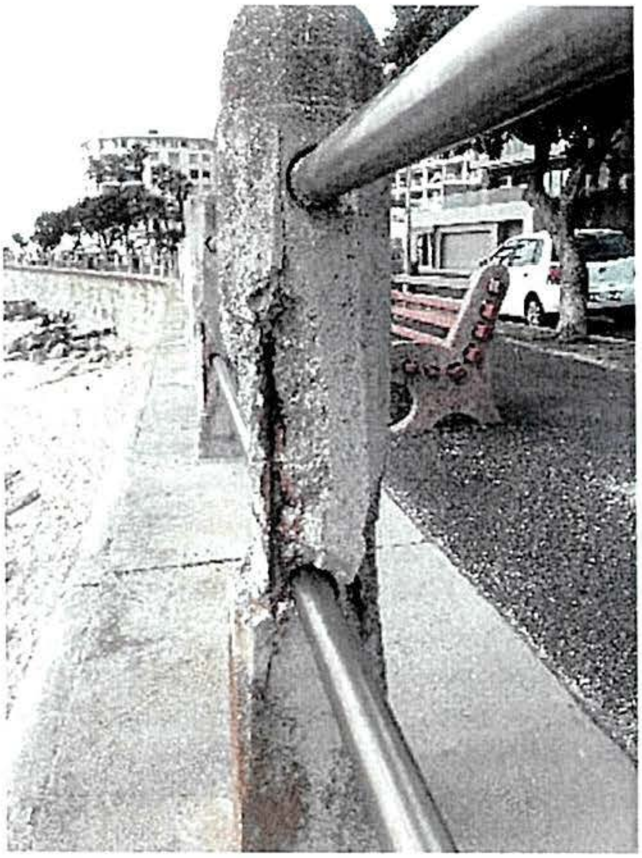


Figure 4: Concrete bollard located on the Atlantic Seaboard in a Spray Zone. The Concrete bollard shows heavy corrosion damage on the coast of Sea Point (Taken Oct 2016)

2.6 Material Characteristics

a. Permeation

Permeation is the movement of fluids through the pore structure of the concrete under a pressure gradient. Permeation in concrete occurs in a saturated or semi-saturated media (Rendell , et al., 2002). An increase of voids in the concrete pore structure causes an increase of permeability. The moisture conditions and type of fluid have an influence on the permeation of concrete and ultimately its durability. The simplest example of permeation is the flow of water through a saturated sample.

A measure of permeability helps to predict the carbonation rate of concrete (Alexander , et al., 2009). The performance-based test used is the oxygen permeability test which measures the amount pressure decay of oxygen passing through a section of concrete.

b. Capillary Absorption

Capillary absorption is a process; water is drawn into the concrete via a network of fine "tubes" by capillary action. This process is dependent on the pore geometry and degree of saturation of the concrete (Alexander , et al., 2009). This denotes to greater absorption of water, the larger capillaries are present as well as an increase of the degree of interconnectivity. The movement of water through the concrete via capillary action is referred to as the sorptivity. This flow mechanism is complex, as evaporation may occur and the water may change the "network" within the concrete, which may vary over time, but more likely increase the capillary flow (Rendell , et al., 2002).

c. Diffusion

Diffusion is the process by which liquid, gas and ions move through porous concrete from a high concentration to a lower concentration. This is a mechanism by which ions, moisture and air are transported into and through the concrete substrate. Salts are absorbed on the surface of the concrete creating a region of higher concentration. These salts are diffused through the concrete substrate to a lower concentration (Rendell , et al., 2002). Environmental factors like temperature and atmospheric chloride content, coupled with the concrete's moisture content and the concrete porosity, have an effect on the rate of diffusion. In concrete, water vapour diffusion may occur at about 70% relative humidity and above that diffusion ceases (Rendell , et al., 2002).

For the purpose of this research, the chloride ions, the concrete material properties and the transportation of chloride ions diffusion are the main factors to consider in investigating the depassivation of embedded steel and the initiation of steel corrosion (Alexander , et al., 2009). Furthermore, the factors which induce cracking and voids increase the degree of diffusion and decrease the concrete durability. Over time the diffusion of water and chlorides ions may change the concrete chemically and structurally (Rendell , et al., 2002), affecting the concrete durability.

2.7 Concrete Durability

Concrete durability has become an increasingly important fact in modern concrete technology. The lack of concrete durability is a major problem for cities, where the majority of existing infrastructure are reaching the end of their design life. Concrete durability can be defined as "the ability of a structure or component to withstand the design environment over the design life, without undue loss of serviceability or the need for major repair" (Alexander , et al., 2009).

Concrete durability should be a major consideration for structural engineers, especially during the design of a structure and by ensuring standards during the construction phase. Durability is affected by both internal and external factors, which influences the concrete's resistance to deterioration. Knowing the concrete material properties, the design standards, the environmental exposure conditions and the purpose of the structure, these are all factors which influence the rate of deterioration of reinforced concrete structures. Understanding how concrete deteriorates and the relationship between the concrete and the underlying reinforcement is to be considered during the design and construction phases.

The City of Cape Town employs the use of an asset management tool called the BMS (Bridge Management System), the BMS provides information to engineers allowing them to make informed decisions to determine time between rehabilitation works to be performed on reinforced concrete structures, before any major risk to safety. This is achieved by the use of structural condition assessments and monitoring of key factors based on the structure's durability index. This durability index is determined by the following three methods: the oxygen permeability test, the water sorptivity test and the chloride conductivity test. This provides an index of the quality of the concrete cover. These tests highlight environmental exposure conditions, concrete quality, material selection factors and overall concrete durability. The exact procedures and outcomes of these tests are explained in depth in the Concrete Durability Index Testing Manual (Alexander , et al., March 1999). Concrete durability must be effectively incorporated in concrete design and resources must be made available for durability indexes to characterise the effects of mix proportions, material selection, placing, compaction and curing techniques, as well as environment influences. The concrete durability is influenced by the factors discussed below.

Concrete Quality

The quality and permeability of the concrete cover is a critical factor to be considered when corrosion control and the protection of embedded rebar. The permeability of concrete is influenced by the quality of a concrete mix, the degree of compaction, curing and the depth of cover to the reinforcement.

Hardened concrete is composed of hydrated cement products, capillary pores, gel pores and unhydrated cement. The voids in the concrete are defined by their size. These voids are partially filled with air or water. The number of voids is influenced by the water/cement ratio and the degree of curing. With proper design, good workmanship and adequate curing methods allows for full cement hydration, hence an increase of the concrete quality. With the increase of concrete quality, the concrete durability increases by decreasing the number and size of existing voids.

Concrete Mix Design

Concrete mixes used in the past were highly permeable which allowed for the ingress of moisture, chloride ions and carbon dioxide. The majority of reinforced concrete structures experience durability issues and deteriorated at a faster rate due to poor concrete quality and higher permeability. The development of high performance cement (HPC) and other concrete technologies has resulted in improvements of concrete mix designs with a higher degree of strength and improved durability.

The common misbelief of insuring higher concrete durability is the selection of cement type for a high strength concrete. The correct application concrete strength and durability would be proper concrete mix design with good curing methods to increase concrete quality (Alexander , et al., 2009). In South Africa, the use of cement complies with SANS 50197-1 (Addis, 2008). The "common" cements used for civil engineering applications is CEM I, CEM I is a higher strength cement and is the most common cement used in construction projects in the Cape Peninsula. Furthermore, the use of 'pure' cement types isn't economically viable, thus the addition of cement extenders is a common practice.

The use of cement extenders, like fly ash and GGBS, refines the concrete lattice, in particular the Inter Transitional Zone (ITZ) aggregate-cement interface and acts as fine filler, thus making the concrete structure denser, increasing strength and consequently increasing the concrete's durability. This can only be achieved correctly when the extenders are added at the batching plant and proper curing methods are employed on site.

Water/Cement Ratio

The water/cement (W/C) ratio of a concrete mix is a factor, which affects the concrete durability. The rate of diffusion of chloride ions into concrete is affected by the water/cement (w/c) ratio. The w/c ratio contributes to the amount of concrete moisture remaining after cement hydration has been completed. An increase of the w/c ratio results in an increased number of pores and interconnected pathways within the concrete substrate. The amount of water in the initial concrete mix indicates the time required blocking interconnecting pores; sealing the passageways in the concrete from the surrounding environments. Table 1 illustrates the relationship between the w/c ratio and the period it takes for the ingress of aggressive elements to pass through the concrete. During cement hydration, the cement paste structure develops capillaries. These capillaries increase with the increase of w/c ratio above 0, 4 and the quantity of the unhydrated cement components increases below 0, 4 w/c ratio (Rendell , et al., 2002).

Table 1: Relationship between the water/cement ratios
And the time taken to disrupt the capillaries (Rendell , et al., 2002)

Water/cement ratio	**Time to block the pore structure**
0.40	3 days
0.45	7 days
0.50	14 days
0.55	6 months
0.60	1 year
0.70	Never

The concrete cover protects the underlying steel from the ingress of chloride ions through the concrete. The water/cement ratio is critical to determine a proper concrete mix design. A decrease of w/c ratio would have a direct impact on the concrete durability and strength (Alexander , et al., 2009). A low water/cement ratio results in a closed pore structure with a very high resistance to carbonation penetration (Costa & Appleton, 2001), this is the diffusion of CO_2 through the concrete. Conversely, it has been shown that the increase of the water/cement ratio would increase the depth of the carbonation penetration as the concrete is more porous (Rendell , et al., 2002).

Curing

Concrete curing insures enough water is available to be consumed by cement hydration reactions; this allows for the concrete to be hardened sufficiently with minimal pores. Curing creates near perfect conditions for cement hydration in the cement paste, which reduces void/pore sizes. Therefore, correct curing methods decrease the permeation of aggressive agents dissolved in water, and this water infiltrates into the concrete's surface zone.

When the concrete is dried prematurely, causing low water availability this ceases the hydration process. As the hydration ceases, the pore structure will remain open, forming voids, as well as an increased amount of unhydrated cement particles.

Improper curing causes the concrete surface layer to become less dense making the concrete more susceptible to carbonation (Rendell , et al., 2002). Poor curing conditions, which includes insufficient curing duration and the incorrect use of cement extenders, contributes to a weaker cement paste and increased porosity.

The types of curing methods may differ for different concrete elements or structures. Water curing may not be possible on all surfaces, where certain parts of the structure are inaccessible. In principle, it is good practice to cure concrete for at least five days with water at a temperature of 22°C (SANS 10100-2). Further details on curing methods and duration are provided in Fulton's Concrete Technology (Kellerman, 2009) and in the Standard Specification for Road and Bridge Works for state road authorities COLTO 1998 edition, which is still used by City engineers.

Cover to Reinforcement

The concrete cover depth is the distance from the surface of concrete to the embedded reinforcement. The cover assists with the protection of the steel against corrosion (Alexander , et al., 2009). When the concrete cover is compromised it allows for the ingress and penetration of harmful chemicals to reach the surface of the rebar.

Engineers have to consider all factors that may comprise the condition of the concrete; reinforcement detail drawings will help to insure the rebar are fixed and bent with better accuracy, by insuring on site fixing adheres to the technical drawings. Engineers should place emphasis on proper site supervision and regular checking of the cover depths. The use of cover blocks, which are of good quality and density, should be placed on the reinforcement cage at a spacing that would support the reinforcement during the weight of concrete placing. It is important to allow for tolerances in the dimensions on concrete elements and allowances for the bending of the rebar. The concrete cover can be lost if there are insufficient tolerances of the formwork assembly, as well as the bending of the steel, are at their extremes (Theodosiou, 2009). The reinforcing should be detailed to allow relative small movement between the steel rebar and the formwork, to maintain the required concrete cover depths despite these variations.

Another aspect to be considered in the rebar arrangement detailing, is the specification for spliced and rebar overlaps. Rebar overlaps should be in a parallel plane to the surface of the concrete. The incorrect placement of rebar should be corrected to be at right-angles to one another or the rebar should overlap without compromising concrete cover, this is usually by the diameter of the bar (Alexander , et al., 2009).

Concrete Design Detailing

Reinforcement can only be effective when there is sufficient concrete cover, the rebar has to be properly positioned, correctly cut, bent and securely tied to prevent movement during construction and concrete placement (Theodosiou, 2009). The use of spacers to maintain concrete cover depth along with suitable supports will prevent any construction defects.

Concrete cover and proper detailing of the structure can improve durability by:

1. The use of small-diameter bars that are well distributed to assist in reducing crack widths.
2. Avoiding changes of the ratios between concrete cover and steel reinforcement's splice lengths.
3. The designer should ensure sufficient space for compaction around the steel, especially where the steel is spliced.
4. Contact between dissimilar metals should be avoided.
5. Proper designs and provision of movement joints and water drainage facilities in the structure.

The design of certain elements on a structure can prove impractical to execute on site. An increase in difficulty may negatively impact on the quality of workmanship, especially with no proper site supervision. Design engineers should use technical insight to avoid difficulties on site by adhering to standard construction methods.

Construction defects

Concrete structures are subject to many environmental and service life variables, changes to design and unforeseen site conditions. The quality of work and the standard of work executed onsite affect the final concrete durability. It is clear that the durability of reinforced concrete structures and the corrosion rate is influenced by the quality of the concrete cover and the position of steel reinforcement.

To ensure better concrete quality the following practices should be conducted on site. Construction workers should take care during the installation and erection of formwork, supporting structures and simple site logistics; this includes the construction methods that insure proper concrete compaction. The initial design and construction of formwork must be monitored during the construction phase. To eliminate construction defects engineers must use standardised construction methods carried out by skilled workers.

Internal Cracking

To a certain degree all concrete has cracks; this internal cracking occurs once the concrete starts to lose water, due to its environmental exposure, known as drying shrinkage. The increase of temperature of the concrete due to the cement hydration process contributes to the loss of water caused by thermal shrinkage; which also results in internal cracking.

The interface between the cement paste and the aggregates is termed as the Interfacial Transition Zone (ITZ) (Rendell , et al., 2002). These zones are generally more porous and weaker than the surrounding concrete, internal cracking are most likely to occur in these regions, allowing for the movement of water between the aggregate and the cement paste.

Internal cracking caused by concrete shrinkage increases permeability, and affects the concrete durability. The use of high performance concretes can increase the effects of the shrinkage processes.

High performance concrete has higher cement content and the lower the w/c cement ratio, contributes to an improved concrete strength, however this type of concrete would be more prone to micro-cracking thus decreasing its durability.

Concrete Cracks

I. Plastic Shrinkage Cracks

Shrinkage cracks forms after the first few hours after concrete placement when the concrete has not gained adequate strength. The water that is freely available in the concrete mix is able to move up to the surface of the concrete. This upward movement of water and air is called bleeding. This bleed water evaporates at the concrete surface. The loss of volume at the surface causes concrete to shrink. However, the underlying layer of concrete restrains the shrinkage resulting in internal stresses within the concrete substrate that results in cracking.

While the concrete is in a plastic state, cracks form in the cement paste and around the aggregates. Plastic shrinkage cracks are experienced in both unreinforced and reinforced concrete. Plastic shrinkage cracks are normally 1-2 mm wide, 300-500 mm long and 20-50 mm deep (The Concrete Society , 2000).

ii. Plastic Settlement Cracks

The plastic shrinkage allows for the movement of water, and may also cause the movement of denser materials such as aggregates, to move down and settle. Furthermore, the settlement in the concrete may be obstructed by the first level of reinforcement. This causes parallel cracks along the line of the rebar below. The settlement may be visible as undulations of the concrete surface, where the higher points are over the steel bars.

Plastic settlement cracks may occur on vertical faces as well as along the reinforcement bars, where the flow of concrete may be hindered. The formwork may cause the same problems at early stages of concrete hydration. These cracks are usually 1 mm wide at the surface and usually run from the surface to the rebar below.

2.8 Reinforcement deterioration and corrosion

2.8.1 Deterioration of Embedded Steel in Concrete

Reinforced concrete structures uses steel rebar to provide tensile strength in concrete elements. Steel rebar is heavily processed from iron ore, and requires a lot of energy to manufacture. Thus steel has the natural inclination to revert back to its original state, this process is called corrosion. The concrete and cover depth provides rebar protection from exposure to the air and moisture.

Steel reinforcement is inherently protected from corrosion once fresh concrete is place. The concrete provides an alkaline environment around rebar. Cement hydration forms calcium hydroxide in the pore solution at a pH of 12.5; in addition, the presence of sodium and potassium in the cement increases the alkalinity (Alexander , et al., 2009). This alkaline environment provides a stable oxide layer on the surface of the steel; this is called the passive layer and protects the steel from corrosion. Essentially, it is this breakdown of the passive layer that initiates corrosion. This process is called depassivation and is caused by chlorides and carbonates reaching the surface of the steel, and changing the pH of the surrounding concrete.

Chloride-induced and carbonation-induced corrosion are the most common mechanisms of rebar deterioration in concrete structures. In Cape Town, the majority of concrete repairs on structures are due to their proximity to the sea and level of exposure to the harsh environment. The deterioration mechanisms attack the passive protection layer around the steel, as carbonates and chlorides penetrate to the rebar surface. Steel rebar only corrodes when moisture, oxygen and chlorides are available creating ideal conditions for corrosion.

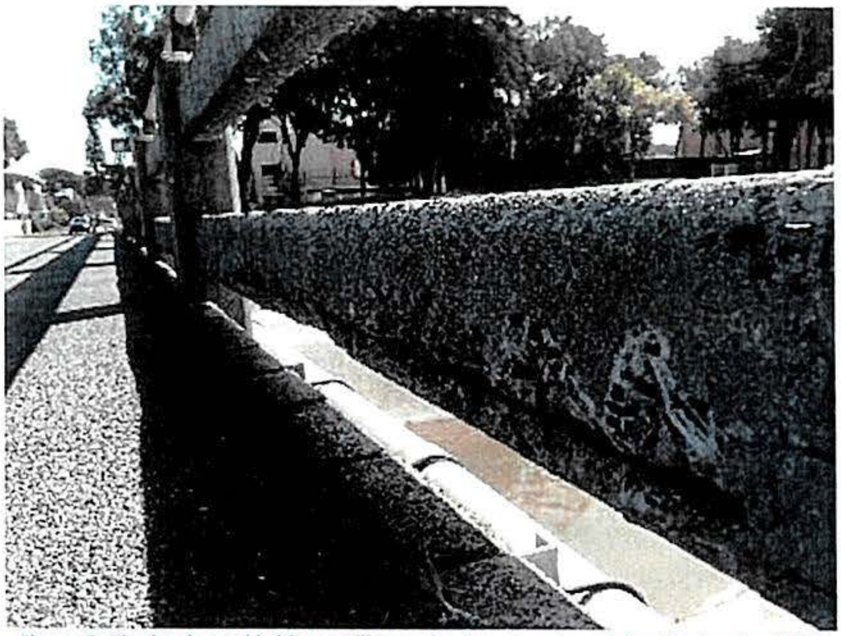


Figure 5. Pinelands road bridge spalling and rebar corrosion. Taken September 2016

In this section, the common corrosion processes caused by chlorides and carbonation are discussed, focusing on the defects caused by and the by-products of the corrosion process. A brief discussion of the electrochemical process and the conditions that facilitate to a perfect corrosive environment. Finally, a brief discussion of concrete deterioration, due to rebar corrosion and the effects on the concrete surrounding the steel are highlighted.

2.8.2 Fundamentals of Chloride Induced Corrosion

Steel corrosion in concrete

There are many research papers on the complex subject of rebar corrosion (Rendell , et al., 2002) & (Alexander , et al., 2009) discussing the corrosion processes and its by-products. Steel corrosion is an electrochemical process, where the steel degrades at the anode and the oxygen is reduced at the cathode. This corrosion process is the flow of electrons from the anode to the cathodic region on the steel surface. This electron flow is facilitated by moisture and oxygen at the steel surface. The concrete pore solution is highly alkaline; it acts as an electrolyte, while the steel bar serves as the metallic path between the anode and the cathode on the steel surface.

The corrosion process produces iron oxide, also known as rust, rust forms on the cathodic region of the parent steel. The rust takes up a larger volume than the parent steel resulting in expansive forces in the surrounding concrete which subsequently causes cracking and spalling of the cover.

