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Repair and Rehabilitation of Reinforced Concrete Harbour Structures

By
Andrew George Rowan
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Department of Civil Engineering
University of Cape Town

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Declaration

I, Andrew George Rowan, hereby declare that this thesis is my own work and all contributions have been referenced accordingly. This thesis has not been submitted in full or part to any other university. I know the meaning of plagiarism and declare that all of the work in the document, save for that which is properly acknowledged, is my own.

Signed by candidate

Andrew George Rowan
February 2007

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- To the Creator God and my Saviour Jesus Christ

ABSTRACT

Damage to reinforced concrete infrastructure due to chloride-induced corrosion is widespread throughout the marine environment in South Africa. This thesis is an investigation into four current concrete repair contracts at harbours in the Western Cape. The works are critiqued in terms of repair philosophy and methodology, and recommendations are made for improving practice.

A literature review is presented, outlining the relevant background to the chloride-induced corrosion of reinforcing steel, specifically in the marine environment. Damage assessment tools and techniques are also presented, and the different repair options that are most common in practice are discussed. The contract documentation for the four contracts is reviewed, and it is highlighted that while the bulk of the project specification is identical, the major differences in the documentation from the four contracts are in the quality and level of detail of the construction drawings. The individual repair methods chosen for various concrete elements are described in detail and commented on in terms of concrete durability. Forensic testing results in the form of chloride profiling and corrosion inhibitor testing at two locations are presented.

This information is drawn together and discussed firstly from a forensic testing viewpoint, and then critiqued on the basis of repair methodologies. The four sites are compared and recommendations are made to improve practice under each 'phase' of a repair contract. The differences in the approaches taken at the four sites are evaluated and commented on. It is evident that there is a difference in philosophy of quality and value in current repair contracts, with some engineers being willing to invest time and money in understanding material behaviour whilst others are satisfied to perform minimum repairs, depending on traditional methods, despite modern advances in knowledge.

While it is evident that forensic testing and repair trials are not commonly being performed on site, the value of such measures in informing damage assessment and repair method choice is shown. It is recommended that forensic testing be included as mandatory in new contracts. An 'anatomy' of a repair contract is presented for consideration for new repair contracts, highlighting the key factors that must be addressed to ensure a high quality repair that also provides value for money.

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1 Introduction

1.1 Background

This thesis addresses the repair and rehabilitation of reinforced concrete structures in the marine environment. In order to give some insight as to how the aspects concerning concrete repair are applied in current practice, it was decided to investigate current repair and rehabilitation contracts in Western Cape marine environments. The Repair and Maintenance Programme (RAMP – explained in 1.2) was selected to provide this data as not only was it a series of current repair projects that mainly addressed damage due to steel reinforcement corrosion, it also presented the opportunity to look at the approaches of a number of different consultancies.

Four locations were available for investigation in this work:

- Hout Bay Harbour (R13 Million repair contract)
- Saldanha Bay Harbour (R10 Million repair contract)
- St Helena Bay Harbour (R11 Million repair contract)
- Laaiplek Harbour (R8 Million repair contract)

The 'base' data for these contracts came from the documents and drawings obtained from the contracted consultancies, as well as information from other sources such as discussions with the relevant engineers. At two of the locations in particular, namely Saldanha Bay and St Helena Bay, there was an opportunity to perform forensic testing on some of the deteriorating structures, and this is presented in chapter 6.

While all these contracts are under one 'banner' being the RAMP contract, it is important to note that they are all performed by different consultancies and contractors. Each consulting company had an individual style of addressing the respective problems. By linking to the literature review presented in the next chapter it will also be possible to see the range of levels of deterioration in each of these structures as well as the many repair options that are covered, from 'do nothing' to 'demolish and reconstruct'.

1.2 Overview of RAMP

The Repair and Maintenance Programme (RAMP) was an initiative of the Department of Public Works (Marine and Coastal Management) to address twelve deteriorated proclaimed fishing harbours. These harbours had received no maintenance since the disbanding of the Fisheries Development Corporation in 1985, and many of the structures are showing signs of major reinforcement corrosion and other damage. 'Status Quo' reports were completed on each of the twelve harbours towards the end of 2001, and an initial amount of R83 million was allocated to the projects. It is estimated that it could require up to double that amount to complete all twelve contracts. At the time of the commencement of this research, only four (see Figure 1-1) were identified as being of interest as they were all 'active' sites in need of major repair due to corrosion damage.

Figure 1-1: Locations of RAMP sites

1.3 Objectives of thesis

This thesis is an investigation into the repair and rehabilitation of concrete structures that have deteriorated through the processes of steel corrosion, and still need to fulfil serviceability requirements. The marine environment is one of the harshest environments that reinforced concrete has to endure, and so it is structures in this environment that will be addressed in the framework of this research.

The repair and rehabilitation of these structures will be evaluated in terms of the different strategies and methodologies that are being used by consultants in present day civil engineering practice in South Africa. Current projects are anonymously compared and it is intended that the end product of this work will give guidance for the present day practices of repair and rehabilitation of corrosion damaged structures in the marine environment. Recommendations will also be given as to how to improve practice. The objectives are listed below:

- Survey and present relevant literature regarding the repair of concrete harbour structures
- Investigate four RAMP contracts in terms of
    - Contract documents
    - Site works
- Perform forensic testing where possible at the sites in order to inform study
- Compare results from investigations across the four sites, focusing on repair methodology, strategy and philosophy
- Make conclusions about current practice
- Give recommendations for improving current practice

1.4 Scope and Limitations

This thesis uses four repair contracts to critique current practice, and there will naturally be some generalisation made when applying results from this 'sample' to general practice. Each contract will be unique and have its own particular nuances, but the lessons learnt from a comparative critique will be valuable to future contracts.

While the Department of Public Works went to great effort to provide information for this research, it was not possible to procure information about the exact costs of the repair contracts.

At the time of this thesis (2003/4-2005), particularly at the time of the initial investigations and data capture, the various projects were at different stages of completion. It was not practical to follow all of these projects from start to finish. Ideally all these repair works should be evaluated after five to ten years to determine success, but this was not feasible in this research.

The remote locations of three of the four sites also meant that information gathering was only restricted to a few site visits per site.

1.5 Thesis Overview and Layout

The thesis commences with a review of relevant background to the chloride-induced corrosion of reinforcing steel in the marine environment. Damage assessment tools and techniques are also presented, and the different repair options that are most common in practice are discussed.

Chapter 3 outlines the methods of investigation that were used to obtain data, from a review of the contract documentation to the forensic testing that was performed. Chapter 4 presents a review of the contract documentation. This investigates the general and particular contract specifications, as well as the contract drawings for the four harbours. In chapter 5 the repair works at the four harbours are presented, and in chapter 6 forensic tests from two sites are presented, wherever possible in the form of tables, figures and graphs.

In chapter 7 the information is drawn together and discussed firstly from a forensic testing viewpoint, and then critiqued on the basis of repair methodologies. The four sites are compared, conclusions are drawn and recommendations are made to improve practice.

2 Literature Review

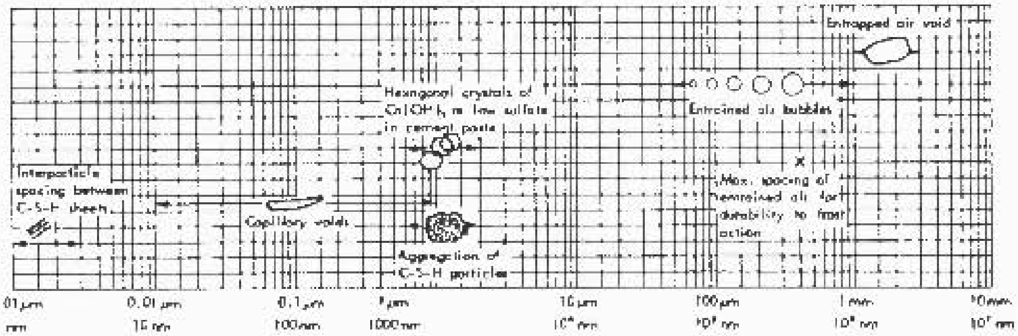
2.1 Introduction

Reinforced concrete structures have finite life spans. Over the course of a structure's life it will be subjected to various deteriorating processes and will, if left alone, eventually fail under serviceability or ultimate requirements. The processes that lead to the deterioration of these structures are complex and in most cases reliant on external influences, including exposure conditions and the presence of oxygen and water. This, coupled with the stochastic behaviour of concrete in the real environment, results in behaviour that is not easy to predict. Many structures require major repair in order to remain serviceable across their entire design life (Strohmeier, 1994). The most common cause of deterioration in these types of structures is a result of the corrosion of the steel contained in reinforced concrete. Not only does the resulting loss of cross-sectional area of the steel present a danger, but the products of the corrosion process are also expansive and in many cases cause the corroding member to crack.

2.2 Concrete in the Marine Environment

Use of concrete in the marine environment is common throughout the world, attributed mainly to the fact that relative to other widely used construction materials such as timber or steel, concrete shows good resistance to water (Mehta: 1991). The properties of concrete, however, are such that the material does not have an indefinite lifespan. An understanding of the microstructure of concrete helps give a clearer view of how concrete deteriorates.

Figure 2-1 is a schematic of the porosity of a typical concrete, and it can be seen that voids in concrete exist on a number of different scales. The penetrability of concrete, i.e. the ease with which agents can move through concrete, comes as a result not only of the size of these pores, but also the connectivity of the pores. This influences the deterioration of reinforced concrete in several ways, and the most prominent is the role that the cover concrete will play in a structure, effectively dictating that rate at which corrosion-inducing chlorides will move into the concrete towards the steel reinforcement.



Dimensional range of solids and pores in a hydrated cement paste.

Figure 2-1: Dimensional Range of solids and pores in a hydrated cement paste (Mehta, 1991)

The more porous the concrete (i.e. the more voids), and higher the connectivity of these pores, the quicker it will allow chloride ion transport. Traditionally engineers have relied on the link between the compressive strength of a concrete and the pore structure to account for durability, noting that typically a stronger concrete will have fewer voids (Richardson, 2002,). The modern engineer, however, needs a greater understanding of the processes that are involved in order to accurately design concrete structures that will be durable.

The environmental conditions that concrete will have to face are as important to address as the properties of the concrete itself. Table 2-1 gives a classification of exposure conditions taken from a research monograph by the University of Cape Town (2001). The categories relate mainly to corrosion of reinforced concrete due to chloride ingress, and thus are all in relation to the presence of salt-laden seawater. The most severe locality is not only that in direct contact with chlorides (seawater), but also in the marine tidal and splash zones, with the continuous wave action compounding the deleterious mechanisms. This is not only due to abrasive action, but also the cyclic drying and wetting of the concrete. A similar categorization of the influence of the environment on chloride penetration is shown for contrast in Table 2-2.

Table 2-1: Categorization of Exposure Conditions: (Mackechnie, 2001)

Marine exposure category	***Marine tidal and splash zones***	***Marine spray zone***
Extreme	*Sea water with heavy wave action and/or abrasion*	*N/A*
Very severe	*Sea water under sheltered conditions with little wave action*	*Within 500 m of shore - heavy wave action and onshore wind*
Severe	*N/A*	*Near shore (>500 m) in an exposed marine location*
Moderate	*N/A*	*Sheltered within 1km of shore or within 30 km of coast*

Table 2-2: Influence of Environment of Chloride penetration – (Schiesl and Bakker in Addis, 2001)

Not aggressive	*Indoor conditions with <70%; constantly and totally immersed*
Moderately aggressive (without chlorides)	*RH always >70%; infrequent major variations in RH; occasional condensation*
Aggressive (without chlorides)	*Frequent major variations in RH; frequent condensation or wetting and drying cycles*
Aggressive (with chlorides)	*Marine environments without direct contact to sea water; low chloride attack in combination with infrequent major variations in RH*
Very aggressive	*Severe chloride attack; sea water splash zone; frequent wetting and drying*
Extremely aggressive	*Very severe chloride attack; frequent chloride splash water to horizontal surfaces*

Assessing the performance of concrete in the marine environment requires the investigation of two areas: the concrete properties and the exposure conditions. However, in practice it is not possible to create perfect concrete properties or a totally non-aggressive environment that will allow for an indefinite reinforced concrete life span. Some of the mechanisms responsible for the deterioration of reinforced concrete are shown in Figure 2-2.

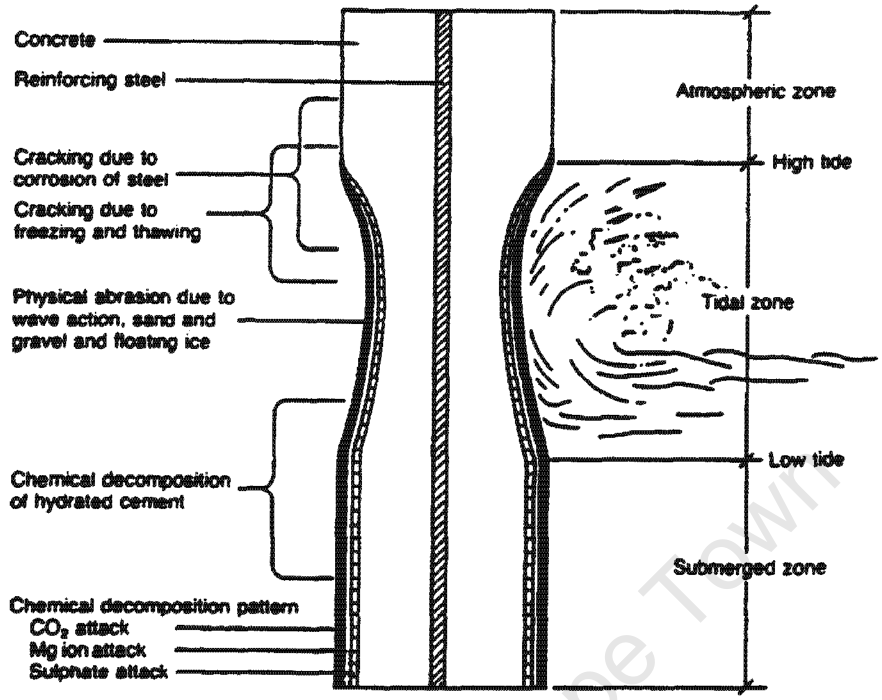


Figure 2-2: Deterioration process in marine concrete (Mehta, 1991)

The diagram above shows a range of deterioration mechanisms that can be broadly classified into two groups, physical and chemical. It has been noted by various authors (Pullar-Strecker, 2002) that the main cause of deterioration in reinforced concrete structures in the marine environment is the corrosion of the reinforcing steel in the concrete, and this process is both chemical and physical. The following section will focus on the chemical fundamentals of concrete corrosion, moving from a theoretical background to the physical damage that corrosion can cause.

2.3 The Corrosion of Reinforced Concrete

The corrosion of steel is an electrochemical process whereby steel is oxidised through various stages to form an expansive hydrated ferric oxide, commonly known as rust. Most steel will corrode in the presence of moisture and oxygen, as it is not stable and actively seeks to convert to an oxidised state. Steel in concrete does not corrode initially because concrete is a highly alkaline material. Concrete contains microscopic pores with high concentrations of alkaline hydroxides in the presence of water (Broomfield, 1997).

This alkaline condition leads to the formation of a passive gamma ferric oxide layer on the surface of the steel, which is a thin film of oxide that stifles any further corrosion by forming a thin coating on the surface of the steel. In this manner, it is the corrosion of the steel that prevents any further corrosion, a favourable anomaly. This passive layer is however not indestructible, and will break down to allow corrosion under two conditions. The first is if the pH reduces below the passive range, which is what occurs during carbonation, and the second is in the presence of sufficient chlorides.

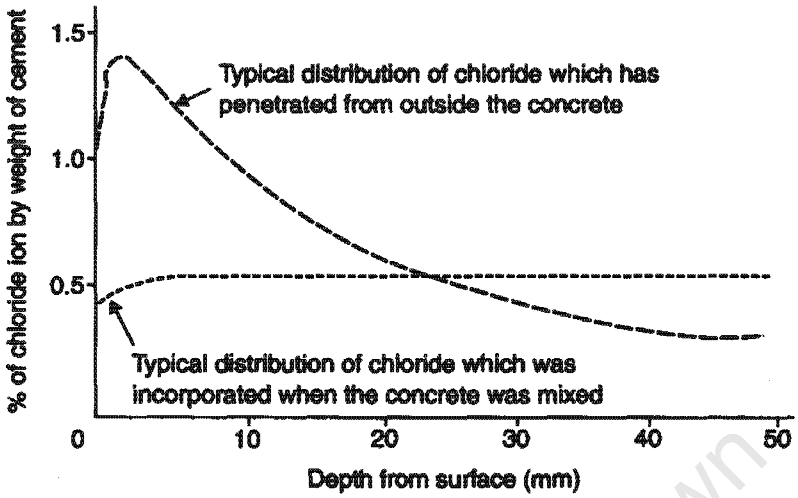
2.4 Chloride Attack

The protective layer of ferric oxide that forms on reinforcing steel in alkaline concrete is broken down by the presence of chloride ions in sufficient concentration. This section will look at the causes and mechanisms of chloride attack.