Steel Passivation

The steel rebar is protected from corrosion due to the high alkalinity of the pore water surrounding the embedded steel. This high alkalinity of the pore water is due to $Ca(OH)_2$ formed during cement hydration. The pore water has a high pH, providing an ideal environment for the formation of a protective film consisting of gamma ferric oxide on the steel surface (Alexander , et al., 2009). Good quality concrete with adequate cover depth prevents ingress of aggressive agents; these agents are capable of destroying the protective film around the steel. This process is called depassivation; Figure 6 shows how much exposure steel can be with enough water, oxygen and chloride ions which creates ideal corrosion conditions.

The depassivation occurs due to these two main processes:

i) The penetration of the carbonation front reaches the steel interface, resulting in a lower pH. This ingress of aggressive ionic solution through to the surface of the embedded steel, results in the disruption of the passivating mechanism.

ii) Ingress of chloride ions in marine environments (Alexander , et al., 2009), when there are sufficient chloride ions beyond the chloride threshold, depassivation occurs. When the depassivation of the steel occurs and corrosion commences, the product of corrosion (rust) cause stresses within the concrete which result in concrete cracking that further exposes the steel to deterioration.

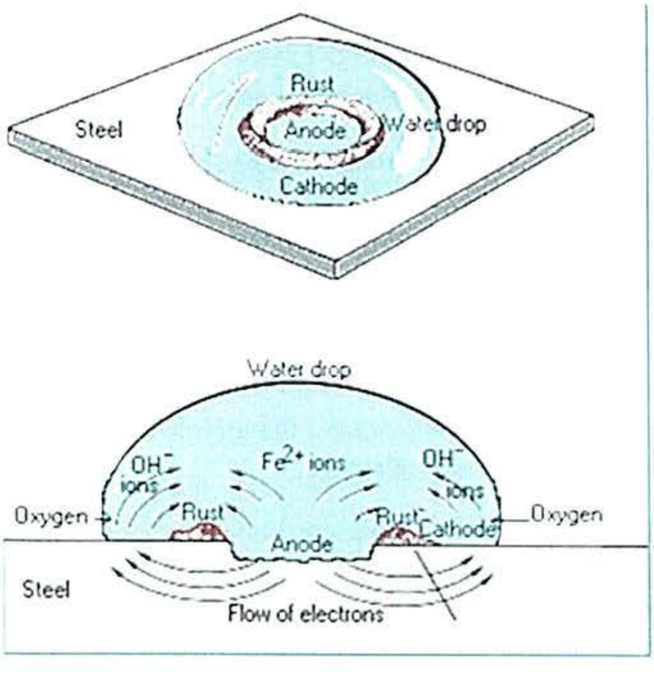


Figure 6: Illustration of the corrosion process on a steel plate and a water droplet (HDGASA, 2015).

Chloride ion Diffusion

Chloride is one of the most detrimental chemicals to reinforced concrete structures and has been extensively documented to be the most common cause of steel corrosion. It is well understood that the chloride ions penetrate the concrete cover and reaches the passive layer causing depassivation and creating ideal conditions for steel corrosion.

The ingress of chloride ions into concrete is a major problem in coastal regions. These structures are constantly exposed to chloride ions and experiences corrosion at an early age. The penetration of seawater into concrete to the surface of the embedded steel creates perfect conditions to initiate steel corrosion. Seawater contains a high concentration of dissolved chlorides that penetrate through the concrete via cracks to the steel. Steel corrosion occurs once there is sufficient water and oxygen, and once the chloride threshold limit has reached around the steel.

The chloride ingress and rate of diffusion into the concrete is related to the moisture content in the pore structure. The higher the moisture content the higher the rate of chloride ion diffusion. Reinforced concrete structures located in splash zones are characterised by cyclic wetting and drying. This allows for the expanding crystallization of salts which eventually causes cracking within the concrete pore structure. The chloride penetration in concrete is more rapid in relatively dry environments where the concrete is occasionally wetted (The Concrete Society , 2000).

2.8.3 Chloride Corrosion

The relationship between chloride and corrosion is widespread and complex (The Concrete Society , 2000). The passive layer, in uncontaminated concrete, is continuously being broken down and reformed. Once the chloride threshold is reached, this passive layer is impacted on reducing the re-establishment of the passive layer. This chloride threshold is reached when the chlorides measured by weight of chlorides in the cement binder by mass. At low levels, the chlorides would slowly break down the passivating layer and eventually an increased amount of chlorides would increase the rate of steel rebar passivating layer breakdown. Once the steel reinstatement of the passive layer cannot "selfheal" the passive layer fast enough (The Concrete Society , 2000) corrosion can initiate.

Corrosion occurs in the presence of oxygen, moisture and chlorides close to the steel surface. The available chlorides are either provided internally, from the chlorides in the mixing water, or externally from chloride ingress into the concrete surface. External chlorides are absorbed or diffused into concrete from the surrounding chloride rich environment.

Once the concentration of the chlorides reaches a limit of 0, 4% by weight of cement, depassivation occurs and then corrosion commences (Alexander , et al., 2009). The pH may not be lowered by the chloride ions, but rather the chlorides inhibit the passive maintenance of this protective oxide layer (The Concrete Society , 2000), the complete depassivation occurs at this stage.

Corrosion process is normally in an aerobic environment and leads to formation of expansive rust causing cracks and spalling of the surrounding concrete. However, sometimes corrosion may occur in anaerobic environments. In this case, a macro cell is formed where a cathodic reaction takes place in the aerated zone of the structure and the anodic (corroded) process occurs at the depassivated area. The corrosion product formed is magnetite, which is not expansive (Costa & Appleton, 2001).

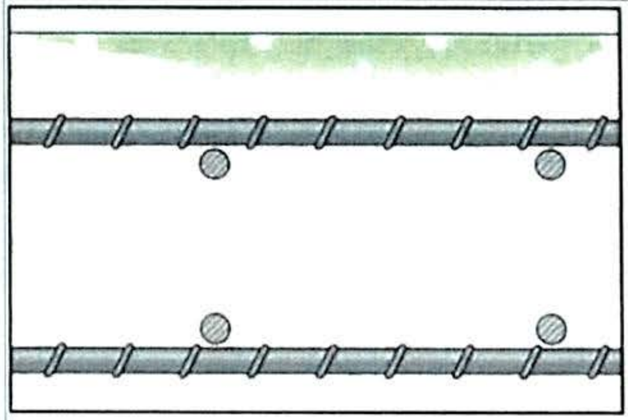
The Three stages of rebar corrosion may be defined by the environmental conditions. Figure 7 illustrates the different stages from the onset of chloride penetration to the surface of the rebar and finally, the effects caused by corrosion by-products. The stages of corrosion are listed below:

Stage 1: Initiation – The concentration of chloride is not high enough to initiate any chemical reactions or the reaction rate is very slow. There is no damage of the steel and this condition may be maintained as long as there is sufficient moisture and high chloride concentrations.

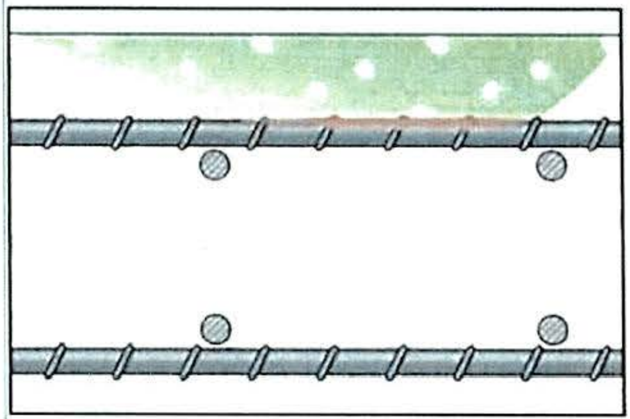
Stage 2: Propagation – The chemical reaction starts and continues, the corrosion and the damage become visible. The increase of aggressive ions coupled with the increase of oxygen and moisture would increase the extent of corrosion.

Stage 3: Deterioration – The rapid breakdown of the steel occurs. The steel continues to oxidise and loose cross sectional area .The steel now has a lower serviceable state.

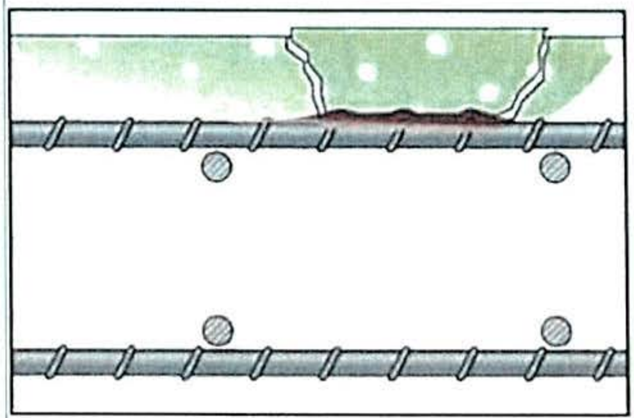
Sometimes common repair solutions may be detrimental to the concrete. For example, concrete patch repairs that are carried out on damaged concrete sections. When this repair is done in chloride-contaminated concrete, the repaired concrete area has an effect on the electric potential difference of the whole system. The replaced steel acts as a non-corroding cathode, while the existing steel region becomes a corroding anode. This results in an accelerated corrosion cell near the patched repair. This is called the halo effect, where a ring anode becomes visible around the effected region and spalling is likely to occur around the newly patched area. This corrosion damage can be visible as soon as two years after repairs are completed (MD Pritzel 2008).



a. Chloride penetration



b. Chloride reaches reinforcing bar



c. Reinforcing bar corrosion spalls concrete

Figure 7: Chloride corrosion is characterized, as the corrosion damage which is localised which also reduces the steel cross section. (The Concrete Society , 2000)

2.8.3 Corrosion process

The steel passive layer is disrupted and the corrosion is active, the reaction is the same for both forms of corrosion, namely caused by chloride or carbonation attack (Alexander & Mackechnie, 2001). The steel dissolves as it gives up electrons at the anode and the positive ions are formed at the cathode. Figure 8 illustrates localised reactions and the interaction of the cathode and anode regions on steel.

The anode regions have a loss of steel through oxidisation during the corrosion process.

Anodic Reaction: $\mathbf{2Fe > 2Fe^{2+} + 4e^{-}}$

Electrons are used up at the cathodic site as the water and oxygen are reduced to hydroxyl ions.

Cathodic Reaction: $\mathbf{O_2 + 4H^{+} + 4e^{-} > 2h_2O}$

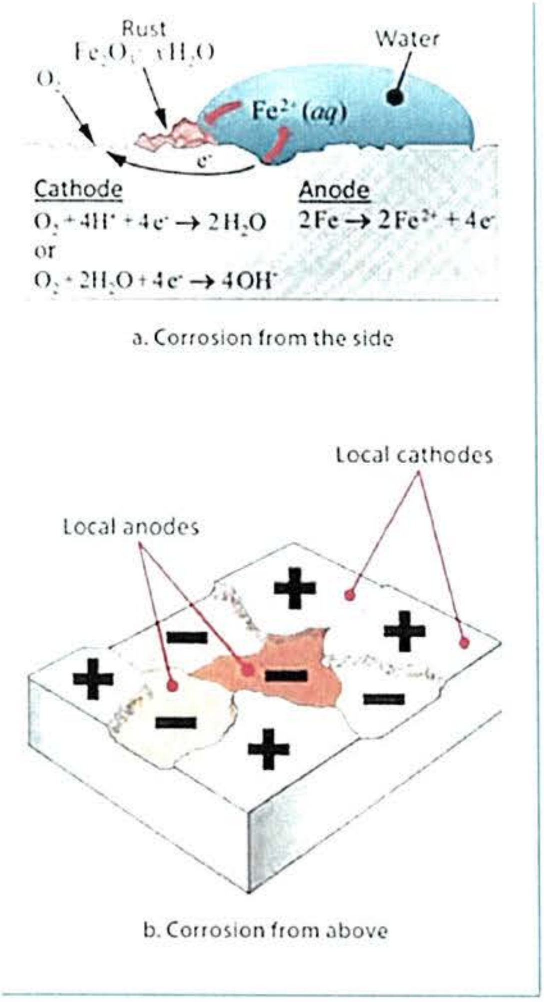


Figure 8: Corrosion of steel – the anode and the cathode zones on embedded Reinforcement (NACE The Corrosion Society , 2012)

Whether the deterioration is due to chloride attack or carbonation, the process and corrosion by-products remain the same. The steel has localised anodes and cathodes as shown in Figure 8. Steel dissolves into solution by giving up electrons at the anode. The resulting rust caused by the hydrated iron oxides and is expansive in nature, taking up a volume 10 times more than the original black steel. The type of rust formed at the steel depends on environmental conditions (Alexander & Mackechnie, 2001). The corrosion product of rust is usually red or brown and forms under high oxygen concentrations. This type of rust is usually soft and breaks off from the parent steel. Black rust forms under low oxygen concentrations and is dense and forms a harder layer around the parent steel.

2.8.4 Carbonation

Carbonation occurs as soon as the carbonation front moving through the concrete substrate reaches the steel surface. The carbonation causes a drop in pH around the steel from 13 to 9. Carbonation-induced corrosion affects larger areas of the bars. The process of carbonation requires the presence of water and carbon dioxide in the pore structure. Normally, carbonation will not occur in saturated concrete. Carbonated concrete has insufficient hydroxyl ions available to repair pits in the passive film around the rebar. Carbonation occurs when carbon dioxide from the atmosphere penetrates the concrete and reacts with hydroxides, and reacts with carbon dioxide to form calcium carbonate:

$$Ca\,(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$

And.

$$Ca\,(OH)^2 + CO_2 \rightarrow CaCO_3 + H_2O$$

When the carbonation front goes through the concrete cover and beyond the reinforcing steel it becomes a problem. The calcium carbonate reduces the pH of the pore solution to as low as 8.5. The lower pH reduces the effectiveness of the passive film as it is no longer stable. The protective layer can no longer protect the steel against moisture and oxygen. The carbonation front in high quality concrete it is estimated to pass through the concrete at a rate of 1,0mm per year (Alexander , et al., 2009).

The carbonation rate is increased in concrete with a high water/cement ratio, low cement content, short curing period, low strength, and high porosity. Relative humidity also has an influence on the rate of carbonation; relative humidity between 50% and 75% has the highest rate of carbonation. When the relative humidity is below 25 % the carbonation rate is considered to be insignificant. Upwards from 75% relative humidity, the pores are filled with moisture and pathways for carbon dioxide and the carbonation front progression cannot occur (ACI 201 1992). Therefore, the conditions which favours carbonation are found at the relative humidity (RH) of between 50 -60 %. The carbon dioxide reacts with the water in the pores and forms carbonic acid, the carbonic acid reacts with portlandite and calcium silicate to form calcium carbonate. The reactions are as follows:

$$CO_2 + H_2O \rightarrow H_2CO_3$$

$$H_2CO_3 + CaO.\ SiO_2.\ H2O \rightarrow CaCO_3 + SiO_2 + 2H_2O$$

and

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO3 + 2H2O$$

Carbonation-induced corrosion is detected on the concrete elements that are shaded from the sun and exposed to constant moisture. Concrete elements with low concrete cover to the reinforcement are highly susceptible to carbonation-induced corrosion.

The rate of carbonation occurs as a function of concrete quality. This quality is dependent on the w/c ratio and the degree of compaction. In general, the rate of carbonation is inversely proportional to the square root of the age of the structure (Alexander , et al., March 1999). The rate of carbonation occurs more rapidly, due to micro-cracking or excessive porosity. The moisture of the concrete in wet and dry cycle conditions has an effect on the rate of carbonation. Carbonation does not occur in wet concrete. When such concrete dries out significantly and to the depth of the carbonated front, carbonation continues further into the concrete cover (Alexander , et al., 2009).

2.8.5 Combination of Chloride and Carbonation Corrosion

Concrete carbonation and chloride penetration can act together increasing the rate of corrosion. The long-term penetration of these aggressive agents has been evaluated (Costa & Appleton, 2001) and researched. The study illustrated the increased rate of corrosion of reinforced concrete when exposed to marine environments. The concrete that has no direct contact with sea water, but are in the spray zones and the atmospheric zones are exposed to airborne chlorides carried by wind and wave actions. The interaction between the chlorides and the carbonation leads to increased rates of steel corrosion, causing instability of the chloroaluminates in the presence of chlorides in the pore structure.

The time from, the initiation of corrosion and when damage starts, is the main difference between chloride-induced and carbonation-induced (The Concrete Society , 2000). Carbonation occurs rapidly in relatively high humidity between 60-70%. However the rate of corrosion is slower at humidity levels above 70%. At higher relative humidity, the chlorides present in concrete are rapidly transported through the substrate, during wetting and drying periods. Hence making marine conditions ideal for rebar corrosion. In addition, the concrete, which is undergoing carbonation may have an increase of chloride concentration ahead of the carbonation front. These chlorides are released when C_3A (calcium chloroaluminate) and portandite reacts with CO2 (Rendell , et al., 2002).

The chloride penetration increases when the concrete is carbonated (Alexander , et al., 2009). This is due to carbonation reducing the chloride binding capacity of the concrete, increasing the amount of free chloride ions available for depassivation. Chloride ions on the ferrous chloride are oxidised to ferric chloride and precipitates as ferric hydroxide. The rate of corrosion is determined by the oxygen available; with no oxygen available corrosion cannot occur (Alexander , et al., 2009).

2.8.6 Cracked Concrete Corrosion Damage

The quality and condition of the concrete cover is directly related to the penetrability of water and air (Alexander, et al., 2004). Sound concrete is more resistant to the penetration of harmful corrosive elements. Concrete that has cracks acts as pathways to the embedded steel, accelerating the corrosion process. The damage would continue and spread to other areas, leading to a decreased capacity of rebar and impacting on the serviceability of the structure (Alexander , et al., 2009). Cracks influence the rate of penetration; this rate is dependent on the crack's width, crack density, frequency, and crack orientation in relation to the underlying steel, the degree of connectivity and whether these cracks are active or dormant.

Concrete damage resulting from corrosion causes cracking, spalling and delamination of the concrete cover. Concrete deterioration of rebar positioned widely apart and closer to the surface, generally results in cracks. Concrete cracking is most likely to occur when the rebar are positioned closer to the surface and widely spaced. Rebar that is deeper and closely spaced will experience delamination of the concrete cover. This delamination occurs once a number of cracks combine before the cracks reach the surface (The Concrete Society , 2000). A crack can be defined as a relationship between the type of damage as a function of the bar size, the bar spacing and the structure's concrete cover.

Figure 9 below illustrates the lifecycle of a reinforced concrete structure and its relationship with concrete deterioration caused by the formation of cracks, delamination and spalling; eventually the structure will experience extensive corrosion, which would require repair or replacement. Damage due to corrosion is firstly seen by rust staining on the concrete's surface, then by the appearance of cracks, followed by delamination and spalling of the concrete cover. If left unrepaired the corrosion would continue causing spalling to be extensive and too costly to repair. The graph below shows the relationship between the degree of damage and the corrosion phases, comparing those factors to the overall service life of a structure. The graph also illustrates the best time for effective proactive maintenance and reactive maintenance.

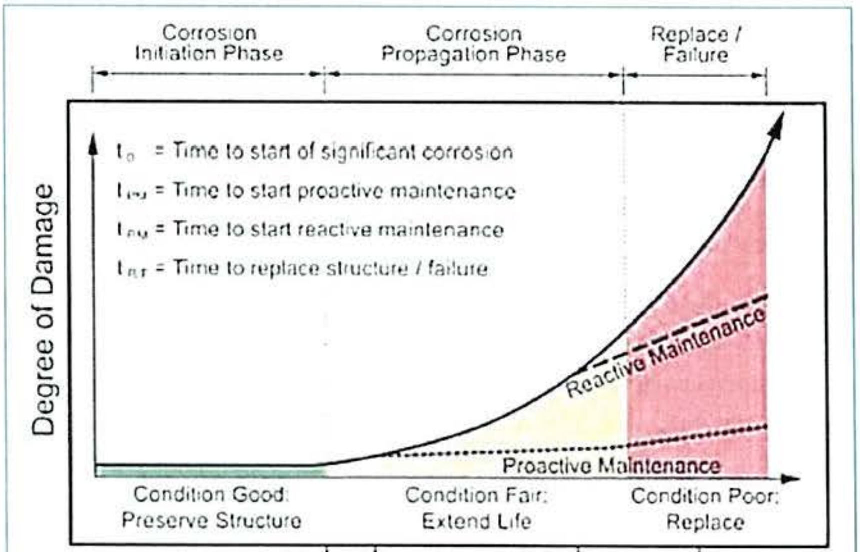


Figure 9: The degree of damage as a function of time (The Concrete Society , 2000).