2.4.1 Causes of chloride contamination

In the marine environment, the main source of chlorides is seawater. Structures do not necessarily have to be in direct contact with the water for chloride contamination to occur, as it is possible for sea spray to carry salts many kilometres beyond the shoreline. The highest chloride concentrations however will be found in concretes that have direct contact with the seawater, but if the concrete is fully submerged or saturated, there is little danger of active corrosion, and so this is of limited interest to the engineer. Therefore the most critical locations in terms of marine structures are either on or just above the fluctuating sea level, as was shown in Figure 2-2.

It is also possible for chlorides to be inherently included in the concrete mix. This is not common, but Pullar-Strecker states that there have been recorded instances of seawater being used in marine construction instead of ordinary tap water (Pullar-Strecker, 2002). This practice results in levels of chloride that are extremely high and promote corrosion from an early age within the structure. Until about the 1960's it was also common practice to use admixtures containing calcium chloride in construction, a practice that has since been discarded. In colder climates, de-icing salts are used to clear roadways and this is also a common cause of chloride contamination (Pullar-Strecker, 2002). Figure 2-3 shows the difference in chloride profiles between concretes that included chlorides in the mix and 'normal' concretes.



Chloride ion profiles in concrete from penetrated and mixed-in chlorides

Figure 2-3: Typical chloride profile of concrete elements in the marine environment (Pullar-Strecker, 2002)

2.4.2 Factors influencing transport mechanisms in concrete

Concrete provides protection against reinforcement corrosion in two ways – firstly, the material properties of the concrete dictate the ability of corrosion causing agents, in this instance chlorides, to permeate or travel into it. Secondly, the cover affects the length of time before corrosion commences. The cover depth will also affect the susceptibility of a corroding structure to crack and show signs of deterioration. Thus not only do these material properties affect the length of time before corrosion is initiated, but also the rate at which the structure will deteriorate. Whilst some would say that the best protection for concrete is more concrete, and adopt larger cover depths for design to prolong service life, others also recognise the need to understand the factors that influence the ingress of chlorides from a material as well as an environmental point of view. Richardson (2002) lists those influencing factors as the following:

- Chloride diffusivity of the concrete
- Sorptivity of the concrete
- Ability of the concrete to bind chlorides
- Water/cement ratio
- Chloride diffusivity of the aggregate

- Degree of exposure to chloride
- Temperature
- Carbonation
- Hydrostatic head (if applicable)

2.4.2.1 Chloride diffusivity

According to Broomfield, the rate of chloride ingress into a concrete can be approximated by the laws of diffusion. The initial transport process appears to be suction, in that the unsaturated concrete readily absorbs seawater. Secondly there is some capillary movement of the seawater, which contains the chlorides, and subsequent to that is what could be termed true diffusion (Broomfield, 1997).

The diffusion of chlorides into concrete produces a gradient that starts at some initial surface concentration and decreases as the depth increases. This surface concentration can be hard to predict, noting that it is possible for the near surface region of a concrete that is in regular contact with water to be diluted in terms of chloride contamination by the constant water action (Broomfield, 1997). Figure 2-3 shows a typical chloride profile, noting the near-surface decrease in concentration.

The chloride profile is most commonly modelled using an error function solution of Fick's 2nd Law, to predict a time to corrosion. The equation is shown below.

$$C_x = C_s \left(1 - \mathrm{erf}\left(\frac{x}{2\sqrt{D_c t}}\right)\right) \qquad (2.1)$$

Where

C_x	= Chloride content by mass of binder at reinforcement
C_s	= Surface concentration of chloride
erf	= Mathematical error function
x	= Depth into the concrete
D_C	= Diffusion coefficient
t	= Time

Equation 2.1 is based upon two parameters that relate to the exposure conditions. The first is the surface chloride concentration, and the second is the diffusion coefficient,

and tables of values for both are given in literature (Mackechnie, 2001), based specifically on the exposure conditions for each structure's location.

Richardson (2002) lists many references that show how ordinary Portland cement concretes performed very poorly in research tests in terms of diffusivity. He goes on to state that the decrease of the chloride ion concentration associated with the use of extenders such as fly ash, slag and even supplementary materials like micro-silica is noteworthy.

2.4.2.2 Concrete Sorptivity

Whilst ion diffusion is the predominant mode of chloride transport, the initial mode is through capillary suction of water into pores near the surface of the concrete. The dry pores absorb water, and have a major influence on the ability of chlorides to move through concrete. This is especially true in climates that have dry and wet cycles, whereby the salt laden moisture will initially be absorbed into the dry pore structure, and the following dry cycles will evaporate the water to leave the salt, which will then begin to diffuse into the concrete as a result of the concentration gradient.

Richardson states that a reduction in the surface absorption can significantly enhance service life by lowering the uptake of chlorides (2002). Alexander et al (1999) echo this sentiment with the development of a water sorptivity durability index test that characterises the near surface absorption of concretes. Such a test could easily be implemented for quality control on site, directly addressing a durability requirement at an early stage. The following diagram is taken from a UCT monograph and shows the effect of different types of curing on the sorptivity characteristics of an OPC concrete.

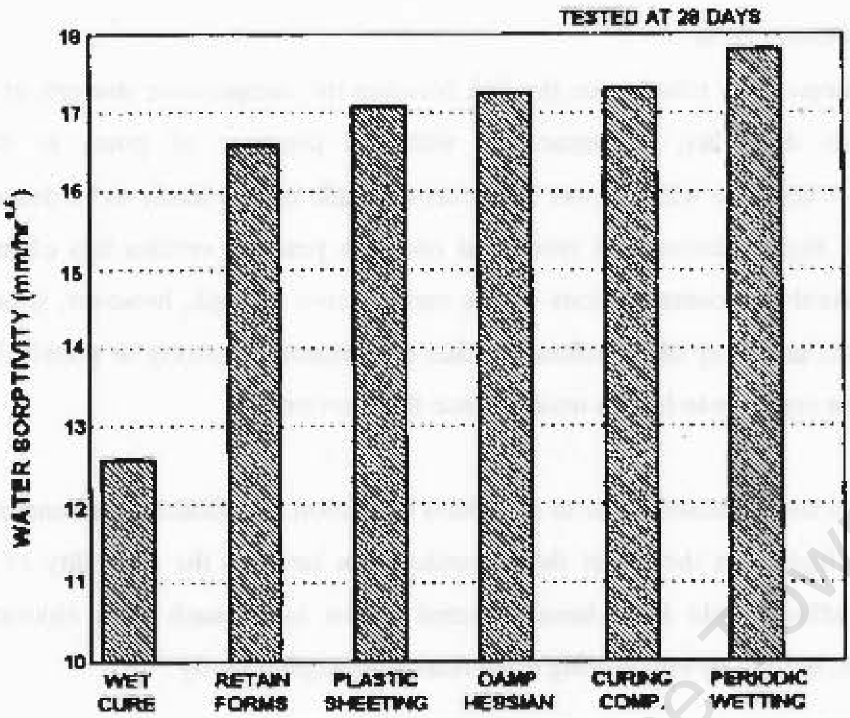


Figure 2-4: The effect of curing on Water Sorptivity (Alexander, 1999)

2.4.2.3 Chloride binding

Chlorides may be present in three states in concrete. As chlorides are transported into the concrete a certain proportion will become bound, and lose mobility. They may be strongly bound, i.e. chemically combined to form calcium chloroaluminates ($3CaO.Al_2O_3.CaCl_2.10H_2O$) and calcium chloroferrites ($3CaO.Fe_2O_3.CaCl_2.10H_2O$), or loosely bound, being adsorbed by calcium silicate hydrate ($3CaO.2SiO_2.3H_2O$) and calcium aluminate hydrate on the pore walls. The remainder is termed free chlorides and it is these free chlorides that are of concern to engineers dealing with chloride-induced corrosion, as it is these chlorides that are responsible for the breakdown of the passive protective layer.

It has been shown that certain cement extenders such as fly ash and slag are capable of binding chlorides more effectively than normal cements, reducing the amount of chlorides moving through the concrete. Adsorbed chlorides are also capable of being 'released', especially due to carbonation. Thus the combined processes of carbonation and chloride ingress are particularly deleterious.

2.4.2.4 *Water/cement ratio*

In the past engineers have relied upon the link between the compressive strength of a concrete and its durability, as associated with the presence of pores in the microstructure. A concrete with a lower water/cement ratio is very likely to be denser than one with a high water/cement ratio, and common practice verifies this claim. Simply basing durability considerations on the compressive strength, however, is not adequate, as there are many other influences that can impact negatively or positively on the ability of a concrete to have a maintenance free service life.

Figure 2-5 shows the increasing time to corrosion activation as a function of concrete grade, and also highlights the effect that extenders can have on the durability of a concrete. This effect could have been detected by an index such as a chloride conductivity test, but not by considering compressive strength directly.

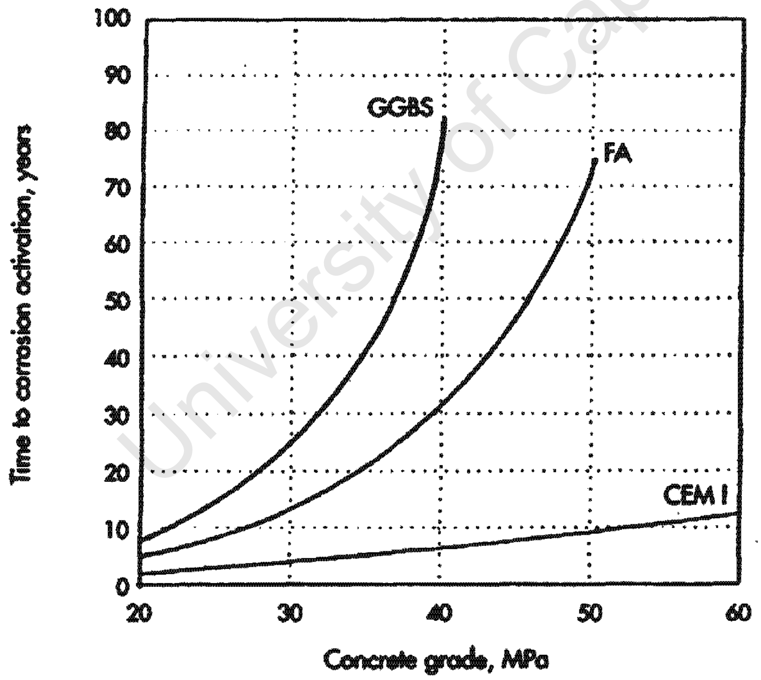


Figure 2-5: Model prediction results of the time to corrosion activation for different concrete types (Addis, 2001)

(CEM 1: Ordinary Portland Cement, FA: Fly Ash, GGBS: Ground Granulated Blastfurnace Slag)

2.4.2.5 Chloride diffusivity of the aggregate

Richardson points out that the diffusivity characteristics of the aggregates can play a role in either improving or negatively impacting the diffusion of chlorides into the concrete as a whole. This would depend on the diffusivity and permeability of the aggregate relative to the cement paste fraction.

2.4.2.6 Degree of exposure to chloride

It is not surprising that the location and orientation of a chloride-affected concrete structure relative to its source of chlorides will to a large extent dictate the movement of chlorides into the concrete. In marine concretes with direct contact with seawater it is the splash zone that typically suffers the most in terms of damage. Also with bridges that encounter de-icing salts, locations such as joints and drains are commonly the first to show major signs of damage.

Thus by addressing the most critical locations in terms of chloride exposure, it is possible to meet the most pressing durability needs. Alternatively, these locations could be regarded as indicators if they are identified as the most likely locations for corrosion damage, and time and expense in terms of the inspection of the entire structure could potentially be saved.

2.4.2.7 Temperature

Dhir (quoted in Richardson) states that the influence of temperature on chloride ingress is also substantial, as a result of two factors. The first is the impact that temperature can have on the diffusion coefficient, possibly doubling the coefficient with a temperature change of 10°C. Secondly, Richardson also states that it is possible for seasonal temperature variations to release bound chlorides (2002).

2.4.2.8 Carbonation

The process of carbonation releases loosely bound chlorides, thereby increasing the free chloride concentration and putting the reinforcing at a higher risk of corrosion. Thus even if a concrete is deteriorating badly due to chloride-induced corrosion, it might be necessary to test for carbonation as well.

2.4.2.9 Hydrostatic head (if applicable)

This is most commonly found in structures that are permanently submerged, and refers to the possibility of a hydrostatic pressure being able to drive chloride-containing water into the concrete.

2.4.3 Chloride threshold

Depassivation occurs when the amount of chlorides at the steel reaches a 'threshold level', and active corrosion is initiated. This threshold value (usually expressed as a percentage of chlorides by mass of cement or binder) is often the benchmark in terms of assessing the service life of a structure. This was seen in a previous section where the time to corrosion initiation was modelled using Fick's second law of diffusion. The reason for this is that the subsequent time from corrosion initiation to serviceability failure may be relatively short in comparison to the time from construction to corrosion initiation. This is especially true for prestressed structures where the consequences of a high corrosion rate are serious.

In Ordinary Portland Cement concretes, a threshold value of 0.4% chloride (by mass of cement) is commonly regarded as being a suitable design value. It should, however, be noted that this value does vary, especially with the use of extenders in newer concretes. Thomas noted that whilst the use of Fly Ash as a cement extender reduced the 'threshold' considerably, the addition of the extender provided better protection because of its affect on the rate of chloride penetration. This is an important consideration, noting that whilst most corrosion damage prediction models focus on 'time to corrosion initiation', the control of the rate of corrosion after the threshold has been reached is also an effective way of managing corrosion in a structure. This is the reason that corrosion of reinforcing steel in permanently saturated concretes such as underwater piles and columns is not as critical as unsaturated locations, in that there is less oxygen available to 'drive' the process (Thomas, 1999).

Broomfield states that a chloride threshold concentration of 0.4% chlorides by mass of cement is the level at which the passive layer on reinforcement is broken down and corrosion may initiate (1997). Mackechnie stated that corrosion threshold depends on various factors that include concrete quality, cover depth and the saturation level of the concrete (2001). Richardson (2002) lists the factors that influence the critical chloride threshold as including the following:

- binder chemistry – especially C_3A content
- ratio of free chloride to total chlorides
- chloride ion to hydroxyl ion ratio
- the water/binder ratio
- the hydroxyl ion concentration
- temperature and relative humidity
- electric potential of the reinforcement.

The large number of influencing factors result in many different reported thresholds in the literature, but the most commonly adopted value is 0.4 % chloride by mass of cement or binder.

The following tables show various estimates of corrosion probability:

Table 2-3: Corrosion Probability after Browne (1980)

Chloride content by mass of cement (%)	Probability of corrosion
<0.4	*Negligible*
0.4-1.0	*Possible*
1.0-2.0	*Probable*
>2.0	*Certain*

Table 2-4: Corrosion Probability (Mackechnie and Alexander, 2001)

Chloride content by mass of cement (%)	Probability of corrosion
<0.4	*Low*
0.4-1.0	*Moderate*
>1.0	*High*

2.4.4 The nature of steel corrosion in concrete

According to Mackechnie and Alexander (2001), once the passive layer has been broken down, the following requirements are necessary for corrosion to occur at a significant rate:

- A reactive metal that will oxidise anodically to form soluble ions
- A reducible material that provides the cathodic reactant (typically hydroxyl ions)
- An electrolyte that allows ionic movement between the material and environment

In concrete in the marine environment, all of the above are provided. The reinforcing steel itself is both the anode and the cathode, whilst the concrete (especially if a high moisture content is present) is the electrolyte.