The graph (figure 10) illustrates the relationship between the cost of maintenance to the condition of the structure, and its relation through different corrosion "zones", estimating when concrete deterioration would be visible. These two graphs are closely related and indicate the required management and maintenance of a reinforced concrete structures over its service life.

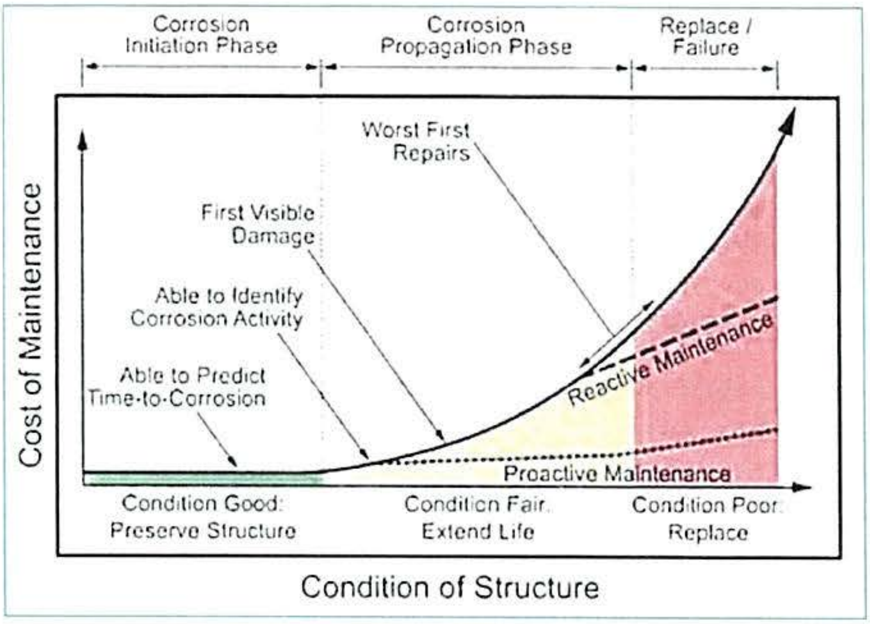


Figure 10: Bridge condition of time and the associated maintenance costs (The Concrete Society , 2000).

2.9 Reinforcement Corrosion Protection methods

2.9.1 Background

Engineers have developed methods to reduce and mitigate rebar corrosion. These corrosion mitigation measures cover a wide range of proactive and reactive protection systems to improve durability of reinforced concrete structures.

The civil engineering industry continues to improve corrosion prevention and mitigation technologies, in order to extend the service life of reinforced concrete structures. The use of alternative reinforcement material and protective coatings are becoming increasingly important in the construction industry. Engineers have started to re-evaluate current construction methods to improve the concrete quality and include the use of surface coatings to increase its resistance to harsh environmental conditions.

The current protection methods mainly address the mitigation of rebar corrosion. Other methods focus on the concrete surface to reduce the time air, water and salt ions are transported through the concrete substrate. Some solutions require reinforcing steel to be coated with metallic coatings, providing cathodic protection to the underlying steel.

Engineers are tasked with the selection of an appropriate corrosion protection method and the selection of alternative materials. Engineers must identify the type of environmental exposure and select the appropriate design factors namely, cover depth, rebar arrangement and the inclusion of anodes or electrically conductive connected to the reinforcement. The use of proprietary products provides a wide range of concrete coatings and hydrophobic solutions that can be used during construction and repair solutions. The corrosion protection of reinforcement is used in conjunction with other forms of corrosion mitigation methods applied to the concrete surface, increasing the protection of the underlying rebar. This is a multi-layered protection approach and would be more effective.

2.9.2 Corrosion Resistant Rebar

Reinforced concrete structures are used extensively throughout the civil engineering industry, along with proper concrete design to reduce any long-term costs and durability issues. Engineers must select appropriate materials and guarantee quality of construction and workmanship. The steel reinforcement consists of bars, rods, strands and welded mesh, and are placed and positioned in concrete to serve a variety of design functions (Theodosiou, 2009). Thus the reinforcement should be properly installed to the engineer's specifications.

The types of reinforcement used for construction in South Africa are in accordance to the South African National Standards, which includes plain, mild steel bars, deformed high yield steel bars and steel fabric. These are specified and standardised in SANS 920:2005, SANS 1024:2006 and SANS 920 respectively. Rebar is designated by R and Y, the R bars are smooth while the Y bars are ribbed or deformed. Mild steel bars have strength of 250 MPa and high-yield bars has a tensile strength of 450 MPa. In South Africa these high-yield bars are formed by hot rolling steel and results in ribs on the rebar's surface.

Engineers can use a number of corrosion protection systems to withstand the harsh environmental exposure conditions and inherent shortcomings of concrete, with specific repair solutions and methods. The correct use of these methods requires good knowledge of all available repair solutions, and the appropriate implementation of these solutions. The City of Cape Town makes extensive use of Hot Dipped Galvanised (HDG) rebar as a corrosion resistant solution and for its application is the best solution for the road authority. This is specified for all projects to compensate for perceived poor concrete quality and poor site construction supervision.

Reinforced concrete structures located on the coastline of Southern Africa has been shown to have its first maintenance intervention within approximately 10 years after construction (Roux, 2008), especially if the steel rebar is left uncoated. The time to its first maintenance could have been extended to over 30 years, if the rebar had been hot dip galvanised (HDGASA, 2005) and this is the strategy the City has employed as the best solution to combat rebar corrosion.

2.9.3 Hot Dip Galvanising (HDG)

The hot dip galvanising process was discovered in 1741 by French chemist Melouin, he proved that zinc was capable of protecting steel from corrosion. This method was used until another French man, Sorel, introduced pickling of the steel into sulphuric acid as a preparatory measure. The first patent was filed in 1837, and this main part of the procedure is still used in the modern process.

Galvanising refers to the galvanic cell that is created when the zinc coat is damaged. The steel in the damaged area becomes a cathode, and the zinc becomes the anode, in this galvanic cell, thus protecting the steel from corrosion. In South Africa, the corrosion protection industry's standard for steel products is to be hot dip galvanised with zinc process.

HDG is the most common reinforcement corrosion protection methods used in South Africa and is relatively simple to apply to most types of steels. In South Africa the production of HDG steel is in accordance with SANS 121:2000 and ISO 1461:2002. The HDG plants specify these standard specifications in order for the final HDG product to be cleaned and inspected for defects insuring a quality, defect-free product (Roux, 2016).

In most environments, the rate of zinc corrosion is very low making zinc an ideal coating of steel rebar. The HDG coating used for steel is zinc and it immediately reacts with oxygen and moisture to form a combination of zinc oxide and zinc hydroxide. These products react with carbon dioxide and convert them into a stable, tightly adhering zinc carbonate film with low solubility (HDGASA 2005).

The corrosion resistive nature of HDG should not replace the need for good quality concrete. HDG should rather extend the ultimate service life of the structure, especially once the corrosive elements have already penetrated the concrete and reached the surface of the rebar (HDGASA, 2005). For this reason, HDG reinforcement has been extensively used in South Africa, especially along the coastline. HDG improves the structural integrity and service life of bridges, coastal structures and industrial infrastructure which benefits the City, reducing maintenance cost for the City of Cape Town.

a. Advantages of HDG Steel

Specification of HDG reinforcement should not be considered as an alternative to good concrete practices (Theodosiou, 2009). The advantages of HDG rebar should help engineers specify design solutions for any environmental exposure condition.

HDG rebar can be used on a structure; especially where the corrosion potential is high and on concrete elements with relatively low concrete cover, such as slender columns and beams. Sometimes the structure's design could have architectural elements with partially exposed steel or where rust staining would not be acceptable. In general HDG rebar delays the initiation of corrosion and reduces concrete cracking deterioration. Hence, HDG rebar is favoured in high chloride environmental exposure to reduce the risk of corrosion.

b. Misconceptions of HDG

The most common misconception is that rebar cannot be bent after galvanising, which is believed to compromise the zinc coating. In addition, the false claim that the HDG coating fails when the surface is scratched or damaged (Theodosiou, 2009).

Some older HDG rebar misconceptions include the reduction of strength caused by the heat of the molten zinc. It is also believed, that HDG rebar could not withstand the inherent pH levels in concrete. The performance of HDG rebar in marine environments was thought to be influenced by the salt content of the air. However, to HDG advantage the magnesium salts in the air helps to assist in passivation. When the zinc starts to corrode it would display white corrosion products compared to darker corrosion products of HDG rebar in rural or urban area, which are darker.

2.9.3.1 Hot Dip Galvanising (HDG) Process

Hot Dip Galvanising is a process of applying a protective zinc coating to steel or iron to prevent the oxidisation and corrosion of the underlying steel. The HDG coatings on steel provide corrosion protection, acting as a non-porous film which isolates the steel from the surrounding environment. Furthermore, the zinc coating is a cathodic or sacrificial protection layer in the galvanic cell. This galvanic corrosion occurs once a galvanic cell is formed by two different metals surrounded by an electrolyte, which connects the galvanic system. The metals become either an anode or cathode depending on their electrode potential in the electrolyte solution.

Metals are characterised by their positions on the electrochemical scale, and a metal's relation to the connection between other metals. For example, if brass is connected to steel, the steel becomes the anode in the cell and corrodes. Using this same principle, steel can be connected to zinc. The zinc becomes the cathode and therefore the steel is protected against corrosion and the zinc becomes an anode (HDGASA).

The process of HDG is a metallurgical reaction between steel and molten zinc. This process forms a series of zinc/iron alloys together with the pure zinc layer. The steel has to be free from all contaminates to ensure a proper chemical bond between the carbon steel and the molten zinc. The contaminates such as paint, weld snags, rust and other substances are removed by abrasive shot blasting, sand blasting or grinding. The shot or sand blasting process removes 100 µm of the steel layer which is ideal for the HDG process.

The steel is cleaned and attached to a crane hangar system to dip the steel into a number of "baths" as part of the process. The steel is cleaned from grease, oils with degreasing chemicals and dipped in a dilute hydrochloric or sulphuric acid bath to pickle the steel removing rust and mill scale. The steel is then rinsed in a water bath. The water cleans and removes the acid from the steel surface as HDG does not work in an acidic environment.

The product is then dipped in a flux tank containing zinc ammonium chloride solution. This solution creates a barrier on the surface of the steel that prevents oxidation for about 40 mins. The solution leaves a salt layer on the steel that turns into chloride gas when dipped in the hot zinc; in addition this removes a thin layer of steel during vaporisation. A fluxing agent is then applied; the fluxing agent is used to dissolve surface oxides on the steel and on the molten zinc, ensuring metallic contact. There are two different ways of applying flux that have proven to give good coating quality (HDGASA).

Wet galvanizing is mainly used for small steel components and tubing. In wet galvanizing, the bath is divided into two parts. One section of the bath contains the fluxing agent (ammonium chloride) which is deposited on the zinc surface. Then steel items are dipped through the molten flux into the zinc. The steel is moved into the flux free section of the zinc bath. The zinc bath contains molten zinc which is at a

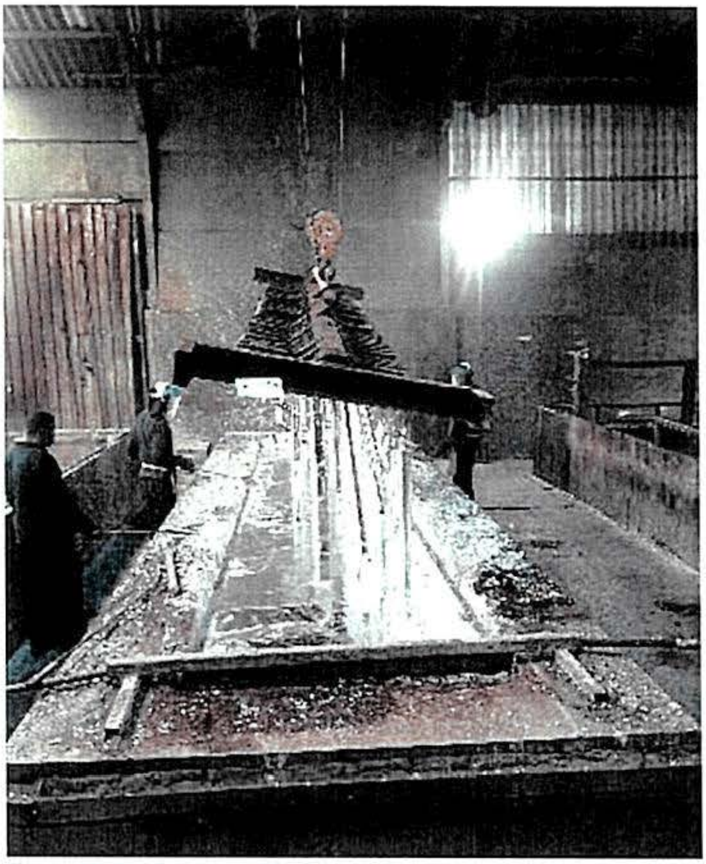


Figure 12: Steel sections lifted from the zinc bath at Galvatech (HDG plant tour 10/08/2016)

temperature of 450°C. This is a particularly dangerous process as all the free water rapidly expands and vaporizes causing explosive reactions, jettisoning molten zinc from the bath. The longer the steel is left in the zinc bath, the greater the thickness of zinc galvanizing surface (Roux, 2016). For example, steel which is thicker than 4mm would be dipped for four - five minutes to achieve adequate thickness of the zinc layer. The longer the steel is dipped, the thicker the zinc coating. The zinc or iron alloy provides better corrosion protection the thicker the alloy. However, the coating thickness should be less than 200 μm, if any thicker, the zinc coating tends to be more brittle and the coating tends to flake (Roux, 2016).

The flux residue and oxides are skimmed from the surface of the zinc bath; finally the product is lifted through a pure, smooth zinc surface. The HDG steel products are then exposed to the atmosphere. The zinc layer reacts with the oxygen to form zinc oxide, which then reacts with carbon dioxide to form zinc carbonate. This is usually a dull grey by-product that is harder than the underlying zinc and steel and assists with the corrosion resistance.

Finally, the steel is quenched in a sodium dichromate (0.5-1%) solution (at 100°C) that passivates the outer layer and retards the formation of zinc oxides, hydroxide or white rust formation during storage before concreting. This process changes the appearance of the fresh shiny HDG steel to a dull grey sheen. If the steel is going to be coated in a paint product, the HDG steel may not be passivated or dipped in plain water and alternatively just air-cooled (Roux, 2016).

The HDG products are then inspected to conform to SANS 121:2011 specifications. At this stage, the steel may get wet and the formation of white rust may occur. The white rust may be removed and wiped away once the surface has no more moisture. The white rust is not a major deficiency. It would be a defect once the rust is thicker than the layer of zinc which is visible with the formation of black rust spots. (Roux, 2016). Before the product is dispatched to the construction site, all sharp edges and attached steel lumps are removed with hand files to insure the thickness of the zinc coating is not compromised.

2.9.3.2 Performance of HDG

Hot Dip Galvanising essentially provides a twofold corrosion protection system. One layer of defence acts as a physical protection barrier. The other layer acts as a cathodic protection system, protecting the underlying steel from minor uncoated areas, normally caused by chips and micro cracks on the surface (HDGASA, 2005). This is useful when handling, fixing and bending damages occur after zinc coating has bonded to the parent steel.

The zinc will essentially will be sacrificed to protect the underlying steel. As the zinc is electro-negative compared to the parent steel, the zinc is the anode and the steel is the cathode, as per the galvanic series. Thus small uncoated steel areas will be cathodic protected by the zinc layer. The thickness of the zinc or iron alloy needs to be controlled as this coating tends to be brittle and may flake the thicker it becomes. However, coatings that are 200µm are estimated to be 30% better than pure zinc (Roux, 2016).

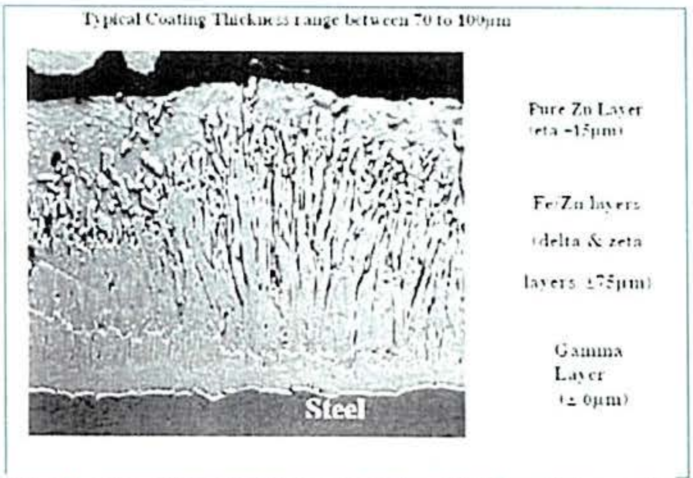


Figure 11 : zinc coating on the parent steel and the bond between the concrete (HDGASA, 2015).

The galvanized steel is placed in fresh concrete; creating a highly alkaline environment with a pH greater than 12.5. The corrosion of the zinc which occurs at pH greater than 12.5 is a misconception (Grantham, 2003). During the first few hours, the concrete is at a pH below 13.3, due to the presence of sulphate ions in Portland cement. Once the concrete hardens and cures stopping the reaction and is inhibited when the galvanised rebar is chromate passivated (Houston, et al., 1972).

During the initial contact between the fresh concrete and the HDG rebar, the formation of zincates grows on the outer region see figure 13, this is the interface between the zinc layer and the concrete. The zincates consume 5 to 10 um of the outer zinc layer, this is the passivated layer. The hardening of the concrete over 7 to 10 days leaves a layer of zincates on the original zinc/iron alloys intact and is able to provide corrosion protection (Roux, 2016). Concrete curing reduces the pH levels of the concrete to a range between 8 and 12. In this environment the zinc performs better compared to unprotected steel when CO_2 starts to ingress the hardened concrete.

Hot dipped galvanised reinforcement offers advantages over normal steel exposed in the same environmental conditions. These advantages are evident due to the HDG rebar effectiveness to decrease the risk of corrosion, its resistance for low cover members and the corrosion protection of steel before it is fixed on site (HDGASA, 2005). The problem of pitting corrosion that may occur on HDG steel tends to be lower than that of normal black steel. The zinc coating around the pits is a poor cathode, reducing the effectiveness of the cathodic mechanisms. It also reduces or completely removes the corrosion process.

Advantages

HDG rebar offers additional protection to reinforced concrete structures with low concrete cover, or structures with a higher design life. In practice, HDG rebar can be used in conjunction with normal carbon steel rebar that can reduce overall cost of construction. The HDG improves the aesthetics of a structure, as it will not exhibit steel corrosion, or rust staining, when steel is partially exposed (Alexander , et al., 2009).

An old pedestrian bridge located in the Western Cape was scheduled to be demolished after 40 years of service, a HDG case study was conducted by the Galvanised Association of South Africa (HDGASA, 2015). The bridge is located 50 meters from the sea and was constructed with HDG reinforcement. The concrete bridge elements were examined, as well as the condition of the steel/concrete interface. The findings showed that the hot dip galvanized reinforcement was at a cover depth in a range of 45 to 50 mm and the zinc coating thickness of 70 to 90 μm (see figure 16). The outer layer of the steel was dull grey with no significant zinc layer degradation. Generally, the galvanized coating thickness was found to

Figure 15 Isolated rust in regions of minimal concrete cover. (HDGASA, 2015)

be between 240 and 260 µm. The study concluded that the 40-year-old pedestrian bridge performed significantly well, considering its location to the sea and the surrounding harsh marine exposure conditions. The HDG reinforcement allowed for the pedestrian bridge to have slender concrete members with minimal concrete cover, and has proven to be durable, extending the service life of structures. The use of HDG rebar and proper concrete mix design, good concrete cover and quality site practice can improve the durability of any structure.