Figure 2-6 is a schematic representation of the corrosion process:

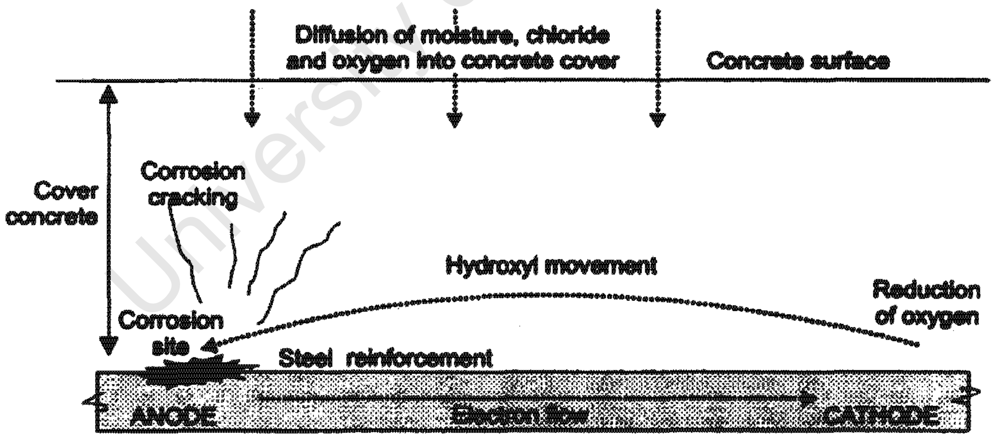


Figure 2-6: Schematic representation of corrosion in reinforced concrete (Mackechnie and Alexander, 2001)

Four states of corrosion are mentioned, each one being dependent on the surrounding environmental exposure conditions:

- Passive State: This is the state of steel in sound concrete before deterioration occurs. Only small levels of corrosion occur, and this is to maintain the protective passive layer of the steel.
- Pitting corrosion: In localised anodes and cathodes, caused by the breakdown of the protective ferric oxide layer. This is usually activated in marine concretes by the presence of chlorides at the steel.
- General corrosion: This is due to a breakdown of the passive film over a larger area. This will cause multiple corrosion sites on the steel, and is common in carbonated concrete.
- Active, low potential corrosion: This is common in concrete in the marine environment that is permanently underwater. There is not enough oxygen present to restore the passive film on the surface of the steel.

Only 'pitting' corrosion and 'general' corrosion are of major concern to the practising engineer, because of their ability to corrode at a significantly faster rate than passive and low potential corrosion. At this stage the distinction can also be made between what is termed micro-cell corrosion and macro-cell corrosion. This refers to the relative distance between anodic and cathodic sites. In micro-cell corrosion, the anode and cathode are closely spaced, and this is favoured in concretes that have a high resistivity and deep cover. Macro-cell corrosion occurs more frequently in concretes that are contaminated with salts.

The corrosion reactions that occur are summarised as follows (Broomfield 1997)

$$Fe \rightarrow Fe^{2+} + 2e^{-} \qquad (2.2)$$

This is the anodic reaction, where the steel dissolves into the pore water of the concrete, and gives up two electrons. These two electrons are taken up at the cathode:

$$2e^{-} + H_2O + \tfrac{1}{2}O_2 \rightarrow 2OH^{-} \qquad (2.3)$$

This is the cathodic reaction and is depicted in the diagram, and the two hydroxyl ions that are released increase the local alkalinity at the steel, thereby strengthening the

2.5 Corrosion Damage in Concrete

2.5.1 Expansive corrosion products

Corrosion products are expansive and increase in volume as they are oxidised. This results in the formation of large internal pressures that are exerted on the cover concrete, and this could lead to cracks being formed and even the delamination of the cover concrete. The relative volume of the corrosion products are shown in Figure 2-7 (Mehta 1991):

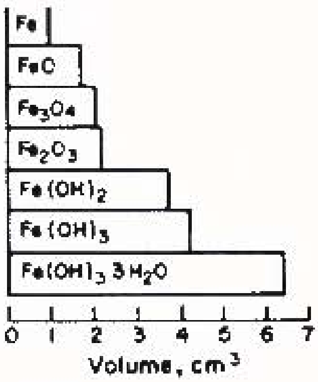


Figure 2-7: Relative size of corrosion products

The following diagrams show the manner in which the expansive products of the corrosion process are capable of causing concrete damage. The first shows longitudinal cracking along the edge beam of a breakwater, while the second shows a spalled patch of deck soffit, where the concrete has broken off from the structure to expose the steel reinforcement.

Figure 2-8: Concrete deteriorated due to cracking and spalling (Source: Author)

If a structure cracks and spalls, it may not represent a direct danger in terms of ultimate failure at that particular location. From a serviceability point of view, however, it might need repair, because aesthetically it will affect the use of the structure.

There exists an anomaly within spalling in the fact that 'weaker' concretes, or rather concretes in which there are more pores, will be less susceptible to the pressures that are built up by expansive corrosion products. In more dense concretes, there will be fewer voids into which the corrosion products can move, and thus stresses will be built up at a higher rate.

2.5.2 Loss of cross-sectional area (Pitting corrosion)

This is a directly mechanical application, where the same force exerted on a smaller area will have a higher stress. Thus, as the cross-sectional area of the steel is reduced through corrosion, there is a chance that the steel member will no longer be able to carry its design load. This is further intensified by the pernicious nature of steel corrosion, often attacking highly localised areas while leaving surrounding areas undamaged. Figure 2-9 below shows a heavily corroded steel reinforcing bar, and its loss of cross-sectional area can clearly be seen:

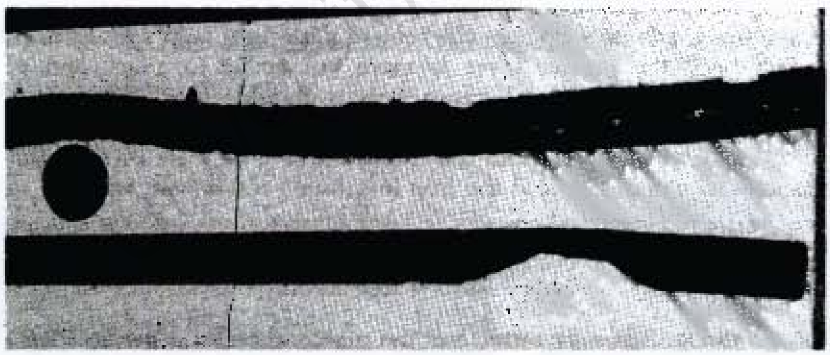


Figure 2-9: Loss of cross-sectional area due to corrosion (Broomfield, 1997)

2.5.3 Corrosion products as an indicator of damage

Brown rust staining and minor cracking are useful to the practising engineer as they give warning signs as to the performance of the structure. Many structural assessment methods are based solely on identifying these effects, and using them to make a prediction of the expected future performance. Mackechnie and Alexander (2001), specification must have an understanding of what the deterioration processes are, and how the particular repair method that is being adopted will counter this deterioration.

Mehta (1991) describes the 'normal' steps that are taken in repair as being:

- Evaluation of the condition of the concrete
- Determination of the causes of damage or deterioration
- Selection of the repair materials
- Selection of repair methods, including the methods of surface preparation

It is often the case in reinforced concrete damage that the condition of the concrete will give a good insight into what deteriorating processes are involved. The previous section on the damage caused by steel corrosion discussed how it was possible for there to be clear indicators of damage in a structure that is undergoing active corrosion, typically in the form of cracks, spalls or rust staining.

2.6.2 Damage and corrosion indicators

While the ability of a corroding structure to give 'warnings' as to its performance is desirable, and often used as the basis for assessment, an experienced engineer will be required to assess the performance of a structure even when these indicators are not present. Basing assessment on the presence of cracks, spalls and corrosion staining might suit the requirements of the most cursory of investigations; a fuller insight can only come from the use of more detailed test methods. Invariably this will be at a greater cost, but as is stated by Mays (1992), the fraction of a typical repair budget spent on initial investigations and specification preparation is minimal. A brief discussion of the most commonly used forensic investigation methods follows.

2.6.2.1 Visual Surveys

The initial investigation will almost always be a general visual inspection, paying attention to the following (Addis, 2001):

- Nature and pattern of cracking
- Distortion or displacement of structural elements
- Evidence of reinforcement corrosion

- Evidence of poor quality concrete
- Systematic recording of observations

Pullar-Strecker (2002) echoes the importance of the last point, stating that especially in cases involving litigation; a good series of photographs of damage will go a long way to back up the engineers' assessment of the level of damage. In marine conditions, where access may be problematic, Mehta (1991) states that the use of a diver or underwater camera may suffice.

2.6.2.2 Carbonation

The most common test for carbonation uses a phenolphthalein spray gun, applied to a core or piece of concrete. This phenolphthalein solution will remain colourless in carbonated concrete. In uncarbonated concrete, where concrete is still alkaline (pH > 9), the solution will turn pink or purple. The importance of this test can be twofold, in that while it checks for carbonation as a corrosion inducer, carbonation can accelerate the rate of chloride ingress.

2.6.2.3 Chloride Ingress

Whilst it is possible to perform simple and approximate chloride ingress tests on site, it is the common practice of the majority of engineers who require this information to dispatch extracted cores to a forensic laboratory for chemical titration testing. This results in a chloride profile for the core, plotting the chlorides (expressed as a percentage by mass of cement/binder) versus depth. The critical chloride percentage is typically adopted as 0.4%, and some judgment would be made as to whether or not active corrosion is taking place at the level of the steel.

Mackechnie and Alexander (2001) give some limitations to be aware of when adopting this test approach:

- Presence of chlorides in aggregates may give misleading results
- Chloride contents in cracks and defects cannot be accurately determined
- Slag concretes may be difficult to analyse with colorimetric titration methods
- Relatively large samples are required to allow for the presence of aggregates

2.6.2.4 *Half-Cell Potential*

This is a measure of the electrode potential between the surface of a concrete and the reinforcing steel. The test procedure uses an electrode in a solution of a salt of its own ions (for example copper: copper sulphate, silver: silver nitrate), which is applied to the surface of the concrete. A connection is made to the reinforcing and the electrical potential between the two is measured, and expressed in millivolts. ASTM (American Society for Testing and Materials) expresses the categories for these results as follows:

Table 2-5: ASTM Rebar Potentials as referenced in Mackechnie and Alexander (2001)

Rebar potential (-mV Cu/$CuSO_4$)	Qualitative Risk of Corrosion
<200	*Low*
200-350	*Uncertain*
>350	*High*

The following are the limitations of such tests:

- Interpretation of results must be done with caution (preferably by a specialist)
- Rebar potentials from carbonated concrete are difficult to interpret (the reading is a mixed potential of anodic and cathodic sites)
- Delamination may disrupt the potential field producing false readings
- Environmental effects will influence potentials (e.g. temperature and humidity)
- Rebar potentials cannot be directly correlated with corrosion rates
- Stray currents may affect measured potentials

2.6.2.5 *Resistivity*

A test of concrete resistivity measures the ability of a concrete to conduct current flow between the anode and the cathode, recognising that this will to a large extent control the rate of corrosion of steel in concrete. This is dependent on the pore structure of the concrete and also the degree of saturation, implying that poor quality saturated concrete will have a lower resistivity and a higher conductivity (and therefore be more likely to enhance corrosion). Alternatively, good quality concrete with a low degree of saturation will have a lower conductivity, a higher resistivity and will thus be more capable of stifling the corrosion process (Mackechnie and Alexander, 2001).

The most common test method for resistivity uses the Wenner Probe, which has four probes in contact with the surface of the concrete. The two outer probes transmit an alternating current and the two inner probes measure the potential difference. The resistivity is then calculated relating the current flow, potential drop and spacing of the probes (Mays. 1992).

Mackechnie and Alexander give a rough assessment of the likely corrosion rate:

Table 2-6: Concrete Resistivity Classes

Concrete Resistivity (kOhmcm)	Likely corrosion rate given corrosive conditions
<12	*High*
12-20	*Moderate*
>20	*Low*

2.6.2.6 Corrosion Rate

Using linear polarization principles, it is possible to measure the actual corrosion rate in reinforced concrete. Mackechnie and Alexander regard this method as being the only reliable method of assessing the corrosion rate of a damaged structure. However, they also state that single readings are generally unreliable as a result of the effect of environmental and material influences. The relatively large expense that is associated with the equipment required for accurate corrosion rate testing is also a limiting factor. The following table shows a qualitative assessment of corrosion rates:

Table 2-7: Qualitative assessment of corrosion rate

Corrosion Rate ($\mu A/cm^2$)	Qualitative assessment of corrosion rate
>10	*High*
1.0-10	*Moderate*
0.2-1.0	*Low*
<0.2	*Passive*

2.6.2.7 Cover Survey

According to Pullar-Strecker (2001), assessing the depth of cover is important as it can give some indication of the likelihood of future cracking and spalling. In structures that have already spalled badly and are showing exposed steel, this evidence is readily available. In locations where the concrete is still 'sound' an electromagnetic cover meter can be used to determine the depth of the steel cover to an accuracy of ±5 mm.

Caution should be taken with this method when assessing large cover depths, and where steel is bunched, crossed or closely spaced as this could confuse the instrument and result in inaccurate readings. Calibration of the cover meter is also advised (Addis, 2001).

2.6.3 Ranking systems

It is possible to draw together the previously mentioned indicators into a comparison and to weight them according to the level of damage, in order to compare structures to determine the relative level of damage.

Andrade proposes a simplified method of evaluating damage in corrosion affected structures, relating these tests to each other and to other parameters such as the environmental exposure and the type of structure. This results in an index that will give a guideline as to what the urgency of intervention is, and how soon the deteriorating member will need to be addressed. (Andrade, 2004):

2.6.3.1 Environmental Aggressivity (EA)

The particular location in which the structure is found is classified according to the EN 206 code. The more deleterious the environment, the higher the assigned index will be, ranging from 0 (XO) to 4 (XS3). For the model proposed by Andrade, the focus was on corrosion damage and thus certain of the above exposure classes, such as freeze/thaw attack, were disregarded within the index scale.

Table 2-8: Summary of exposure classes and environments in EN 206-1

Degradation phenomenon	Sub-class	Environment
No risk of corrosion attack	X0	*Unreinforced concrete: all exposures except freeze/thaw, abrasion, chemical attack Reinforced concrete: very dry*
Corrosion induced by carbonation	XC1	*Dry or permanently wet*
	XC2	*Wet, rarely dry*
	XC3	*Moderate humidity*
	XC4	*Cyclical wet and dry*
Corrosion induced by chlorides other than from seawater	XD1	*Moderate humidity*
	XD2	*Wet, rarely dry*
	XD3	*Cyclical wet and dry*
Corrosion induced by chlorides from seawater	XS1	*Exposure to airborne salt*
	XS2	*Permanently submerge*
	XS3	*Tidal, splash and spray zones*
Freeze/thaw attack	XF1	*Moderate water saturation, no de-icing agent*
	XF2	*Moderate water saturation, de-icing agent*
	XF3	*High water saturation, no de-icing agent*
	XF4	*High water saturation, de-icing agent or sea water*
Chemical attack	XA1	*Slightly* aggressive environment*
	XA2	*Moderately* aggressive environment*
	XA3	*Highly* aggressive environment*

* Quantified in respect of the chemical characteristics of groundwater (SO_4^{2-}, pH, CO_2, NH_4^+, Mg^{2+}) or soil (SO_4^{2-}, acidity)

2.6.3.2 *Damage Corrosion Index*

The index that will be combined with the Environmental Aggressivity index (EA) is the Damage Corrosion Index (DCI), which will seek to place the level of actual corrosion damage on an index scale. This uses the common forensic tests that have been discussed such as corrosion rate measurement, depth of chloride penetration, resistivity, and combines the results from each. Each individual test that is performed on the corroding location will give a result that is then placed in an index, ranging from Level I (no corrosion damage) to Level IV (major corrosion present). A combined DCI is then obtained from the following equation:

$$DCI = \frac{\sum_{i=1}^{n} Corrosion \quad Indicator \quad level_i}{n} \tag{2.7}$$

It is possible to weight certain results more heavily than others, if it is felt the reliability or accuracy of some tests are higher than others.

2.6.4 Simplified corrosion index

The simplified corrosion index (SCI) is the combination of the Environmental Aggressivity index (EA) and the Damage Corrosion index (DCI). The two numbers are now on a comparable scale (1-4), and a simple average will suffice. Andrade states that the number produced should fit into the following categorization:

Table 2-9: Simplified Corrosion Index (Andrade, 2004)

Level of Corrosion	Simplified Corrosion Index
Very Low Corrosion	*0-1*
Low Corrosion	*1-2*
Moderate Corrosion	*2-3*
High Corrosion	*3-4*

2.6.4.1 Type of structural element

Provision can be made in this type of indexing system for the type of structural element that is being investigated. This refers to the idea that some members will be more critical than others in terms of the likelihood of ultimate structural failure. Andrade made provision for this by the use of tables that would assign an index based on the amount of shear reinforcement present (assuming shear failure as the most likely failure mechanism), taken as an indicator of the likelihood of sudden failure.

2.6.4.2 Safety Margin and Consequences of Failure

Where such information is available and accurate, the amount to which the section capacity is over-designed can also be factored into the index considerations. The consequence of the structure's failure is also categorised, noting the connection between the two. The higher the consequence of failure and the lower the amount to which that particular structure is over-designed, the more critical the index will be from a 'necessity of rehabilitation' point of view.

2.6.4.3 *Urgency of Intervention*

The final index into which the structure is placed is a categorization that gives guidance as to the 'urgency of intervention', i.e. it gives some indication as to the appropriate steps that are to be taken and in what timeframe. For this to be done, all the index values that have obtained until now, such as the Structural damage index, the Corrosion index and the consequences of failure are drawn together as a final Structural Damage Index (SDI) value. The classification follows from this value, and an example taken from Andrade's work is shown below:

Table 2-10: Structural Damage Index after Andrade (2004)

SDI Value	Urgency of Intervention	Action Needed
Negligible	>10	*Periodic inspections*
Medium	5-10	*Reassess structure within this time*
Severe	2-5	*Structural assessment within this time*
Very Severe	0-2	*Repair or detailed structural assessment need within this time*

2.6.5 Discussion

A summary of the most common methods in current use in local engineering practice has been presented, and an attempt has been made to show how they are capable of guiding the decisions that are made regarding repair and rehabilitation.