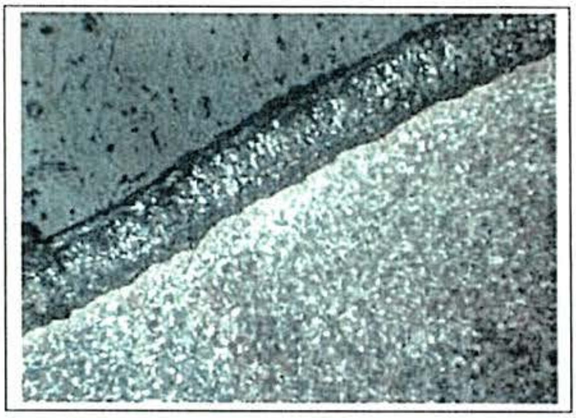


Figure 16: Micro analysis of the interaction between the parent steel and the galvanized region, also indicated is the bond with the bond. (Rendell , et al., 2002)

Disadvantages

Galvanic corrosion occurs in stainless steel rebar are used or the use of stainless steel bolts in conjunction with HDG rebar occurs; this is not a recommended practice due to the occurrence of localized oxidation. Any cutting or welding of the HDG rebar reduces its resistance to corrosion. A major disadvantage of HDG rebar is hydrogen evolution that occurs once zinc comes into contact with fresh concrete. Many researchers consider the formation of this gas to compromise the bonding between the concrete and the rebar (Theodosiou, 2009). The hydrogen evolution coupled with the formation of calcium hydroxyzincates was believed to reduce the bond strength between the reinforcement and the concrete. Moreover, it should be noted, this type of zinc corrosion reaction is only active during the initial concrete curing period of between six to 10 days, but cease as hydration reaction slows down. The cement hydration reaction and zinc corrosion rate can be controlled with the presence of chromates. Chromates are introduced in the final stages of the HDG process, or mixed into the concrete as an additive. This will mitigate the effects that cause the reduction of bond strength (Roux, 2016). However there is lack of proper investigation in galvanising bar bond strength and must be further studied.

Most of the relevant literature has conflicting findings that either support or deny the effectiveness of bond strength of HDG steel when compared with uncoated black steel in concrete. Studies have been conducted to evaluate the differences in bond strength between zinc coated steel and normal carbon steel.

The addition of chromate is a common solution to increase the bond strength between the HDG rebar and the concrete. The chromates are added to the concrete mix or by giving the bars a chromate passivation treatment during the HDG process. However, the addition of chromates to the concrete mix is not recommended, as soluble chromates are carcinogenic and exposure should be minimised (Theodosiou, 2009). The best recommendation is to passivate the galvanised elements in chromate solution at the HDG plant, as this is more effective than mixing it in the concrete before casting. Evolution of hydrogen will then be very short-lived and may well cease within approximately one hour (HDGASA, 2005).

Kayyali and Yeomans (1995) tested the bond strength and resistance of coated reinforcement in concrete. Their scope only investigated ribbed bars in beams finding that the ultimate capacity in flexure had no significant difference between the black steel and the HDG steel. The use of pull-out tests concluded there is no difference or reduction of the bond strength between HDG steel or uncoated steel. In fact some researchers have mentioned that HDG rebar might marginally improve the bond strength. Clearly more research is needed to draw definitive conclusion of HDG rebar bond strength.

2.9.4 Cold Galvanising

Cold Galvanising is another method that provides steel rebar with corrosion protection and is normally applied onto the black steel in paint form. The Cold Galvanizing process refers to the zinc rich epoxy used to touch up steel components. Good quality cold galvanising products may contain up to 90% zinc (Roux, 2016). Another form of Cold Galvanising is electro-galvanising; however this has a zinc quantity as little as only 10-50 g/m^2. Electro-galvanising method uses electrolysis to make metals adhere on the surface of parts of steel.

The Hot Dip Galvanizing produces a much thicker layer than Cold Galvanizing. Cold Galvanizing lacks a smooth good-looking final appearance which HDG provides. The adhesion of Cold Galvanizing to the underlying metal is inferior. The cost of cold galvanising is much cheaper than Hot Dip Galvanising. This form of galvanising is not recommended in the Cape Peninsula and should only be used to touch out on certain HDG steel items (Roux, 2016).

2.9.5 Stainless Steel (SS)

Stainless Steel rebar is more commonly used in structures located in high chloride environments, such as coastal areas or corrosive industrial environments. Stainless Steel bars can be used in concrete roads, bridges and sea walls. Stainless Steel rebar offers a corrosion protection solution which is maintenance-free and provides a high corrosion resistance (SASSDA, 2014). Many engineers are reluctant to specify Stainless Steel due to the initial high cost. However, a project may only be increased by a small percentage more than the normal steel cost and proves to be more cost effective in the long term.

Stainless Steel is protected by a chromium oxide layer on the surface which is highly resistant to levels of chloride ion (Theodosiou, 2009). Corrosion can only occur when there is enough oxygen to oxidise the highly reactive chrome that forms an impermeable protective barrier against corrosion. Stainless Steel reinforcement can tolerate chloride levels of up to 7%, whereas carbon steel can only tolerate chloride levels of 0-0,4%. Hence the concrete cover with underlying Stainless Steel would not crack, spall and deteriorate like normally reinforced concrete. This drastically reduces the structure's maintenance and repair costs.

The initial cost of Stainless Steel rebar would reduce the actual cost of repair throughout a structure's life. It is important that the outer most layer of reinforcement be more resistant to corrosion. Therefore, the design of a structure may allow for the first layer of normal steel can be replaced with Stainless Steel rebar (Theodosiou, 2009). The use of Stainless Steel rebar is not common in South Africa, because of the initial high costs. The production of SS rebar has ceased in South Africa and if required, it has to be imported, further increasing the construction costs.

There are two types of Stainless Steel rebar products available, namely solid stainless and stainless clad steel rebar. The solid stainless steel rebar can be bent, cut and weld in the field, and is also very resistant to chips and scratches eliminating the need for any coating. Stainless clad steel rebar is however more vulnerable to surface defects (BS6744:2001 & ASTM A955) and must be handled with more care.

With the use of corrosion prediction models, engineers can predict and design appropriate replacement of carbon steel rebar with stainless steel rebar. A combination of carbon steel and stainless steel can be used in the same structure and would not require a separate barrier between the two zones. This will have a stainless steel rebar designation required, especially in areas where it is difficult or expensive to repair.

The design standards of SSR concrete structures eliminates the need for concrete coatings, allowing an increase in crack width of 3mm and a reduction of concrete cover of 30mm (BS & ASTM). In addition, Stainless Steel has very few disadvantages; one disadvantage is the high strength of stainless steel attributing to be more brittle and may cause brittle or sudden failure of RC elements (Theodosiou, 2009). The application of stainless steel rebar is not acceptable, where no oxygen is present and high fatigue is likely to occur. South Africa has successfully used 3CR12 type Stainless Steel bars in the construction of four pedestrian bridges at Warner Beach on the SA South coast and the ore Jetty in Saldanha bay (Theodosiou, 2009) and has performed well.

The 3CR12 bars was developed in the 1970's by Columbus Stainless. It is now recognised as the world's most specified SS rebar with 12% chromium. Its price is competitive; it is highly corrosion resistant, and the chromium is particularly advantageous in wet abrasive applications. Unlike other ferrites, stainless steel can be welded in thicknesses of up to 30 mm. 3CR12 meets the European standard 1.4003, included in specifications of Eurocode EN 10088 and it also conforms to ASTM A240-UNS-S41003.

The 3CR12 bars provides a superior alternative to coated carbon steels which have poor corrosive resistance. It reduces costs through eliminating the need of protective treatments and allowances. It has proven to performance better in the mining industry, in the materials handling sector and sugar industries due to its resistance to atmospheric and wet abrasive corrosion conditions (SASSDA, 2014).

2.9.6 Fibre Reinforced Polymers Rebar

The use of non-metallic rebar such as fibre reinforced polymers (FRP), glass reinforcement polymer (GRP) or carbon fibre reinforcement polymers (CFRP) rebar in RC elements has increasingly been used to prevent the effects of corrosion in harsh environments.

The FRP bars are manufactured using a variety of fibres and epoxy. The most common one is glass fibre but carbon, basalt and other fibres are slowly being introduced. The polymer matrix consists of the fibres and an epoxy or vinyl polymer. The material properties that govern the bond interaction between the concrete and the FRP rebar may be an issue. The concrete cannot bond physically or chemically to the surface of a FRP bar. To solve this weak bond issue, the surface has an addition of grit which is being tested to increase bond strength and even the incorporation rebar with ribs.

The FRP bars have unique physical and strength characteristics. The FRP bars are composed of fibres that provide its strength and stiffness, and also contain an epoxy matrix that protects and transfers the loads between the fibres. This creates a material that has attributes stronger than the components on its own.

Advantages

The use of FRP bars in RC structures eliminates the risk of corrosion when compared to normally reinforced concrete members. This vastly increases the service life of a concrete structure. There are hundreds of successful engineering applications of FRP bars in America and Europe. FRP rebar has favourable material properties such as good stiffness ratios and non-corrosive properties making these structures impervious to chloride ions and chemical attack. FRP bars have tensile strength greater than that of steel (Theodosiou, 2009).

FRP bars can be used on any concrete structure or elements susceptible to chloride corrosion. They can be used as an alternative to epoxy coated rebar, Hot Dip Galvanized steel, solid stainless steel rebar or stainless steel clad rebar. FRP bars can also be used in conjunction with normal steel on concrete members with low concrete cover exposed to harsh environments. It is transparent to magnetic fields and radar frequencies; furthermore, FRP are electrically and thermally non-conductive. Therefore, FRP rebar ideal for use in hospitals, around their magnetic scanners, or on structures with maglev railway infrastructure. They are also being used on large concrete bases and mining applications to reinforce ground areas, where normal rebar can damage the mining equipment. Locally, the Sea Point seawall rehabilitation project has adopted fibre reinforcement to be used in the concrete bollards, to mitigate the corrosion experienced by the existing steel reinforced bollards historically used on the Atlantic Sea Board (Vink, 2017).

Disadvantages

The initial higher cost of FRP is considered as a disadvantage. The cost of glass RFP reinforcing bars is approximately three times that of black steel, whereas carbon fibre for pre-stressing tendons is 10 times more expensive than black steel (burgoyne, 2009). In South Africa, FRP bars are only available in straight bar types and special orders are required to make other common shapes, this may further increase costs.

The cost of FRP bars would decrease if the demand for FRP were to increase as manufacturing would become easier. The use of FRP bars for repair and retrofitting would become an attractive solution and more cost effective on structures with an extended service life. The maintenance costs of repairing a deteriorated structure would be lower than repairing a traditional steel reinforced structure. The cost savings benefits would outweigh that of the maintenance costs for steel bar replacement or extra corrosion prevention systems specified during rehabilitation.

FRP bars have little flexibility and are very brittle; this would be problematic in certain applications where steel rebar is to be replaced. Carbon fibres are known to have higher strengths compared to steel but with the same stiffness properties. This means FRP elastic strain capacities are in the range of 1,5% -

2%, as opposed to steel rebar strain capacity of 0.2%, and concrete has cracking strains of 0.01% and compression strains of 0.1%.

FRP bars are brittle and results in sudden failures, however steel rebar is more elastic and allows for plastic deformation before critical failure. When the steel rebar fails, the ductility allows for the redistribution of the loads, therefore the structure would not suddenly fail. By contrast, FRP rebar would fail suddenly and the neighbouring rebar would be overstressed and then fail as well, drastically affecting the overall strength of the entire structure, causing critical failure.

Another issue of FRPs is the bond strength interaction between the concrete and the FRP bars. Normally in concrete, steel rebar de-bond from the concrete, reducing critical stress build up along the rebar, however, FRP bars are too brittle and would fail critically as they would not de-bonding (Theodosiou, 2009). To combat this probably, FRP bars may be coated with sand, or twisted to increase the concrete bond strength. Shear strength of FRP bars are lower than normal steel (Theodosiou, 2009).

2.9.7. Concrete Surface treatments

Hydrophobic coatings are products used to treat the concrete surface to produce a water-repellent surface. The pores and the capillary network are not filled with the hydrophobic coating but instead lined with them. This breaks up the water surface tension on the concrete's surface and prevents its passage through the pores and thus still allows for vapour to diffuse through the concrete structure.

Cementitious coatings are defined as a coating that improves the surface of the concrete structure by increasing its resistance against external influences like moisture and air. If the concrete surface has cracks of a width of up to 0,3mm, it can be successfully repaired by the ability of these concrete coatings "bridge" these cracks, waterproofing the surface (SIKA , 2005).

Corrosion inhibitors have the ability to increase the chloride concentration threshold, which is required for the corrosion process, delaying the time of corrosion initiation. This means that normal steel rebar would not corrode under the normal chloride threshold of 0,4%. The corrosion inhibitor impregnates into the pores of the concrete and improves corrosion protection of undamaged reinforced concrete, especially where the reinforcement is at risk of carbonation or chloride attack.

Penetrating corrosion inhibitors are good at dealing with carbonation induced corrosion and high levels of chlorides (Grantham, 2003). The corrosion inhibitors penetrate through concrete at a rate of a few mm per day and to a depth of approximately 25 to 40 mm in one month after application (SIKA sa, 2006). These products have been successfully used in the metal industry. Inhibitors have the ability to delay both the anodic and cathodic corrosion reactions, by penetrating into the concrete and surrounding the rebar and has be used extensively in the repair and maintenance of existing concrete structures.

Cement Coating systems has the advantage of not changing the appearance of the concrete structure and can be used on RC structures where corrosion has already occurred. The City of Cape Town has successfully used it on certain rehabilitation projects, it is an attractive solution as it is a very simple, and does not require a lot of skill to apply; it has proven to be an effective corrosion mitigation solution.

2.10 Cost of Corrosion

The estimated cost of concrete repair of existing bridge infrastructure in the USA is over $50 billion, and $1 to $3 trillion for all concrete structures. In Europe, steel corrosion has been estimated to cost about $3 billion per year. In South Africa, the concrete repairs due to corrosion of structures are very common, especially along the coastline (Zhou, et al., 2001). The city of Cape Town hasn't tracked the true financial costs of corrosion repair and concrete rehabilitation, as it hasn't been treated as a high priority for City projects. However it is important to address the corrosion problems with the use of better durability design and with the use of alternative rebar materials or corrosion resistant methods.

The use of black steel instead of HDG steel is a better option when the concrete quality is high which ensures concrete durability. Concrete of good quality provides protection of the embedded steel, thus eliminating the need for additional corrosion protection.

Alternatively, the use of sacrificial zinc anodes placed at regular intervals may help to prevent steel corrosion. However, this solution may prove to be too costly on larger construction projects. The better solution used by the city of Cape Town for reinforced concrete structures located near the coast or in areas with high possibility of corrosion, is HDG steel as it is more economically viable when viewed in long term maintenance models.

The cost of Hot Dip Galvanizing steel reinforcement is insignificant compared to the cost of repairing deteriorated concrete due to rebar corrosion (HDGASA, 2005). Although the construction costs are subjected to factors, such as the price of concrete, price of steel, site location, and contractor's overheads. The use of HDG steel increases the overall cost of the reinforced concrete structure between 5% to 10%, this has been the case on all City concrete rehabilitation projects (Roux, 2008), (Vink, 2017). Engineers may re-evaluate the use of HDG, with the strategic use of HDG rebar in certain locations, especially in areas of high-risk corrosion. This clever use of HDG rebar may reduce the overall cost to as little as 0.5 to 3%. Whatever the final project costs may be, the specification hot dip galvanised steel would be more economical than many of the alternative corrosion protection methods and would provide long term savings as it would reduce the maintenance and repair costs.

While other methods of reinforcement protection are available, City engineers has specified Hot Dip Galvanized rebar as the best corrosion preventative method and has been specified for a large number of city projects. It is a philosophy of "Prevention is far better than cure" and is specified for all construction projects in the city of Cape Town in an effort to prolong RC structure's service life. The so-called De Sitter's "Law of Five" sums up the design philosophy that should be taken by engineers and is stated as follows:

One dollar (approx. R13.00) spent in getting the structure designed and built correctly is as effective as spending $5 (approx. R65.00) when the structure has been constructed but corrosion has yet to start, then spending $25 (approx. R32.-00) when corrosion has started at some point, to spending $125 (approx. R1 62.-00) when corrosion has become widespread".[Exchange rate October 2015 (HDGASA, 2015)]. Meaning that neglecting the future costs of maintenance and repairs may prove more costly over time than addressing the problem at the project's inception stage.

Many City engineers are convinced that HDG is the best corrosion prevention solution and is specified at the design stage to reduce costs in the future. In South Africa, Hot Dip Galvanizing reinforcement adds value and longevity to concrete structures. The HDG rebar has the added benefit of compensating for lack of concrete durability and quality, and may only increase the cost of construction by a small margin (0.5% - 3 %) of the total project costs (Wilmot, 2007).

Cost Comparison of Different Types of Rebar

The tables below illustrate the comparison of steel rebar and the other common corrosion resistant rebar available in South Africa. It is up to the design engineer to select the most appropriate and cost effective solution.

Table 2: Comparison of the types of corrosion resistance materials

Type of Rebar	Corrosion resistant compared to black steel	Scratch, chip resistant	Bending and cutting	Chloride Threshold	Cost (R/Kg or R/m)
Black steel	n/a	n/a	Remains the same	0-0.4%	R 13/Kg
HDG steel - non chromium coated	Zinc corrosion occurs	Good resistance	Not recommended after HDG	0.5-3%	Same cost as below.
Hot Dipped Galvanised (Zinc Coated Steel)	Relatively good	Very resistant – cathodic bridge	Not recommended after HDG	0.4-5%	Base cost of Black steel - 8mm & under(R11.25) 10mm & over (R 10.55/Kg)
Solid Stainless Steel *	Highly resistant	yes	yes	>5%	R 36/Kg excluding importing
FRP bars (SIKACARBODUR BC12)#	No corrosion	No effect	Cannot be bent	No effect on Chloride	R 1035.80/m

*Solid stainless steel _ SA supplier but not the right grade (M. Basson from SASSDA (not available in SA, can import from Spain) & Eurosteel Thurston (thess@eurosteel.co.za) (SASSDA, 2014)
#FRP from SIKA available in South Africa

Table 3: Cost ratio of carbon rebar to corrosion resistant rebar

Rebar Type	Ratio
Carbon Steel (at R/kg)	1.0
Galvanized	1.5
Solid Stainless Steel	3.0
GRP	10.0

3 Rebar Corrosion of RC Structures in the Cape Peninsula

3.1 The Cape Peninsula

The Cape Peninsula is located on the south-western tip of South Africa, in the Western Cape Province. The Cape Peninsula is surrounded by the Atlantic Ocean and the Indian Ocean on the Eastern side. It has two large bays that are perfect locations for harbours, namely the Table Bay and the False Bay areas.

Cape Town is the central business district of the Western Cape and is located at Table Bay harbour situated on the coastline of the Atlantic Ocean. Cape Town is the first city of South Africa and was colonized in 1600s. It is well known for tourism and is one of the major economic hubs in South Africa, the city experiences constant development, construction and economic growth.

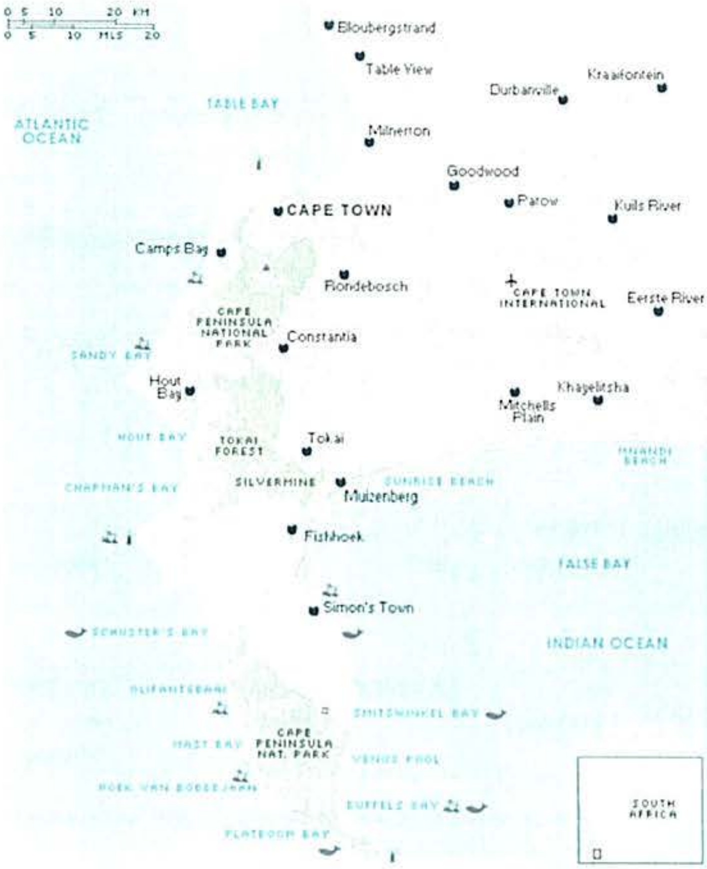


Figure 15: Map of the city of Cape Town and the greater Cape Peninsula (www.sa-venues.com)

3.2 Transport for Cape Town

The City of Cape Town is responsible for all road infrastructure and governmental structures. However, many of these RC structures are aging and exposed to extreme environmental conditions, and over time cause deterioration of the concrete, shortening the service life of these structures. Furthermore, the deterioration of the embedded steel in concrete is the most common form of failure in reinforced concrete (RC) structures. There has been lots of research in the concrete maintenance and repair industry. The South African construction industry conducts extensive research of concrete repair and rehabilitation.

Transport for Cape Town is responsible for the construction and the maintenance of concrete bridges, retaining walls, culverts and other road-supporting infrastructure. In total the municipality is responsible for 20,000 concrete structures; most had been built during the 1950s and 1980s. This means a large number of these reinforced concrete structures are older than 50 years. Many of these structures are currently showing signs of deterioration, mainly caused by rebar corrosion.

These deteriorating structures are located within 3km from the coast and shows visible signs of rust staining, cracks and concrete spalling. This type of deterioration is caused by chloride salts in the moist sea air and in splash zones along the seaboard.

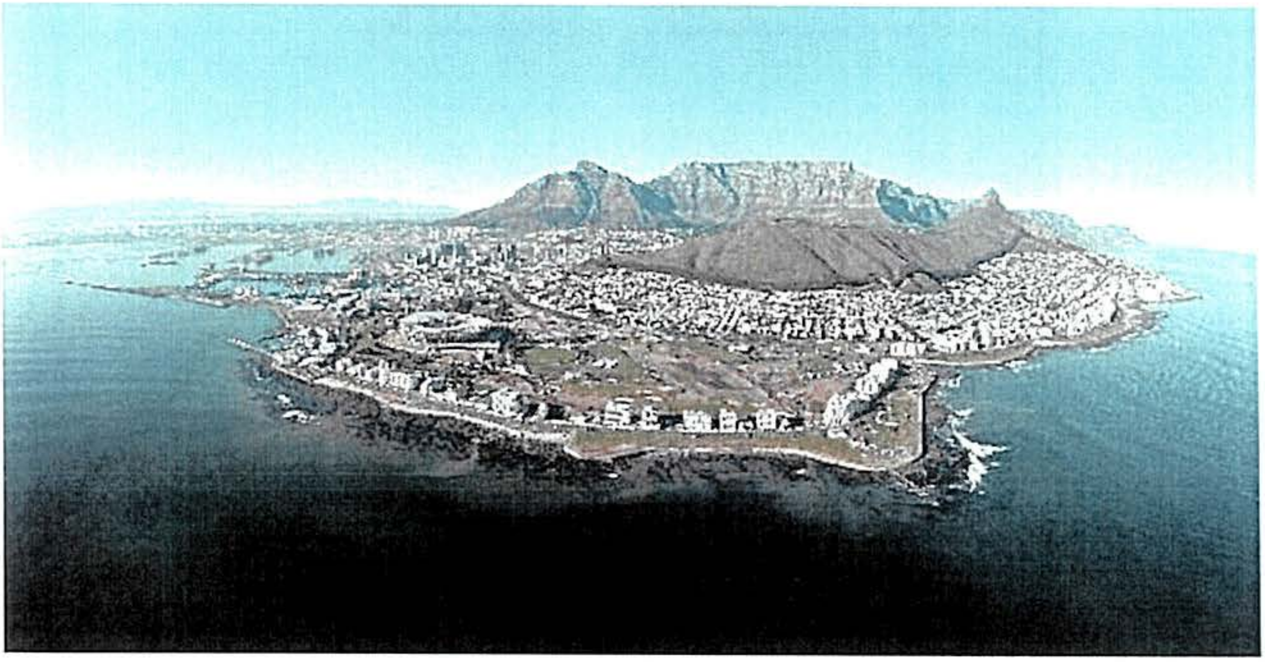


Figure 16: The City of Cape Town the Atlantic sea board in the foreground, harbour on the left and cape flats and the Muizenberg area behind Table Mountain in the background (Bruce Sutherland, 2009) Taken during the construction of Cape Town stadium for the 2010 Soccer World Cup.

The City of Cape Town has a large number of bridge structures located in the City; the known number of structures is 787, this includes footbridges, road and rail bridges, culverts, subways and pipe culverts with the majority being reinforced concrete (Roux, 2008). These structures are managed with the assistance of a Bridge Management System (BMS). The BMS prioritises the maintenance of reinforced concrete structures or concrete elements as they reach the end of their service life. It should be noted that the South African provincial road authorities such as Transnet, SANRAL and PRASA are responsible for their own structures, and forms part of a national transportation mandate, and includes major harbours, national roads and railways.

The City of Cape Town continue to develop and grow economically, with an increase in construction of new reinforced concrete structures, including the construction of new transport systems, the new dedicated concrete bus lanes for the MyCiti (Integrated Rapid Transit system). Furthermore, the City of Cape Town is responsible for coastal structures like sea walls, small harbours, spillways and concrete retaining structures. The structures that are included on the BMS are concrete light posts, medium highway/road barriers, concrete stairs and concrete buildings. The city has large amounts of reinforced concrete structures that require a lot of technically skilled manpower and resources to maintain. The area indicated by the red boundary in Figure 17 illustrates the City of Cape Town municipality region.

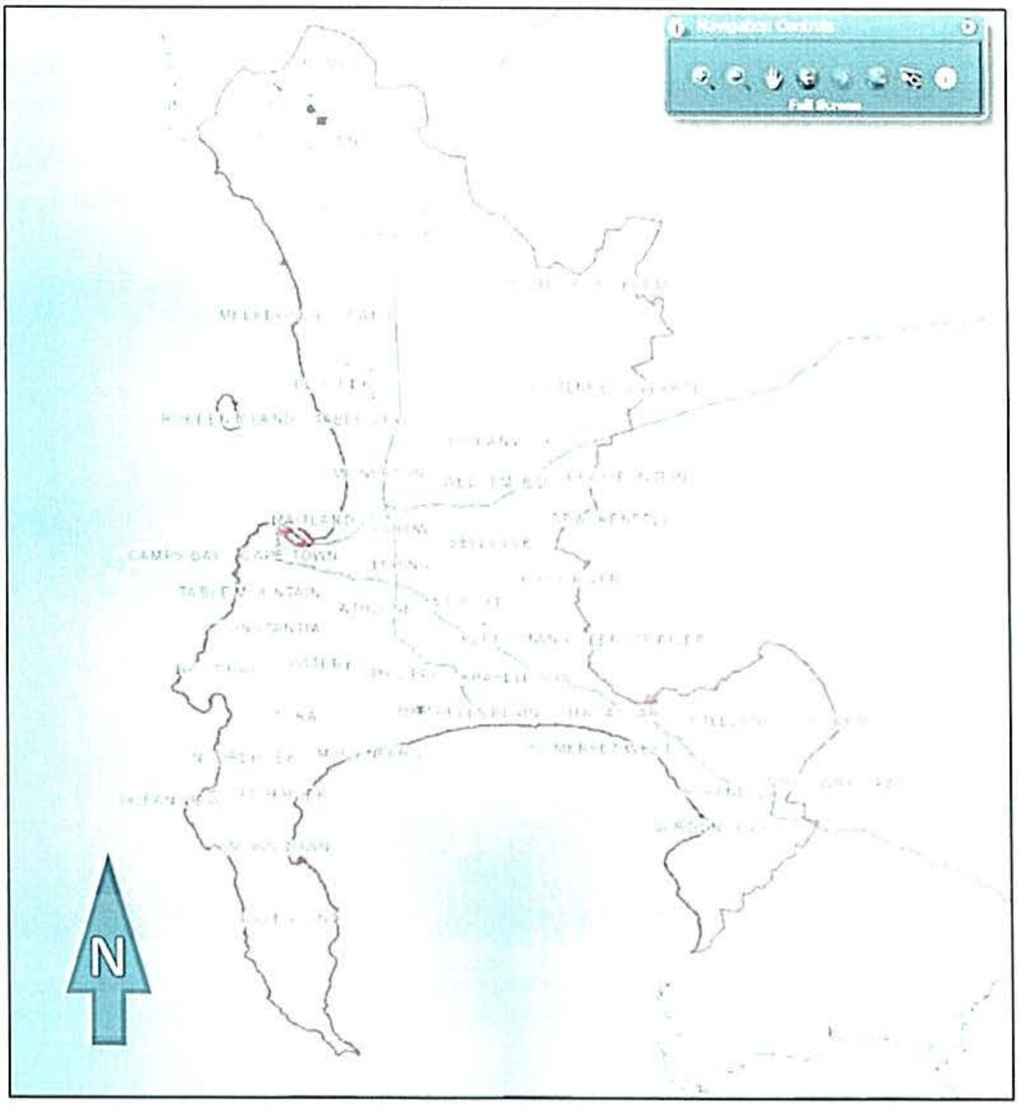


Figure 17: City of Cape Town shown on ArcGIS Map: The Cape Peninsula region and the area which falls under the jurisdiction of the City of Cape Town, with the coast line on the Atlantic Ocean. (ArcGIS 2016)

3.3 Existing Standards and City Practices

The concrete design and construction practices in South Africa follow prescriptive concepts and specified in the South African National Standards (10100-1:2000) Reinforced Concrete Design.

Transport for Cape Town has implemented the use of HDG steel for all repair and rehabilitation projects. This is done regardless of any use of any prescribed standards during the design and detailing of concrete elements. Usually the concrete cover, concrete mix design and curing methods are standardised to conform to existing structural elements or set to a particular standard, no matter the environmental exposure conditions or different site conditions. This approach is not cost-effective and results in overdesign and higher project costs.

3.4 Concrete Durability Standards

3.4.1 European Standards

EuroCode standards are closely related to SANS, it defines the exposure class and the type of deterioration which would be experienced in a particular exposure class. This is a general design standard but works well when concrete durability designs are considered.

Table 4 : Exposure Class: EN 206

Exposure Class	Relates to	No. of sub-classes
XO	no risk of corrosion attack	no sub-classes
XC	Carbonation induced corrosion	4
XD	Chlorides not from Sea Water	3
XS	Chlorides from seawater	3

**EN206 environmental conditions and general durability requirements*

3.4.2 South Africa

The concrete durability standards used in South Africa form part of the SANS design codes and focus on concrete durability specification targets based on Oxygen Penetration Index, Chloride conductivity and sorptivity tests. The common design methods are a combination of prescriptive and performance based specifications that are being employed for design and specification of civil engineering projects (Section 8.2 SANS 10100-2 and Section 10.8 SANS 10100-2 and Table 6000/1 Concrete Durability Specification targets). These tables has design specifications for both carbonation induced corrosion and chloride induced corrosion in different exposure condition classes, providing engineers a useful design tool to select cover depths, typical binder blends and recommended values of sorptivity tests ensuring the concrete's quality.

Exposure Classes

The exposure class classification for carbonation- and chloride-induced corrosion has been drafted in terms of the European standards EN 206. (Bickley, et al., 2006). The specifications for concrete cover are based on durability indices that measure the chloride conductivity, the transportation of moisture and air through the concrete. The South African Durability design utilizes the following tests: chloride conductivity index, oxygen permeability index and water sorptivity index. These tests measure the concrete's durability characteristics and provide input for life cycle models and bridge management systems. This assistance engineers to predict the performance of reinforced concrete structure over its service life. However, this type of specification and service life modelling is observed by some in the civil industry as being too sophisticated for most designers and engineers (Bickley, et al., 2006), and unfortunately the attitude in the City of Cape Town's Municipal Authority.

Another proposed specification approach would determine the exposure conditions that the RC structure would be located in. This specification approach would then require the concrete mix and other construction qualities to be known.

Table 5: Exposure Classes

Exposure class	Relate to	No. of Sub-classes
XS0	Exposed to airborne salt	2
XS1	Permanently Submerged	2
XS2	Permanently submerged on one side	2
XS3	Tidal splash and spray zone	2
XSC	Chloride induced corrosion	3

From Eurocode (Table 2.5.1.1 (b))

Most designers are not experts in concrete technology and concrete durability standards; therefore it would be necessary to divide typical exposure environments into classes to enable appropriate recommendations to stipulate adequate durability standards (Newman & Choo, 2003). This system categorizes the exposure environments into major designations by the degree of impact on the concrete and the reinforcement durability.

For the purpose of this dissertation, an extract of the following exposure designations (table 4, 5 and 6) to class the concrete deterioration from carbonation-induced corrosion and chloride-induced corrosion, including their sub-clauses divided into increasing levels of moisture and other environmental exposure factors. However, the freezing, thawing and chemical attack conditions are excluded as it is rarely a durability design consideration in the Cape Peninsula.

The use of this exposure classification system in EN206 has improved design solutions to combat concrete deterioration and has led to better selection of corrosion resistant methods and durability standards. Exposure conditions from European Standard for concrete are given in the table 6 below.

Table 6 Exposure Classification system from the European Standard for Concrete

Class Designation	Description	Informative Examples where classes may occur
1 No risk of corrosion or attack		
XO	For concrete without reinforcement or embedded metals, and concrete structures in dry regions	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation Where concrete containing reinforcement or other embedded metal is exposed to air and moisture, the exposure shall be classified as follows:		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet and rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure XC2
4 Corrosion induced by chlorides from sea water: When concrete containing reinforcement or other embedded metal is subject to chlorides from the sea water or air carrying salt originating from sea water, the exposure shall be classified as follows:		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to and on coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal and splash and spray zones	Parts of marine structures

**(EN 206-1 2000)*

This exposure class system is an improvement over the older British Standard Code of practice for structural concrete (BS 8110-1, 1997) as it classified the environment by its effect on concrete and reinforcement (Newman & Choo, 2003). The American Building Code (ACI 318-02) does not have an exposure classification system, but does identify certain conditions which require special attention.

South African Chloride Ingress Model

This is an empirical performance-based prediction model for concrete structures in marine environments, and was developed by Mackechnie (1996). The model uses the results measured from the different durability index tests and characterises early age concrete properties and relates them to the potential durability performance of concrete exposed to a particular range of marine environments. The classification of marine environments is based on EN206-1 (2000), and modified for South African conditions (MG Alexander, 2003). In short, this model is based on the relationship between early age properties validated with long-term chloride ingress data from marine concrete structures (MG Alexander, 2003). Table 7 categorizes the exposure class and the type of environmental exposure.

Table 7: South African environmental exposure classes (after BS-EN-206-1, 2000) for chloride-induced corrosion

Exposure Class	Description
XS1	Exposed to airborne salt but not in direct contact with seawater
XS2a+	Permanently submerged
XS2b+ XS2a +	Exposed to abrasion
XS3a+	Tidal, splash and spray zones
XS3b+ XS3a +	Exposed to abrasion

+ These sub-clauses have been added for South African coastal conditions

3.5 Cape Peninsula Climate and Environmental Exposures

A major consideration for engineers is the interaction of the concrete, material composition and the environment exposure. The climate and weather conditions have an impact on the concrete design and construction practices. It is important to note, the type of environmental and exposure conditions that exist in the Cape Peninsula. With this knowledge, designers are able to design durable concrete structures and select the appropriate rebar material or the addition of corrosion mitigating measures such as concrete coatings or corrosion inhibitors.

Engineers in Cape Town are responsible for structures mostly located in close proximity to the coast. The region is uniquely located in the Southern tip of Africa. The peninsula is surrounded by the Atlantic Ocean and the Indian Ocean, and experiences wind that constantly carries water from the surrounding oceans on to shore. It should be noted that the majority of Cape Town's infrastructure is located within 5 km from the coastline.

For the purposes of this dissertation, some background was given on the environmental condition experienced in the Cape Town Municipal area (Figure 17), and the collection of weather data of Cape Peninsula climate from the South African Weather Service (SAWS), World Weather Online, Weather2 and Windfinder.com. The City of Cape Town has weather stations at Molteno (City Centre) and Cape Town International Airport (Cape Flats). The Data was provided by Transport for Cape Town: Catchment Planning department, which has weather data collected and recorded over a period of 38 years. Rainfall data was collected from the Department of Catchment Planning from key locations by weather stations and analysed in conjunction with the online weather resources.

3.5.1 General Climate

The Cape Peninsula is located in a Mediterranean-type climate and experiences cold wet winters and dry hot summers (Cowling , et al., 1996). Winters have high rainfall accompanied by north-westerly winds. These winds are caused by a high-pressure system over the Atlantic Ocean. The wind blows onto the South Western coastline and across the Cape Peninsula (Cowling , et al., 1996).

The Cape Peninsula can also be described as a marine environment, as illustrated in Figure 18. However, due to variations of land geography, location of the coast and wind patterns, factors all contribute to microclimates and variations of exposure conditions in different regions. Identifying these microclimates is a good indicator of the expected concrete durability.

For example, a reinforced concrete structures in the Muizenberg area experiences more airborne salt blown onto the structure's surface, whereas structures located on the Atlantic Sea board would experience dissolved salt in the splash zones and less from the prevailing winds. In other cases, the morning fog may carry salts on to the surface of concrete structures. This means the seaward areas experience more severe exposure conditions.

The majority of the Cape Flats region is close to sea level with no natural mountains. This part of the City is generally flat and stretches from Muizenberg to Paarden Island towards the North. The City and Atlantic seaboard is flanked by the Table Mountain range located at the South. Table Mountain has a height of 1082 m and has a significant effect on wind and rain patterns on the areas around the mountain.

The Northern Suburbs are located 25km from the City Centre and is located at a higher elevation than the Cape Flats. It is generally hotter and drier than the rest of the city. These factors reduce the effects from the airborne salts blown from the sea and do not reach the Northern regions. The main type of deterioration experienced by reinforced concrete structures in the Northern part of the city is mainly carbonation and rarely experience chloride-induced corrosion.

3.5.2 Exposure zones

The location of exposure zones in the Cape Peninsula can be divided as shown in Figure 18 below (Ralfe, 2012). These exposure zones have been selected according to existing weather monitoring and air quality stations located in these zones. The research effectively divides these zones depending on the environmental factors like wind, temperature, relative humidity and air quality (which includes chloride concentrations, CO_2 and other pollutants) based on records from air quality and weather monitoring stations. Furthermore, this dissertation investigates historical weather and climate information that has shown the exposure zones and seasonal climate differences.