Focus was given to work proposed by Andrade, where a simplified method of corrosion assessment was used to evaluate the need for repair, and some discussion is necessary about the appropriateness of such a system.

The use of any system of assessment will be doomed to failure if there is no understanding of the basic mechanisms of deterioration, from understanding what the governing inputs are to how they are recognised and as to how 'best' to treat concrete that has been damaged by such mechanisms. Within a system such as this simplified approach, more than a single method of assessment is used and the results are combined for an index of the corrosion level. It is questionable whether it is best practice to combine different sets of results with the same weighting to obtain an average. The reason for this is twofold. Firstly, the accuracy of the tests might be such that one may predict or report the level of corrosion damage with a much higher accuracy and confidence than other tests. It then follows that this set of results would be given higher weighting in the combination of results, thus having more influence on the average value. Secondly, if the engineer responsible for compiling data from a number of tests is not confident of the accuracy of the results or the methods that are being used to obtain them, he/she should be allowed the opportunity to either reduce the impact of that set of results on the final value or to discard them completely.

The other concern that arises out of the proposed methods for simplified indexing is that the ultimate goal is to place the particular location into a scale that will represent the action to be taken. The idea in this approach is that it is sufficient to allow a structure to deteriorate until a specific level of damage is reached such that its combined indicators (such as corrosion damage, consequence and failure and relevance of damage) place it in a category that recommends that remediation take place as soon as possible, or at least within the next few years. From the viewpoint of simplifying the assessment process and unifying different investigation and recommendation techniques, this is a step forward. From the viewpoint of what the most effective method of repair is in terms of saving money and prolonging service life, it is limited in the simple fact that certain repair measures are most effective when applied at levels of deterioration that might not coincide with the level of deterioration that the structure would be allowed to reach under the aforementioned approach. Some treatment and repair methodologies, when applied at an earlier stage when damage is not at a critical level, will perform better and give the structure more protection against future damage, as opposed to allowing the structure, untreated, to reach a damage level that requires more costly and possibly less successful methods to be instituted.

The strength of this approach lies in its ability to identify structures that are in the most need of repair, and this is all too often the scenario that engineers are faced with. Thus, when deciding which structures should be repaired first amongst an array of deteriorating structures, and more importantly how the oftentimes small allocated repair budget should be spent, a simplified approach to assessment finds itself being well suited to the task.

2.7 Repair Options

2.7.1 Introduction

There are different strategies that can be selected for the rehabilitation of concrete that has undergone reinforcement corrosion. These range from surface applications of protective treatments to expensive electrochemical techniques and even the demolition and reconstruction of the structure itself. Typically the cost of the repair is proportional to the level of deterioration of the structure, but when a decision must be made between different options, each will have a different impact in terms of cost and period of protection. Also, the selection of some repair options are dependent on the deterioration that has already occurred, and so might not be a viable option for use if the structure is in an advanced state of deterioration. Methods of repair will be discussed in this section only as they pertain to the treatment and repair of chloride-damaged structures.

2.7.2 Do nothing (Leave alone)

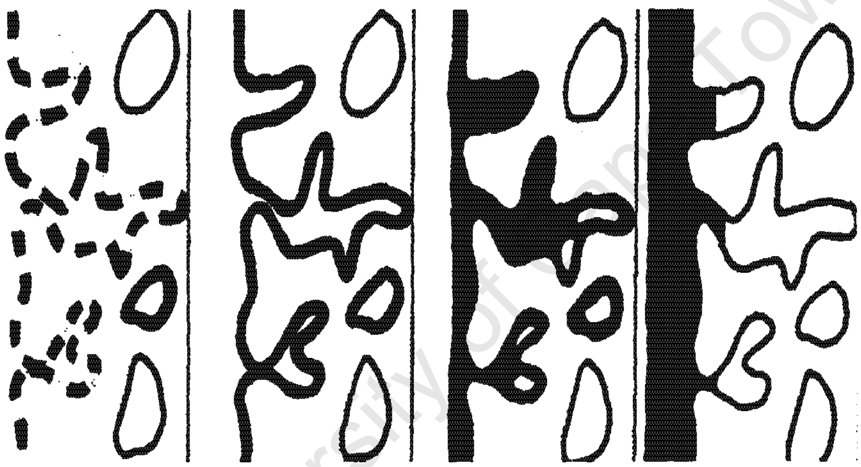
An option open to the engineer is not to perform any repair on the structure. There may be varying reasons for this, for example if the structure is nearing the end of its design life, of if it is not being used as intended and there is no need for it to be fully functional. Alternatively, there may be insufficient funding and thus the decision is guided by monetary considerations. Wherever this approach is adopted, care should be taken as to the safety of the people who will interact with the structure, especially as it moves toward eventual failure. Thus on certain occasions, it is best to use this strategy in combination with demolition (Scott, 1997)

2.7.3 Physical and chemical repair techniques

2.7.3.1 Coating (Barrier) systems

Barrier coatings and surface sealants are preventative methods that are used in structures where active corrosion is not occurring. The purpose of a barrier system for concrete repair and rehabilitation is to prevent the movement of these chlorides into the concrete. Broomfield notes that once chloride corrosion has initiated, a barrier sealant will do little to prevent further corrosion (1997). He stated that, typically, once the steel had started to corrode, there is already enough moisture and oxygen in the concrete for the steel to further corrode and crack the concrete. Once the concrete is cracked, moisture and oxygen will have free access to the steel. He further noted that because of the fact that a barrier coating would not restore the passivity to the steel, their use after corrosion has been initiated provides little protection.

Modern coating systems use silanes and siloxylsilanes to penetrate into the pores of the concrete and form a hydrophobic layer. This layer allows the movement of water vapour through the membrane, allowing the concrete to breathe while still retaining impermeability to water. The different types of surface protection are shown in Figure 2-10:



(a) (b) (c) (d)

Figure 2-10: Surface protection, ranging from intermittent layer (far left) to full barrier (far right) (Mehta, 1991)

Mackechnie and Alexander (2001) suggest that the use of hydrophobic sealants is better than the use of full barrier coatings in that they reduce the moisture content of the concrete, lowering the potential for corrosion to occur. They also state that hydrophobic sealants are best suited for use in uncontaminated concrete that is free from cracks and surface defects. Unfortunately the recent advent of these types of treatments means that there is in fact little real data from in-situ experiments to substantiate the claims of the manufacturers. Another major deterrent with this treatment is the dependence on correct procedures to be followed on site, especially with regard to the methodology of surface treatments and coating applications.

2.7.3.2 *Patch repairs*

The manner in which chlorides are able to ingress into the concrete and break down the passive layer of the reinforcing steel is such that instances of reinforcement corrosion can be isolated, with the formation of distinct anodic and cathodic sites. In this case, it may be suitable to remove the chloride-contaminated concrete from that particular location and replace it with a patch repair.

Patch repairs and chloride-induced corrosion

Chloride induced corrosion is much more aggressive in nature and although the locations of corrosion are well isolated, there is usually a high rate of corrosion present. The main danger with this type of corrosion is that it is possible for large areas of reinforced concrete to be corroding without visibly showing signs of corrosion.

If the concrete surrounding a patch repair still contains chlorides it could result in the formation of incipient anodes on either side of the patch. According to Mackechnie and Alexander (2001), not only does this initiate corrosion adjacent to the patch, it could also result in patch failure within as little as two years.