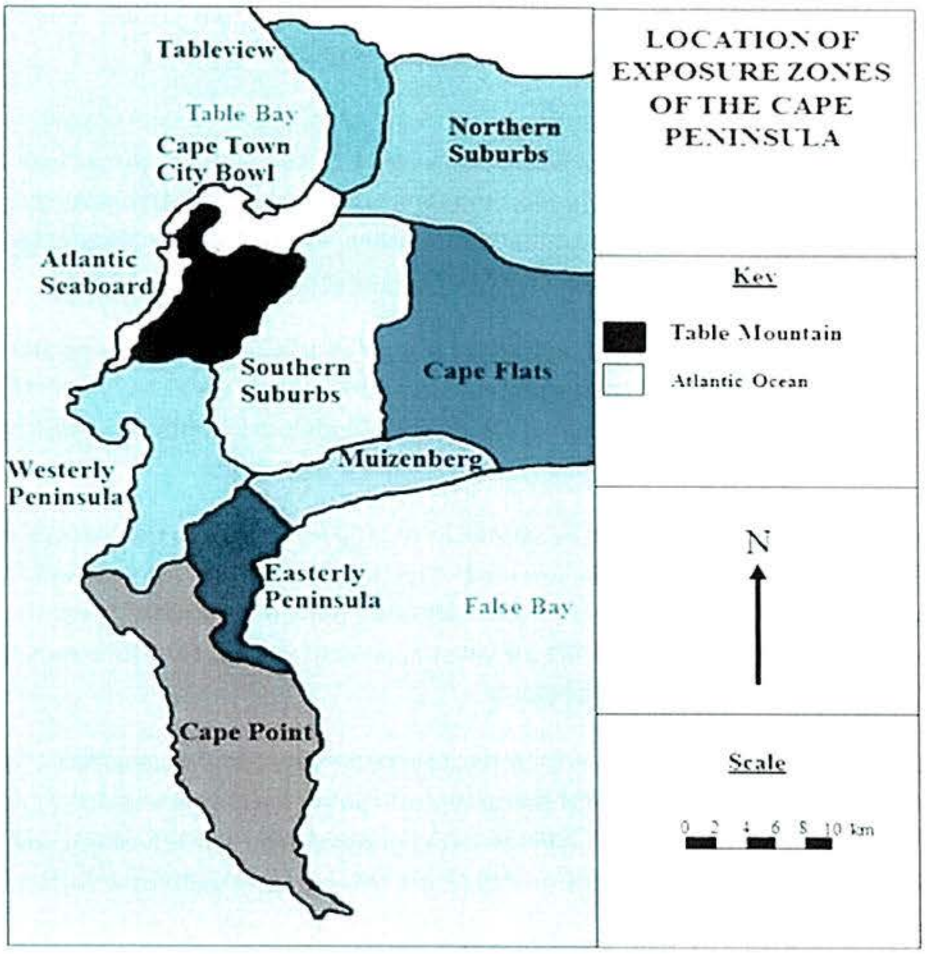


Figure 18: Cape Peninsula Divided into exposure zones (Ralfe, 2012)

Concrete structures in Cape Town are saturated for a longer period of time due to the higher number of rain days during the winter periods thus creating conditions ideal for increased chloride ingress during these months. Figure 19 below shows the exposure zones in the Cape Peninsula and the average seasonal precipitation.

The Cape Peninsula can be divided into exposure zones monitored by City authorities. These exposure zones classify exposure conditions in these zones and would be useful for future design considerations of reinforced concrete structures (Ralfe, 2012). Existing weather stations monitor climate and weather factors such as wind speed, wind direction, precipitation, rain intensity, temperature and relative humidity.

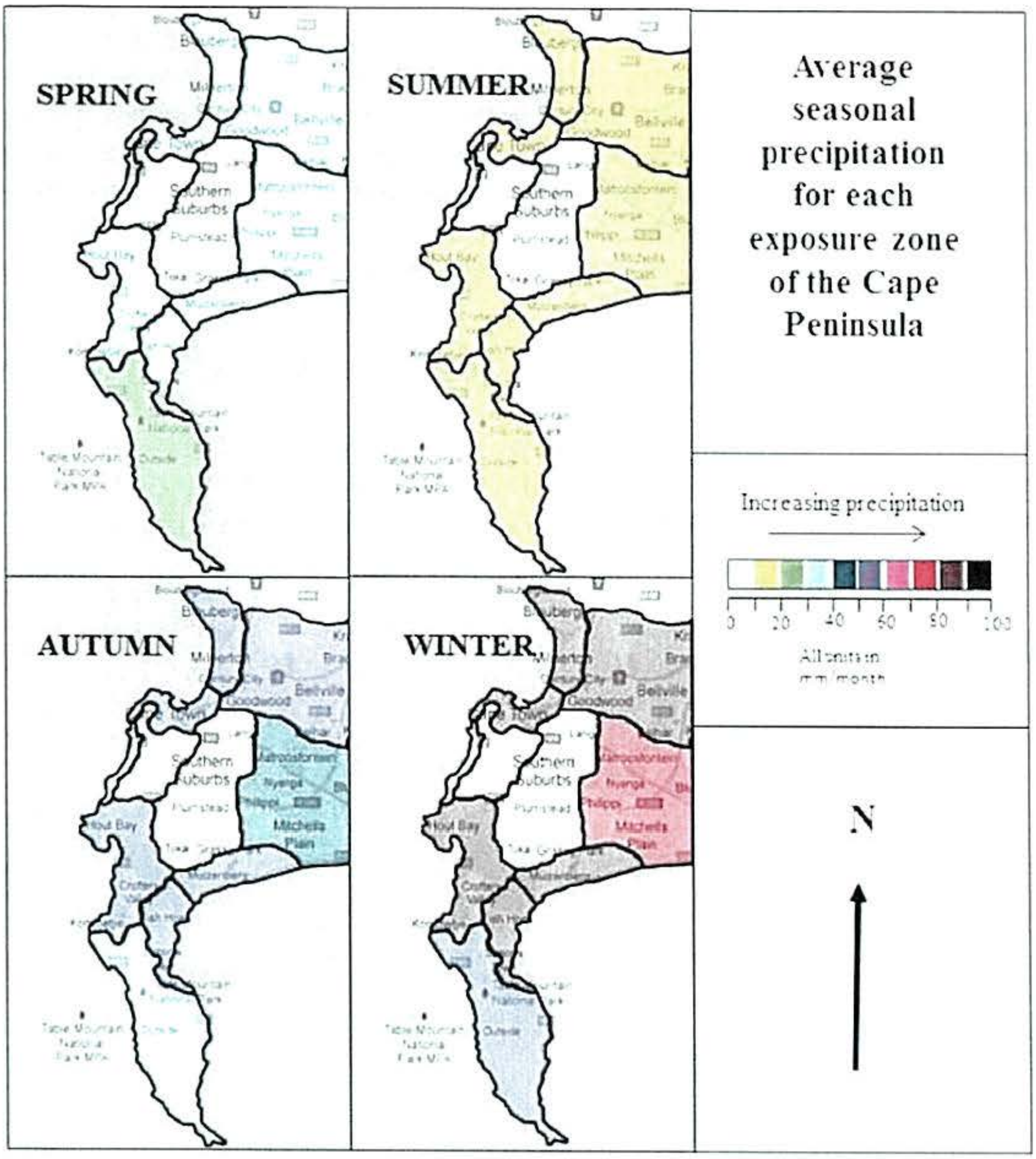


Figure 19: Average seasonal precipitation for each exposure zone (Ralfe, 2012).

3.5.3 Average Wind Speed and Direction

The Cape Peninsula experiences two main winds; during winter, the wind is described as a North-Westerly wind with an average wind speed classified as a strong breeze. In summer, the wind is Southerly or South Eastern, and is experienced over a longer period of time. This wind is normally referred to as the Cape Doctor. The Cape Doctor sweeps over the Peninsula and the Cape Flats,(Cowling , et al., 1996). Both winds blow over marine environments and are a huge contributing factor when classifying the exposure zones in different regions. The wind picks up moisture from False Bay waters and blows it around the Eastern flank of the Table Mountain range, pushing up against the slopes of Table Mountain creating clouds and rain. Thus the more lush greenery found on the mountain slopes and the Southern Suburbs.

Figure 20 illustrates the location of wind monitoring stations, these are indicated by the "wind vain" arrow, the website Windfinder displays live wind and weather updates and has historic data of wind direction and speed, precipitation and temperatures. It is important to highlight the four key areas in the Cape Peninsula to illustrate the type of weather and climate patterns experienced in these regions. These regions are Muizenberg, the Atlantic Seaboard, Cape Flats and the Tafelberg regions.

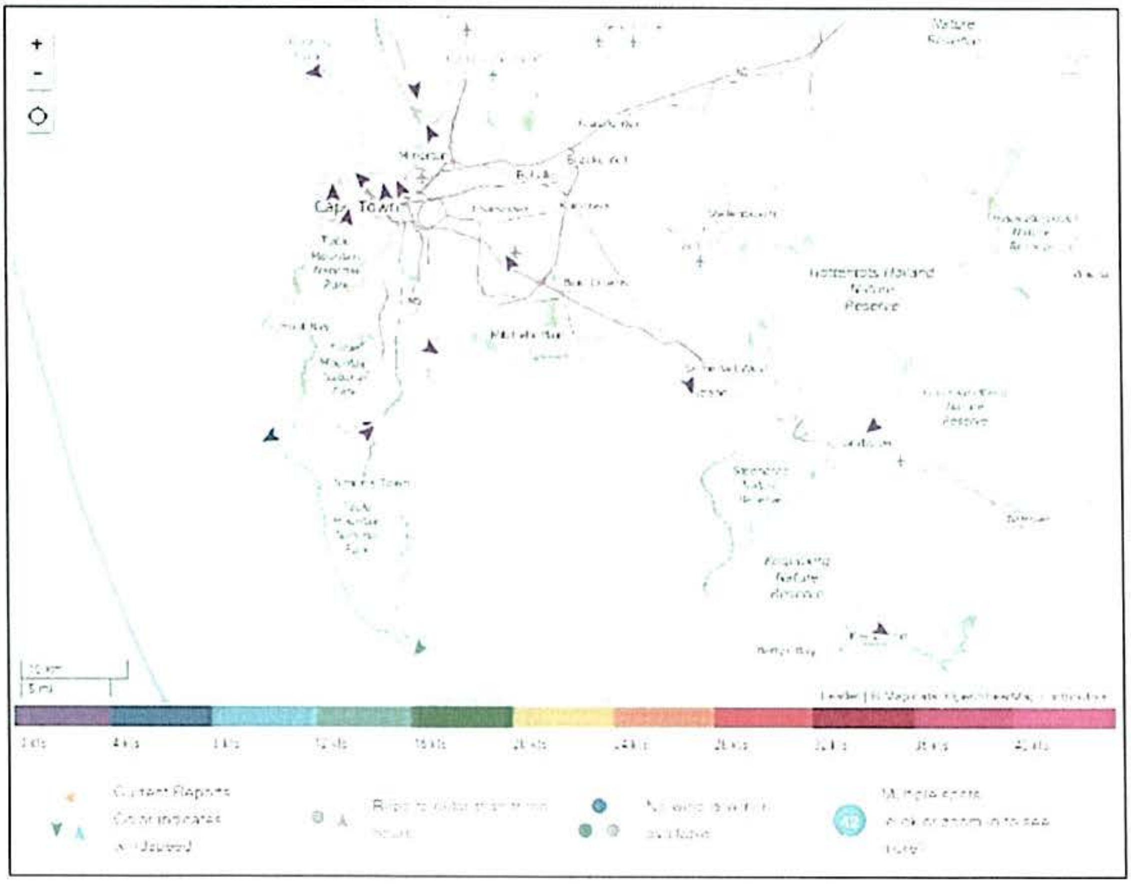


Figure 20: The wind monitoring stations in Cape Town (August 2016) indicating the wind directions and positions of the stations with an arrow, the colour indicates wind speed. On this particular day the wind speeds were relatively low (www.windfinder.com).

Muizenberg

Muizenberg is located next to the coastal area in the Southern region of the Cape Peninsula along the False Bay coast. This area experiences high off shore winds, which are accompanied by rough seas. Off shore winds are dominant during the winter months but less intense.

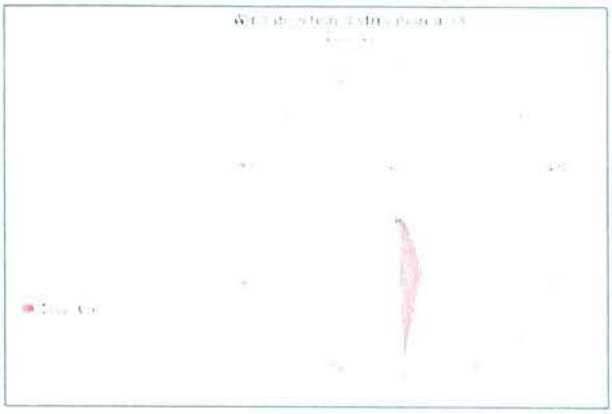


Figure 21: Wind direction, speed and Distribution during summer months at Muizenberg, which shows a predominate wind from the South (www.windfiner.com).

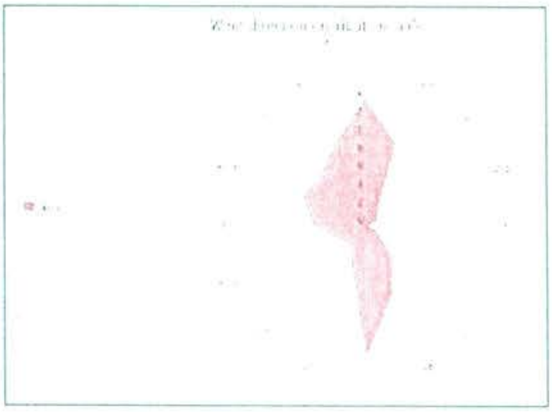


Figure 22: Wind direction, speed and Distribution during winter months at Muizenberg (www.windfiner.com)

Figure 23: The annual wind and temperature patterns recorded at Muizenberg beach (windfinder, 2016)

Atlantic Sea board: Sea Point

The region is located on the Atlantic coastline and located close the City centre and the main harbour. The majority of the city's buildings and infrastructure are located within 5km from the Atlantic Seaboard. The Atlantic Seaboard region experiences very rough seas during the winter months.

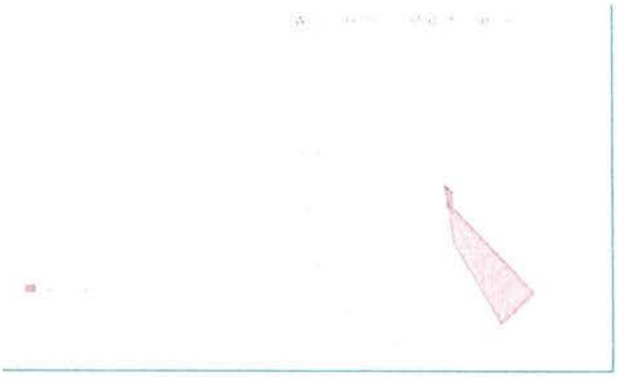


Figure 24: Wind direction, speed and distribution during winter months on the Atlantic Seaboard (www.windfinder.com).

Figure 25: Wind direction, speed and distribution during the summer month on the Atlantic Seaboard (windfinder, 2016).

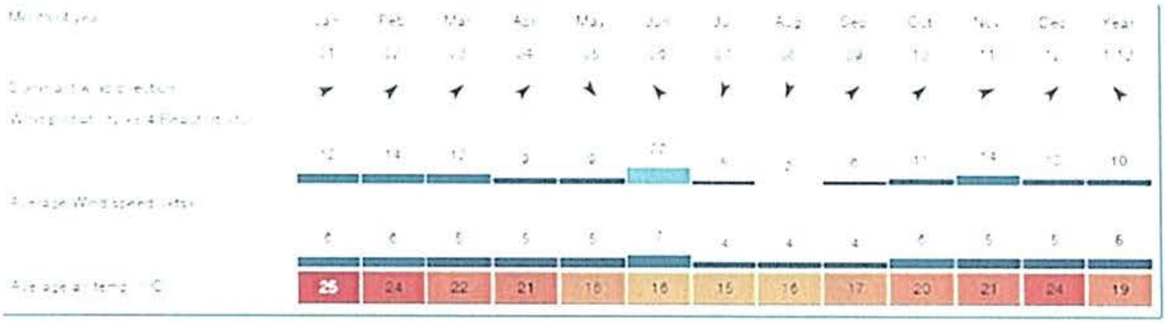


Figure 26: The monthly averages recorded wind data and associated temperatures on the Atlantic Seaboard. (windfinder, 2016).

The Cape Flats Region

This region is located between the Table mountain range and the Holland's Mountain ranges. It is generally flat and connects the Southern Cape Peninsula to the Northern Cape Peninsula. The wind blows through this funnel, originating from the False Bay (Muizenberg) region, sweeping across the Cape Flats, to Table View located on the Atlantic coast. This region of the city experiences the same type of wind speed and distribution experienced by the Muizenberg area because it is located close by, however the air contains less salt due to proximity from the ocean.

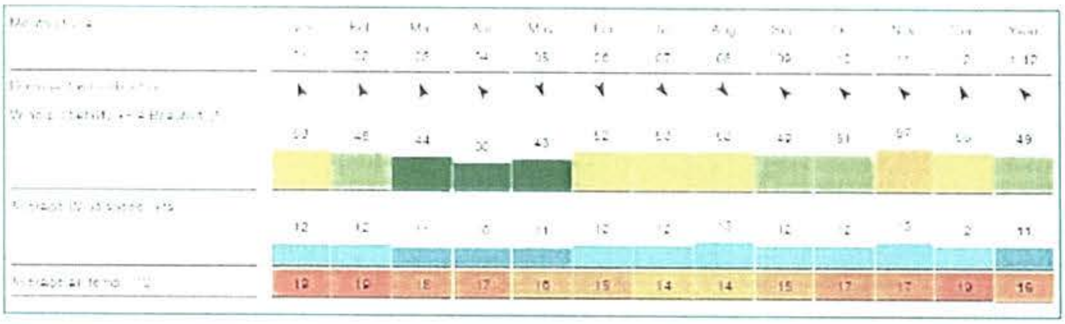


Figure 27: The monthly wind data and temperature averages recorded in Cape Flats region. (windfinder, 2016).

Wind Speeds

The general wind speeds recorded in the different regions display the type of wind and the intensity during different seasons. The average wind speeds over a year are shown below.

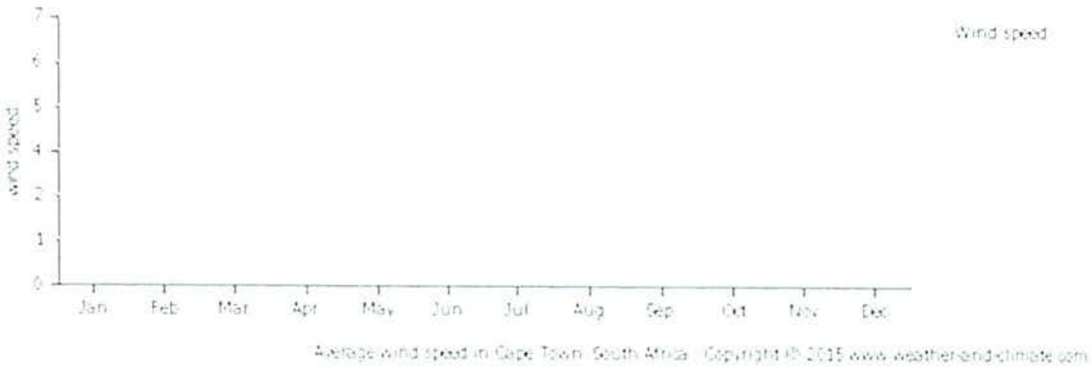


Figure 28: The monthly Average wind speed in Cape Town over a period of a year. (www.climate-and-weather.com).

The high winds experienced during the summer seasons contain higher concentrations of airborne chlorides. These winds cause the oceans waves to be greater than the normal winds blowing over the sea water to the shore (Cowling , et al., 1996).

The Cape Point region has the highest recorded wind speeds and accompanying higher recorded chloride ion content. The Muizenberg region experiences high winds and is located on the coast between the False Bay and the Atlantic Sea area. This region has high wave actions and associated higher airborne chlorides. The average wind speed during different seasons and per region is illustrated in the Figure 29 below.

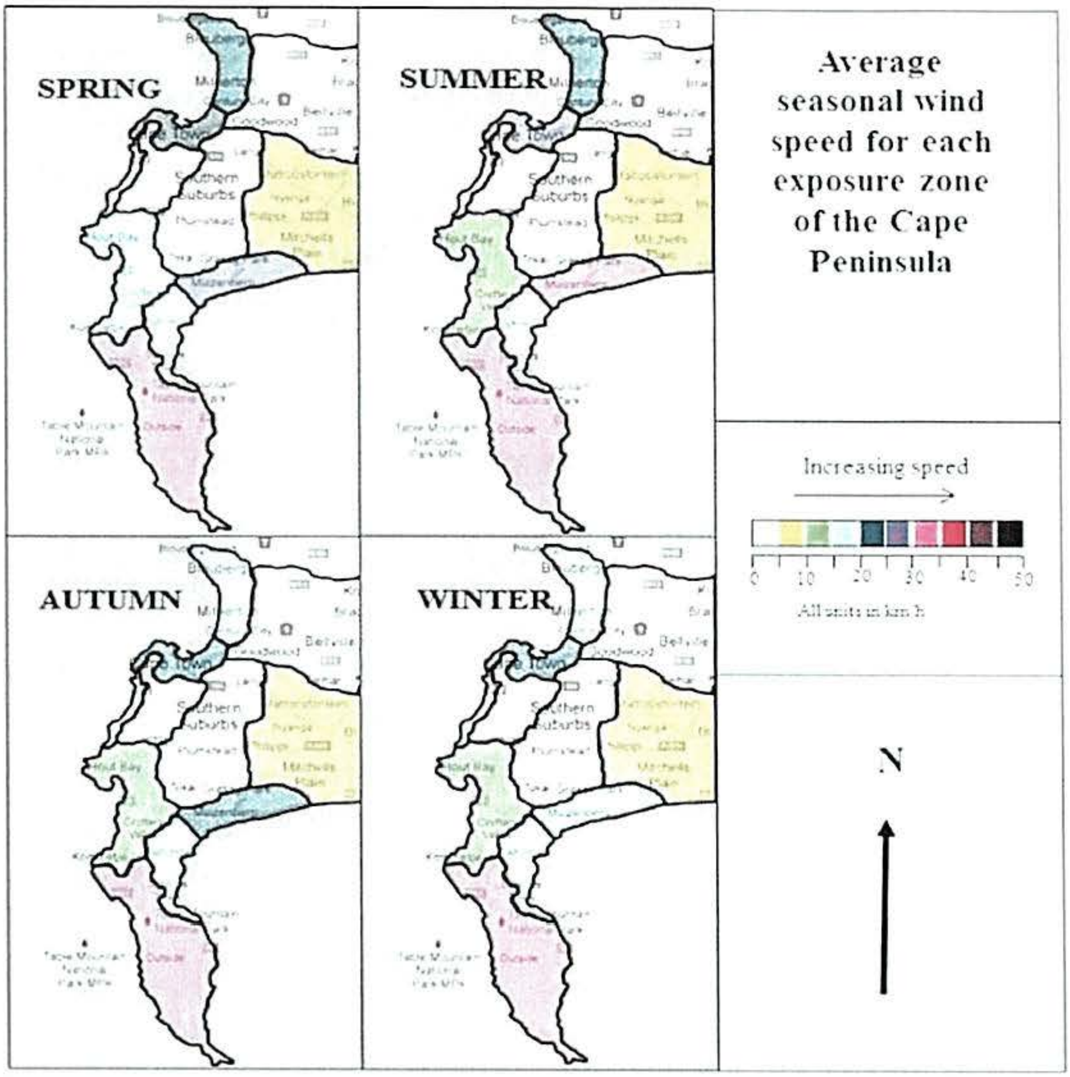


Figure 29: Average seasonal wind speed for each exposure zone throughout the year. (Ralfe, 2012)

3.5.4 Precipitation

The Cape Peninsula experiences lower precipitation during the summer months (November to February) – this is the dry season. Conversely, during the winter months (May to September), the region experienced higher precipitation with the highest rainfall during June. This is illustrated on Table 7 the average rainfall measurements recorded over 38 years. The general rainfall patterns in the Cape Peninsula indicate the rainfall increase and the associated rain days during the winter season. Figure 30 and Figure 31 show the relationship between rain days and precipitation. Conversely, the summer months are accompanied by less rain days and lower precipitation.

The weather patterns record in some regions of the Cape Peninsula and shows a correlation between the increased rainfall intensity compared to other regions. This is dependent on the location and distance from the sea, the type of season and the geographical layout. This is an attribute for microclimates found within the City.

Table 7: Average Rainfall – Cape Town Airport (38 years)
With a Catchment area 300m^2 (TCT, 2016)

Year	mm rainfall per month	Litres per month
Jan	11.60	3480
Feb	18.0	5400
Mar	22.10	6630
Apr	55.50	16650
May	76.70	23010
Jun	98.30	29490
July	96.90	29070
Aug	73.70	22110
Sept	41.70	12510
Oct	32.70	9810
Nov	13.70	4110
Dec	13.9	4170
TOTAL	**554.80**	**166440**

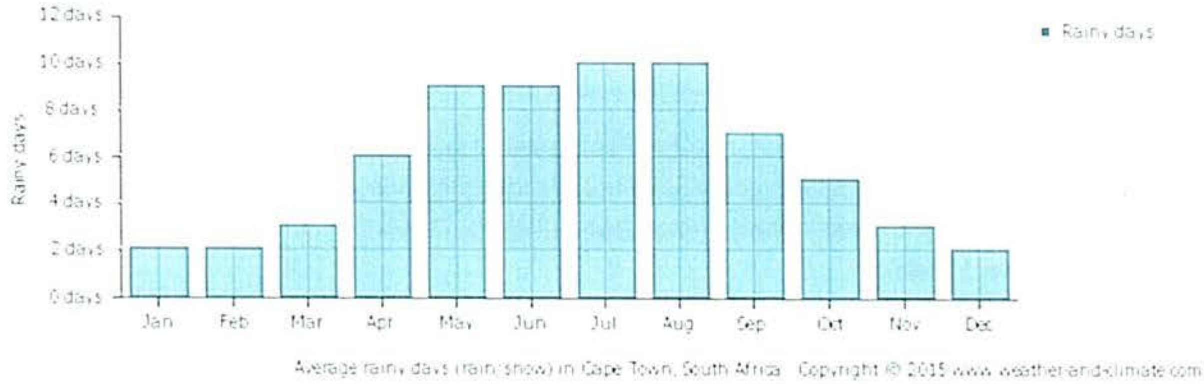


Figure 30: Average rain days per month in Cape Town over a period of a year.

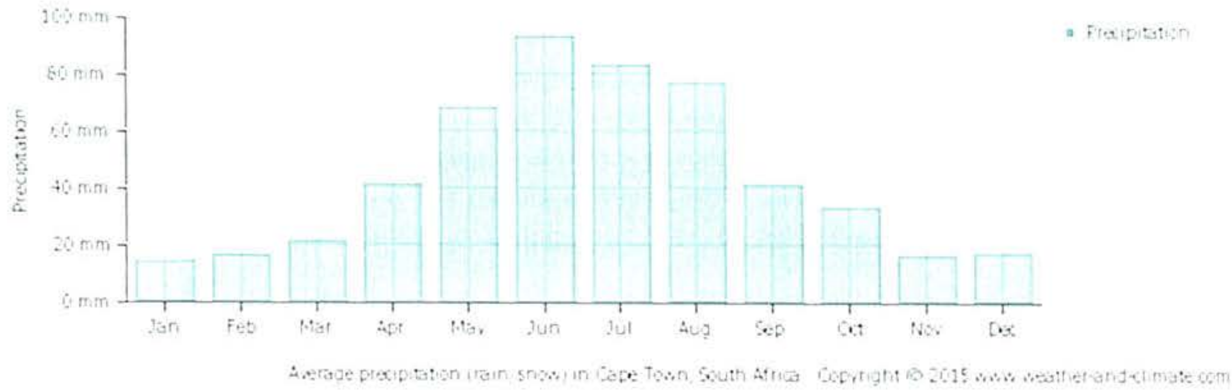


Figure 31: Average precipitation per month in Cape Town over a period of a year.

3.5.5 Relative Humidity

Relative humidity (RH) is the ratio between the amount of water vapour and the saturation water vapour density, expressed as a percentage. The temperature has an effect on water vapour in the air, the higher the temperature; the lower the moisture in the air, and conversely, a colder climate would have more moisture available in the air. Relative humidity is therefore a function of both moisture content and temperature. Relative humidity is derived from the associated temperature and dew point for the indicated time period (windfinder, 2016).

The vapour pressure is a measurement of the amount of water in a volume of air and increases as the amount of water vapour increases. Figure 32, illustrates the winter months, between May and September experiencing higher RH than the summer months.

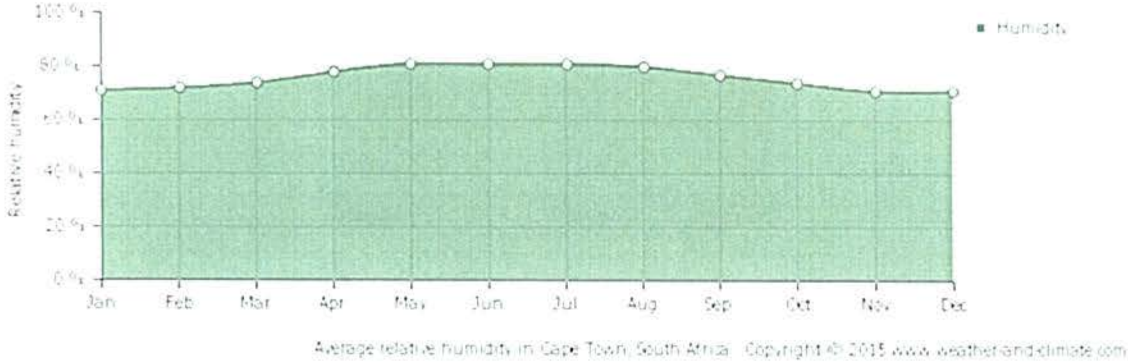


Figure 32: Monthly Average Relative Humidity over a year in Cape Town measured at Cape Town Airport.

The average relative humidity for the Cape Peninsula is around 78% (Alexander , et al., 2009). This is relatively high and is an indication of the amount of moisture in the atmosphere. Usually the region has an associated higher mean annual precipitation, which indicates an increase in relative humidity. (Cowling , et al., 1996).

3.5.6 Temperature

The average annual temperature range in the Cape Peninsula is between 15-27°C, Figure 34 shows the average monthly temperatures during the winter months ranging between 13-19°C. The higher temperature averages also corresponds to longer hours of sunlight per month, as shown in Figure 33.

Temperature and sunlight exposure has a great influence on the rate of concrete deterioration and should be considered during concrete durability design. The duration of sun exposure influences the rate the concrete dries after moisture exposure, and rapid evaporation may negatively affect the hydration of cement products, hence the concrete quality is influenced, affecting the structure's longevity and concrete durability. Other factors include the arrangement of concrete elements and environmental exposure in relation to the sun during the course of the day.

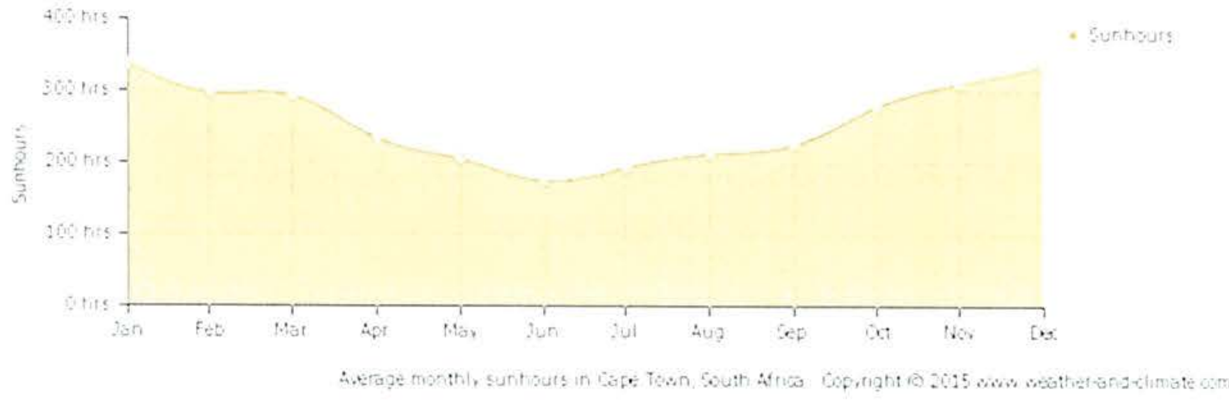


Figure 33: The recorded average monthly sun hours in Cape Town.

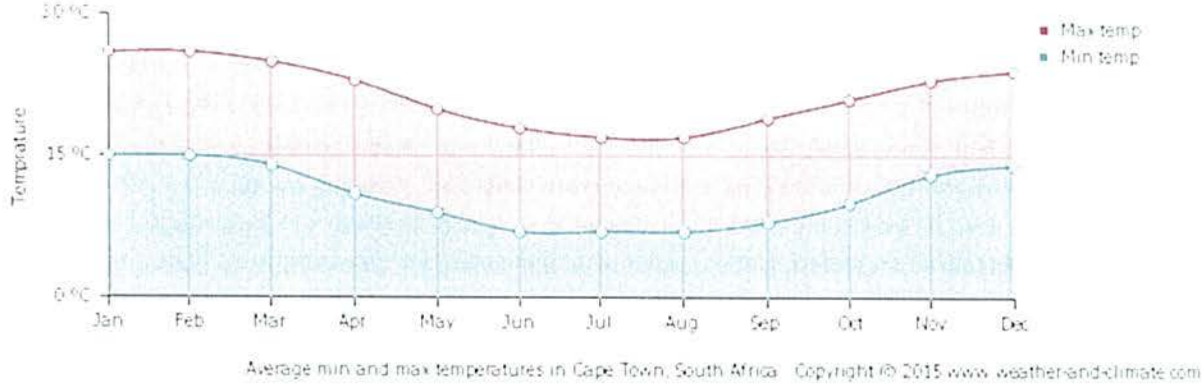


Figure 34: The recorded average min and max temperatures in Cape Town

3.5.7 Interrelationship of Environmental Conditions

The characterization of the climate and weather patterns of the Cape Peninsula is important for proper concrete durability design and understanding the expected deterioration mechanisms. It allows engineers to understand the interaction of the weather and exposure conditions, enabling engineers to design and construct structures that will withstand these harsh environmental conditions. The interrelationship of the seasonal weather should be considered by designers.

Structures build during the warmer seasons are exposed to very low relative humidity whereas during the colder, winter months the relative humidity is much higher. Furthermore, the associated precipitation experienced in Cape Town region is different in intensity and amount during the winter and summer months. This demonstrates that the Cape Peninsula can be divided into exposure zones, and information of these zone's air quality and weather conditions can be monitored and recorded by the monitoring stations. The exposure zones should be consistently updated and monitored by the City of Cape Town to assist in the development of viable concrete design standards and durability life models.

4 Investigation of Existing Structures in the Cape Peninsula

This dissertation has identified five RC structures in the Cape Peninsula, to showcase the types of deterioration and the extent of corrosion experienced in the region. The location of the reinforced concrete structure and levels of deterioration was analysed and assessed. This investigation used limited onsite inspections, non-destructive condition assessments and information assessed in previous dissertations.

4.1 Chloride Concentrations

In 2015, the chloride concentrations of selected reinforced concrete structures located in the Cape Peninsula was measured and studied (Alao, 2015). This study concluded the relationship between factors such as: the distance from the sea, the orientation of structures in relation to the sea and then the level of chloride concentration that was assessed (Alao, 2015).

When the chloride concentration threshold is exceeded, it creates prefect conditions for rebar corrosion. The penetration of the chloride ions into the concrete structure is a huge problem. The determination of chloride concentrations would assist engineers to identify the condition of concrete members and predict corrosion. The concrete surface is gradually penetrated by the sea air increasing the concentration of chloride ions in the concrete substrate. Knowing the depth of chloride penetration would be useful to determine the time it takes the chlorides to reach the passivation layer of the steel and to predict rebar corrosion. The corrosion would eventually cause concrete cracks, spalling and other defects.

The simplest method to collect and analyse chloride samples, was the drilling into concrete and the collection of the dust, and testing the recovered material (Alao, 2015). The chloride concentration was determined in the laboratory using chemical titration methods. The classification of environmental exposure in relation to chloride attack can be divided as follows, structures located within 0-3 km from the sea, are classified as very severe, while regions within 3 -14 km from the coast are classified as moderately exposed to airborne chloride attack and finally, the distances >14km has an insignificant environmental exposure effect by airborne chloride attack.

In order to classify the severity of chloride exposure in microclimates, the concentration of airborne chlorides must be measured for the existing structures in locations around the Cape Peninsula. Below is a table of the chloride profiles measured by Alao:

Table 8 : Summary of Information on RC structures sampled and analysed in ALAO's study.

Location of the Structure	Structural Component cored	Distance from the sea (m)	Estimated age of the Structure (years)	Average surface chloride content Cs (% by mass of binder)	Orientation
Glencairn Simonstown	Retaining Wall	50	50	2.51-4.5	Sea Facing
Foreshore Freeways	Balustrade	300	40	0.44	Non-sea facing
Foreshore Freeways	Bridge Pier	250	40	0.48	Sea Facing
Helen Suzman Blvd Bridge	Barrier	800	40	0.61	Sea Facing
Helen Suzman Blvd Bridge	Barrier	800	40	0.31	Non-Sea Facing

4.2 Cover Meter Survey

A cover meter survey was conducted to measure the concrete cover of selected RC structures in order to gain a better understanding of the influence of the cover depth and concrete deterioration. The depth of concrete cover is important to ensure that rebar is maintained at a certain depth. The concrete cover will protect the underlying steel from carbonation, the ingress of water and chlorides ions. Conversely, reinforced concrete elements with excessive concrete cover are problematic as it increases crack widths of deteriorated concrete. Thus, the importance of proper concrete design is dependent on the structure's environmental exposure conditions and purpose.

A cover meter was used to measure the cover depths; this cover meter was an electromagnetic apparatus which uses an electric current in a coil winding in the handheld head to generate a magnetic field; which interacts with the embedded steel. This interaction determines the steel's electric conductivity and its magnetic permeability, as a magnetic field propagates back to the head containing a second coil and is able to detect and measure this difference. The higher the magnetic field reading the greater the bar size and decreases with an increase depth of the reinforcement.

By making certain assumptions about the embedded rebar and assuming there is only one bar, the cover meter is able to interpretative the readings and indicate the cover depth. A skilled operator using sophisticated equipment can map the reinforcement, determining bar position and orientation determining the bar diameter, and determine cover depth.

This dissertation investigated the depth of the reinforcement of structures in Table 9. The structures are included in the case study section and have either been recently repaired, newly built or requires rehabilitation intervention. The cover readings were taken of different elements to illustrate the effect or defects of certain concrete elements on the same structure. RC structures either have design faults or practical problems. The depth of the concrete cover was measure on accessible concrete elements, the measurements are tabulated below and a full discussion provided in the individual case studies.

Table 9 : Cover Depth measurements on selected RC structures in the Cape Peninsula

Structure Name	Concrete Member	Cover Measurements (mm)	Condition of Member	Comments
Mike Pienaar Footbridge	Pier	20	Minor spalling	Located in the Northern region of the Cape Peninsula.
	Deck coping	0-20	Extensive spalling	Corrosion caused by carbonation
Sea Point Sea Wall	Bollard (old design – containing steel rebar)	20-25	Cracking and rust staining	Bollards that are older than 10 years
	PreCast Sea Wall Panels (new)	50	No signs of deterioration	Recently constructed (2015)
Alma Road Bridge	Balustrade	0-20	Extensive spalling and rebar corrosion	
MYCiti Bus Concrete Roads	Concrete Road	35	Minor shrinkage cracks	Constructed 2010
Foreshore Freeways	Balustrades	0-25	Extensive cracking and spalling with signs of rebar corrosion	Bridge is 40 years old and close proximity to the sea

4.3 RC Structures in the Cape Peninsula

To better understand the common types of concrete deterioration experienced in the Cape Peninsula a number of structures has been identified and assessed. This section investigates reinforced concrete structures located in the Cape Peninsula region, highlighting the environmental exposed conditions and its associated deterioration effects of the concrete structure. These aging structures were designed and constructed with normal steel rebar with no form of corrosion mitigation measures.

City engineers have standardised the use of HDG reinforcement for all RC structures undergoing repair and rehabilitation. In addition, all new concrete handrails and new construction projects are specified with HDG rebar.

The following are examples of concrete repair and rehabilitation projects in the Cape Peninsula. These RC structures were in the early stages of rehabilitation or have been recently completed. This case study also includes some structures that have been identified for future maintenance.

4.3.1 Mike Pienaar Pedestrian Footbridge

Mike Pienaar Footbridge is a reinforced concrete pedestrian bridge located in Bellville, the Northern region of the Cape Peninsula. This is an important pedestrian bridge which facilitates the movement of the public to the nearby school and hospital crossing over the busy Mike Pienaar roadway.

The bridge showed signs of concrete cracking and spalling on the deck copings, only after a number of months of falling debris onto the roadway had the bridge been reported to the roads department. The concrete pieces dislodged from the existing coping, and fell into the road way, posing a major risk to the safety of road users. The deterioration of the concrete cover was attributed to being less than 20mm. A bridge condition assessment (Roux, 2015), concluded that carbonation induced corrosion was the main cause of concrete deterioration, supported by the appearance of minor rust staining and corrosion that only occurred on elements with low concrete cover.

This footbridge required emergency concrete repair works with immediate make-safe measures to reduce any risk to public safety. The repair works included the replacement of the deteriorated concrete copings which included a redesigned rebar arrangement, and the use of HDG rebar to compensate for the lack of concrete cover and increase corrosion resistance. The concrete quality was improved by using class W50 concrete (50 MPa) strength and better construction and curing methods. A final concrete coating system was applied to the entire bridge to provide an extra layer of protection from environmental exposure.

Figure 35: Mike Pienaar Footbridge before (right) and during concrete repair works (left) (2015).

4.3.2 Sea Point Sea Wall

The Sea Point Sea Wall was constructed to protect certain parts of the Atlantic shoreline from extreme waves and tidal action. The Sea Wall also reduced the effect of seaweed that washed onto the shore and decayed, generating an undesirable stench in the area. The wall stretches 3.2 km starting from Granger Bay to Saunder's Rock. The Sea Point Sea Wall is 85 years old and was in dire need of extensive

Figure 36: Rebar corrosion of a bollard on the sea point sea wall. (2016)

rehabilitation and upgrading. The existing wall was constructed with granite blocks and the back of the wall filled with spoil material, then levelled to form walkways, lawns and recreational spaces. However, these stones dislodged from the seawall and the cement dissolved through abrasion and wave action.

The phased rehabilitation program started in 2012 at a cost of R 19.3 million (City of Cape Town, 2013), part of the rehabilitation included the replacement of the existing sea wall. The most adequate solution employed was the construction of new precast wall panels, installed on solid foundations and anchored back into compacted fill behind the wall. The seawall's precast panels had 50 mm concrete cover, a concrete strength of 50 MPa, a concrete mix containing 30% fly ash, the use of HDG rebar and the anchors, these were specified due to the harsh marine exposure conditions. The specifications would ensure that the durability standards specified in the design were met and prolonged service life. Precast elements were produced at factory level of standards with no defects accepted on the final panels. Any concrete panel with low cover or poor compaction was rejected. The concrete façade of the panels mimics the previous granite block design and arrangement to retain the original aesthetics.

Furthermore, the City of Cape Town traditionally treated the balustrades and railings of the Sea Wall as consumables. This meant that the balustrades are regularly replaced by the closely local maintenance depot, when and where it was needed. This was a low-tech solution that worked quite well on the Sea Point seaboard. On average, the balustrades lasted 20 years before the concrete deteriorated and the rebar corroded and had to be replaced. The balustrades are set in a weaker concrete base in the existing coping and workers can be easily replanted and replace bollards (Mackie, 2016).

The improved design required more durable concrete upstands and balustrades, the new members replaced the steel rebar with FRP bars and only anchored with a single HDG bar (Vink, 2017). This is an improvement over the city's traditional maintenance approach and a better engineering solution as it would reduce repair and maintenance costs.

4.3.3 De Waal Road Bridge

The De Waal Road Bridge has recently undergone concrete rehabilitation and repairs. The De Waal Road Bridge is located in the Southern Suburbs (Diep River) of the Cape Peninsula. The bridge crosses over the Southern Suburbs railway line, this railway line connects the regions of the East with the Western residential areas in the City.

Figure 37: De Waal road bridge showing signs of concrete spalling and rebar corrosion with inadequate concrete cover. (taken 2016)

The De Waal Road Bridge rehabilitation project consisted of the installation of new concrete precast handrails, installed on the existing concrete parapets and the repair of existing concrete elements that has rebar corrosion, concrete cracking and spalling. It was evident that the bridge elements not exposed to sunlight experience more moisture retention and therefore a greater degree of concrete deterioration and corrosion.

The installation of new concrete handrails forms part of a city-wide rehabilitation program, this program replaces any vandalised steel or aluminium handrails with precast concrete handrails. The concrete handrails are precast units which are adjusted slightly depending on design requirements and existing bridge conditions. The handrails are installed on Y20 rebar anchors and epoxy grouted into place. The concrete is specified to be high strength to ensure durability; this will increase the concrete density and refinement of the concrete microstructure. All the rebar in the handrails are HDG and has a minimum concrete cover of 40mm.

The final part of the rehabilitation required a concrete coating system applied to all the concrete elements, both new and existing sections of De Waal Road Bridge. This will provide additional protection and improve the durability of all concrete elements. In addition, the coating improves the aesthetics of the bridge, blending the repaired areas with the existing bridge elements and the new precast concrete handrails.

Figure 38: A precast concrete handrail delivered and installed on site. The handrails are steam cured and wrapped in plastic before delivery to site.

4.3.4 Alma Road Bridge

Alma Road Bridge is a small road bridge located in the Southern Suburbs; the bridge crosses a section of the Liesbeek River Canal. The existing bridge balustrades had very low concrete cover and the quality of the concrete was very poor and very porous. The cover depths measured were very low, between 0 -20 mm. The concrete elements displayed signs of concrete spalling and extensive rebar corrosion. This type of corrosion has been caused by a combination of chloride and carbonation exposure. City engineers decided that it would be too costly to apply concrete patch repairs and cement coating systems. The only viable solution was to demolish the existing bridge balustrades and replace them with an improved balustrade design with an appropriate concrete cover, adequate concrete quality and the use of HDG rebar. The rehabilitation and repair of Alma Road Bridge is at the project initiation phase and requires funding approval, to appoint a contractor. The rehabilitation of this bridge is scheduled for October 2017.

Figure 39: Extensive concrete spalling on existing concrete balustrade of Alma Road Bridge (Fatima Adams, July 2016)

4.3.5 Foreshore Freeways

The Foreshore Freeways was built in the 1960s, the freeway bridges form part of a greater elevated highway system that was planned for the city, a highway system would link the road network through the City and over the Foreshore precinct. However, due to the lack of funding and difficult economic times, the highway bridge network was not fully completed. Some sections of the highway bridges are still in use today.

During its lifespan the bridges has undergone minor concrete repairs on the bearing seats and the support column caps but not on the existing concrete balustrades. The concrete balustrades exhibit widespread concrete spalling and extensive corrosion of the underlying steel rebar. This type of deterioration is due to low concrete cover and the freeway's close proximity to the sea, increasing the risk of chloride attack. The existing Foreshore Bridges has about 9 km of bridge balustrades and the large majority of these balustrades has extensive signs of concrete deterioration and rebar corrosion (Roux, 2008). The chloride concentration measurements of the concrete balustrades, measured by Alao

(2015) observed high chloride concentrations close to the rebar. The balustrade has a concrete cover depth of 30mm.

City engineers have identified the Foreshore Freeway bridges for future rehabilitation with significant resources to be spent on the repair of the concrete balustrades. The rehabilitation will include proper condition assessment of the bridge structure; the selection of appropriate concrete repair solution and the replacement of corrosion resistant rebar. The rehabilitation would consist of concrete crack and patch repair, including the application of corrosion inhibitors to the concrete surface and a final concrete coating system.

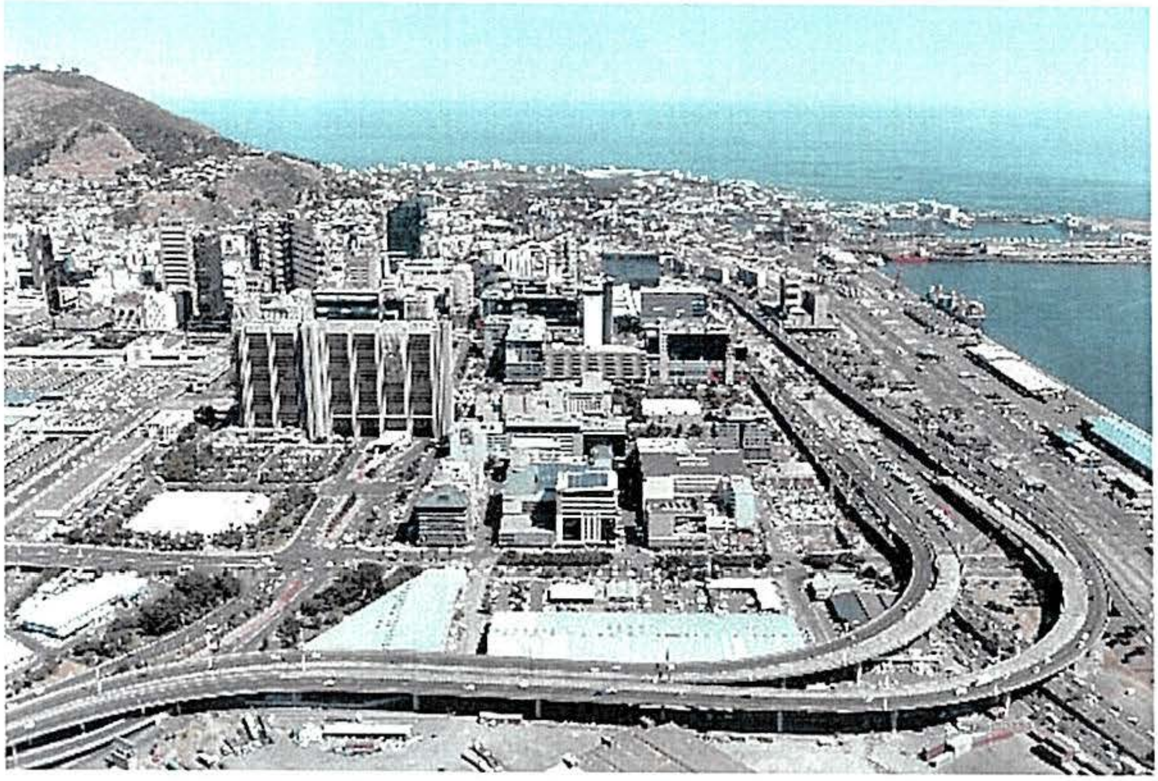


Figure 40: Foreshore Freeways consists of 9 km of Roadway Bridge, it is located at the edge of Duncan Docks less than 500m away from the harbour and links the Nelson Mandela Freeway to the edge of the CBD of Cape Town. (Bruce Sutherland, 2009)

Figure 41: Concrete spalling on existing balustrade of the Foreshore Freeways, Cape Town (2015)

Figure 42: Extensive spalling and rebar corrosion on the Foreshore Freeways, Cape Town (2015)

4.3.6 MyCiti Bus Infrastructure

The MyCiti Integrated Rapid Transport (IRT) network is a dedicated bus route currently being constructed on existing roadways, which provides an improved public transportation for the City of Cape Town. The road upgrades coincided with the hosting of the FIFA Soccer World Cup 2010, and ever since has been rolling out to across the City. The MyCiti network infrastructure and routes will continue to rollout across the city for the next 20 years. The dedicated bus routes are constructed with continually reinforced concrete roads and the bus stations constructed every 500m along the route. A red pigment is added to the concrete road to differentiate the busway from the normal roadways.

The structural elements of this network include the bus station's substructure and 35 km of reinforced concrete roads (Kobus, 2016). The reinforced concrete roadway was constructed to withstand heavy loading, to minimise continual maintenance and repair works. The busway was constructed from continuously reinforced concrete and is designed for a 40-year lifespan; however it should be noted that the reinforcement is not hot dipped galvanised. Some engineers believe that this is a design fault.

The bus stations are located very close to the coastline, especially along the West Coast road and the Atlantic Sea Board. The bus stations are constructed with elevated platforms, to allow users access to the high floor buses. This meant that the design and construction of RC sub-structures with HDG rebar. All the concrete elements has a concrete strength of 50 MPa and 40 mm cover depth. This type of concrete design would perform well in the exposure conditions found in the Cape Peninsula.

Figure 43: Concrete used for the bus way and the bus stations (City of Cape Town , 2007)

5. Challenges Faced by City Engineers

The Asset Management and Maintenance department is responsible for the repair, rehabilitation and maintenance of all road supporting infrastructure. This includes reinforced concrete bridges, subways, culverts, stairways, retaining walls, coastal walls and slipways. City engineers face many challengers that negatively impact on service delivery. These challenges range from political influences, shortage of technically skilled professionals and support staff, inadequate training of staff in management roles, long and cumbersome tender processes, and implications with the lowest contract bidder appointments.

The number of competent civil contractors with adequate knowledge is few, with smaller contractors lacking skill in concrete repair solutions and concrete works. City engineers do not have the necessary support from department heads to facilitate regular maintenance projects. Most of the maintenance project funding are siphoned from larger capital projects, these capital projects more politically motivated and well-funded. These are some issues that limit the engineer's resources and restrict service delivery, and delaying scheduled repair works. In many cases, insufficient funding is available for the required maintenance and rehabilitation projects.

In addition, the city has lost the capabilities of its own maintenance depots, especially in a concrete repair capacity. This makes the city incapable of preforming large scale concrete repairs and rehabilitation on its own assets. Thus, all consulting expertise and concrete works are outsourced to companies with the appropriate experience and technical skills, through the city's tender process, increasing the costs of rehabilitation projects and the time period needed from inception to project completion.

One of the management tools available to city engineers is the Bridge Management System (BMS), however, it is not being used to its full potential. The BMS has not been maintained with current conditions of city bridges, culverts and other reinforced concrete structures for 14 years. The BMS must be implemented and populated back to a desirable and effective level, achieving this would require huge amount of money and time and a dedicated technical team. This team would be tasked with the inspection of all the bridges, retaining walls and other structures located in the Cape Peninsula. This information must be populated into the BMS system. It would also require the use of a universal DER rating system, like SANRAL's bridge inspection standards, and the bridge condition assessments.

Most of the city's structures are reaching the end of their design life and are showing severe signs of concrete deterioration, with no immediate repair intervention scheduled. The bridge management system has not been updated as there are no internal bridge inspectors to rate the reinforced concrete structures condition. This lack of BMS information on the condition of the bridges reduces the efficiency of proper bridge maintenance. Currently, the maintenance of these structures is only initiated once there is extensive deterioration or failure. Due to this ineffective method of "project initiation" the processes to get a contractor to repair or rehabilitate are far too long and cumbersome.

The lack of engineers and management tools puts the city at a huge disadvantage. Projects are not selected due to their need to be maintained according to the degree of deterioration. As all repair projects are usually at a critical level before they are selected to be rehabilitated.

The Asset Management Department has a low staff contingent and inappropriate technical skill and expertise level, The lack of experienced staff in concrete repair and rehabilitation technology is a wide spread problem in municipal institutions. This problem forces maintenance departments to outsource work to consultancies and contractors. This increases the costs of a rehabilitation project and increases the time from project initiation to the procurement phases to project excursion. The level of technical skill and professionalism of civil contractors appointed may be also lacking as well as the relevant experience missing due to tenders being awarded to contractors based on the lowest tender prices.

Furthermore, political influence has an impact on the projects are prioritized and repair projects require more funding overruling engineer's recommendations. It is a challenge experienced by all local authorities. If these issues are addressed the rehabilitation and repair of concrete structures would function more effectively.

6. Conclusions and Recommendations

City engineers are responsible for design, repair and rehabilitation of reinforced concrete structures; these structures require routine maintenance or reactive intervention. These efforts are hampered by a lack of engineers and technical staff at the City of Cape Town's maintenance departments, including few skilled workers with enough experience in concrete repair, concrete solutions and asset management. The City requires the use of a well-maintained BMS system, which would be very useful as a management tool, saving the city time and money.

The implementation of a proper BMS would initiate rehabilitation or repair projects in a systematic manner, shifting from the current reactive to a more proactive repair intervention. The BMS system is a tool which requires skilled personnel for effective use. Once the BMS is operating at an acceptable level, the scheduling of concrete repair and reinforced concrete structural rehabilitation can be determined by a management tool and would be accessible to all technical staff.

On a technical level, engineers should be able to conduct condition assessment of structures with a universal structural condition rating system. A condition rating system should be used, similar to the DER system, used by SANRAL. This system will identify the concrete deterioration mechanisms experienced by the structure, allowing for the selection of an appropriate concrete repair solution.

Concrete repair and rehabilitation solutions must be designed and priced in accordance to product specifications and design standards, to insure an increased level of concrete durability and structural longevity.

City engineers must have the technical ability and expertise to design new structures and formulate concrete repair solutions The designs would include the concrete quality, concrete cover depth, the type of curing, rebar arrangement and rebar material selection with regards to environmental exposure conditions, concrete durability standards and concrete specifications in line with current design life models.

Undoubtedly, the need to protect reinforced concrete structures from corrosion is highly important in the Cape Peninsula; city engineers are well aware of the level of protection required for longevity of these structures. Engineers should use corrosion resistant materials that are not only exclusive to HDG rebar as a "standard" for all structures. This is "over design" and not necessary for all applications. In most cases, the use of HDG rebar is not needed, especially for structures located more than 5km from the coastline, also HDG may not always be required on structures with good concrete quality. HDG rebar and alternative corrosion resistant metals are not a replacement for poor quality concrete. Engineers should place more emphasis on concrete quality, concrete design and good construction.

The use of alternative rebar materials other than that of HDG exists, however they are not readily available in South Africa. These alternatives have advantages and limitations that need to be better understood by city engineers. Currently, the use of HDG is relatively cheap in South Africa, its use as rebar is standardised, and it is an industrial standard. The use of stainless steel would be unfeasible, as it is no longer produced as rebar in South Africa. FRP rebar technology is improving and becoming more widespread with improvements in production and construction methods. FRP rebar has already been specified for lightweight concrete elements and retrofitting of existing RC structures. It is just a matter of time before FRP would become more feasible for conventional construction projects in South Africa.

The City would save 10-15% on the cost of the reinforced concrete members if normal steel rebar is used instead of HDG rebar. HDG rebar is currently specified on the citywide Concrete Handrail Replacement Programme. These handrails have a high level of quality and an insignificant structural importance. Most of the bridges undergoing this rehabilitation program are not located within 5 km from the ocean; in addition these handrails have a design life of 30 years with an extra application of a concrete coating system on the handrail. If the HDG rebar were to be replaced with normal black steel rebar, it is highly unlikely to experience corrosion within the 30 years. The individual precast concrete handrails can be treated as a "consumable" and may be replaced whenever they are damage or experience deterioration; some road authorities have maintained this to be a better approach.

All rehabilitation projects require sound engineering judgment to determine the correct use of HDG rebar, but this judgement must be based on factors such as the location of the structure, the structural loading conditions, construction methods and quality standards, to ensure concrete durability and structural longevity. Correct rebar placement and concrete quality can eliminate the need for the use of corrosion resistant alternatives such as HDG rebar. However, currently in the Cape Peninsula, the use of HDG would be the best solution for structures within 5km from the coast; any other application would best be specified to be normal black steel. In the Cape Peninsula, common applications for black steel reinforced concrete members would be the concrete bollards (sea Point Sea Wall), bridge handrails and balustrades (Concrete Handrail Replacement Program). These concrete members would be treated as consumables, being replaced once steel corrosion or concrete deterioration occurs, with a design life of 20 years. This approach is more suited to the City; it would reduce initial project costs and would be more politically favorable to create permanent jobs. The "consumable" concrete products are managed by the city depots, hence reducing contractual costs and expensive corrosion resistant measures. However the city has realized the benefit of using corrosive resistant materials and has incorporated this in the new rehabilitation project. The City has incorporated the use of FRP bars to increase the durability of these bollards and will prove to be a better engineering solution.

In conclusion, engineers should use sound engineering judgment and knowledge for all corrosion mitigation measures and alternative corrosion resistant material selection for both design and construction of RC structures. The design and repair solutions should be supported by a combination of available National Standards, prescriptive and performance based design approaches. The adherence to concrete durability specification targets for all RC structures would increase the longevity of civil infrastructure. Municipal engineers should shift from the current attitude of "blind" over-design and implement more effective solutions by adopting a more sophisticated concrete design methodology and repair strategy.

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