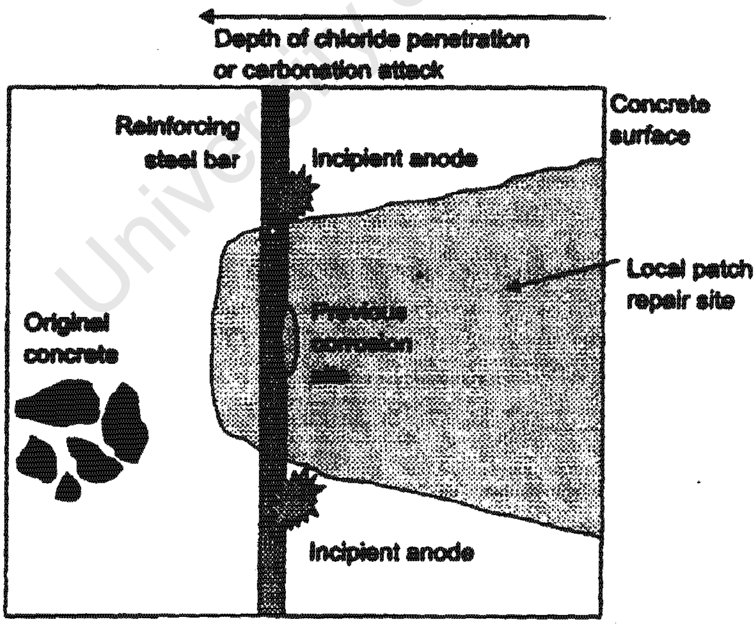


Figure 2-11: A typical patch repair (Mackechnie and Alexander, 2001)

Therefore it is vital to remove all the concrete containing chlorides in a patch repair.

Concrete removal

The first step is to expose all of the corroded reinforcement by removing the surrounding concrete. This is often termed 'breaking out'.

Steel preparation

After the concrete has been removed, the corroded reinforcement bars must be cleaned. This stage is important, as corroding reinforcement will continue corroding unless it is properly cleaned. Once the steel has been cleaned, it is common practice to apply an anti-corrosion coating to the surface of the steel.

Patch application

A bonding aid can be applied to assist the bond between the patch and the original concrete, but most patch material manufacturers claim that this is not necessary.

In terms of materials, the repair mortar that is used as a replacement material to the original concrete is typically a cementitious grout/mortar containing small aggregates or even small fiber-reinforcement to improve micro-cracking properties.

Curing and sealing

The correct curing of repair concrete is very important, and curing can be performed by the use of sealing membranes or polyethylene sheets that are attached to the patch surface to prevent moisture loss.

In site conditions, the main problems that are encountered with the successful applications of patch repair involve the correct preparation of the substrate surface and the mix of the repair mortar. The surface of the steel must be fully cleared of any corrosive products and the concrete surfaces must be free from dust deposits and loose aggregates.

2.7.3.3 Corrosion Inhibitors

Corrosion inhibitors are different to the previously mentioned methods of treatment in that they do not focus on the agents that initiate corrosion, i.e chlorides, but rather with the rate of the corrosion reactions. According to Mackechnie and Alexander (2001, pg. 20), a corrosion inhibitor is a chemical substance that reduces the corrosion of metals without a reduction in the concentration of corrosive agents and this is achieved by the reduction of the rate of the anodic and cathodic reactions at the steel.

The principle in corrosion inhibitors is to develop a thin layer not more than a few molecules thick that will inhibit corrosion at the surface of the steel.

The performance of the inhibitors is dependant on many factors, and the predicted inhibition as a result of these influencing factors is shown below (Mackechnie and Alexander 2001):

Table 2-11: Likely performance of corrosion inhibitors

Likely Inhibition	Corrosive conditions	Concrete conditions	Severity of corrosion
Good	*Mildly corrosive, low chlorides or carbonation*	*Dense concrete with good cover depths (> 50 mm)*	*Limited corrosion with minor pitting of steel*
Moderate	*Moderate levels of chloride at rebar (i.e. <1 %)*	*Moderate quality concrete, some cracking*	*Moderate corrosion with some pitting*
Poor	*High chloride levels at rebar (i.e. >1%)*	*Cracked, damaged concrete, low cover to rebar*	*Entrenched corrosion with deep pitting*

Thus from the above the 'best' concrete to apply inhibitors to would be the concrete that has deteriorated the least, having mild corrosive conditions and showing little damage. The suitability of this method as an early preventative measure is noted.

A corrosion inhibitor that is applied after construction must be capable of 'migrating' into the concrete to the level of the steel. Migrating inhibitors move through unsaturated concrete by vapour diffusion, according to Mackechnie and Alexander (2001). Thus the use of a corrosion inhibitor can have limited success in structures that are exposed to the marine environment, because of the fact that many of the members will contain concrete that is near saturation. Mackechnie and Alexander (2001), state that this is a result of the high moisture and salt levels that prevent significant vapour diffusion.

Therefore, two key factors influencing the success of corrosion inhibitors must be assessed prior to application. Firstly, the ability of the inhibitor to be able to penetrate into the concrete to the level of the steel (which can be assessed through test patches), and secondly, the severity of the corrosive environment (which can be assessed by chloride profiling). This is important because if the corrosion at the steel is too advanced, successful inhibition is not possible. In work done by Rylands (1999), it was suggested that it would not be possible for a corrosion inhibitor to work successfully if the chloride content in that particular concrete was more than about 1.0 % by mass of cement.

Alternatives to the surface application of penetrating corrosion inhibitors are to apply them directly to the steel, which would require concrete to be removed, or to incorporate the admixture in concrete overlays (Broomfield 1997).

2.7.3.4 Electrochemical Chloride Extraction (ECE)

Electrochemical Chloride Extraction is a process whereby the chlorides that are present within the concrete and would cause the passive layer on the steel to break down are removed. It is also known as desalination. This is done by temporarily applying a strong electric field across the cover concrete region, between the reinforcement and an external anode placed on the surface of the concrete. This causes negatively charged ions to migrate from the reinforcing to the external anode, and the result of this is threefold –

- Decreases the potential of the reinforcement
- Increases the hydroxyl ion concentration
- Decreases the chloride concentration at the level of the steel.

(Mackechnie and Alexander, 2001).

A schematic illustration of the process is shown in Figure 2-12:

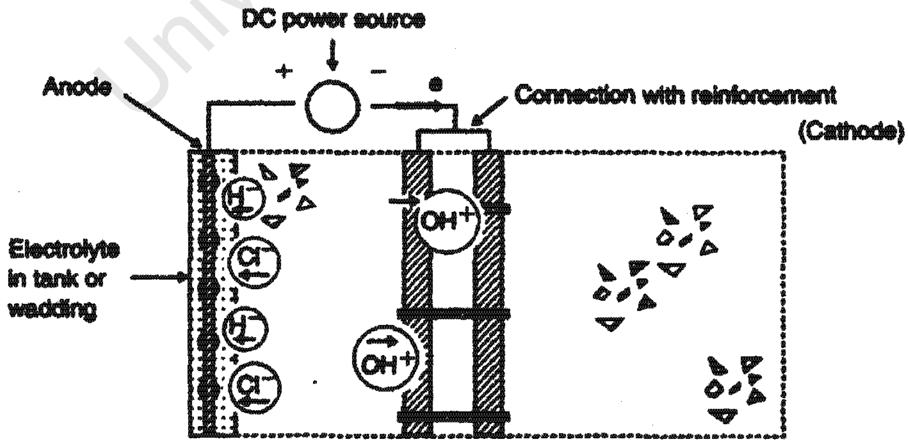


Figure 2-12: Electrochemical Chloride Extraction (Pullar-Strecker, 2002)

According to Mackechnie and Alexander (2001), the success of ECE is dependent many factors, including the following:

- The level of deterioration of the concrete in question
- The physical properties of the concrete, such as cover and spacing of reinforcement
- The length of application and current level
- The ability of the cover concrete to conduct the electrical field
- The presence of defects in the cover concrete, such as cracks and spalls, that might cause uneven chloride extraction

According to Pullar-Strecker (2002), the typical current density that is applied in Electrochemical Chloride Extraction is 1 A/m^2 of reinforcement, applied at a voltage of between 20-100 V. The current is applied over a period of between 2-10 weeks. Mackechnie and Alexander (2001) suggest that for complete chloride extraction to occur, the charge must be applied over more than 8 weeks.

The negative aspects of this type of treatment are as follows:

- It may be possible for surrounding concretes that contain chlorides to replenish the concrete over time, and to reinitiate corrosion.
- The process requires a temporary power supply during application, which can be problematic in isolated structures.
- If the cover to the reinforcement is not constant through the member, it is possible to short circuit the electric field.
- In prestressed concrete structures, there is the danger of hydrogen embrittlement occurring.
- It is possible to accelerate Alkali-silica reaction with this treatment, and special care must be taken with reactive aggregates.

2.7.3.5 Cathodic Protection

Whereas Electrochemical Chloride Extraction is a temporary treatment, cathodic protection is a permanent application of an electric field in the cover concrete. The impressed current ensures that the reinforcement is cathodic and that corrosion will only occur at an externally applied anode, commonly known as a sacrificial anode.

There are two types of cathodic protection, namely active cathodic protection and passive cathodic protection, and the difference between these two is the manner in which the 'power' is supplied to ensure that the steel remains cathodic (Pullar-Strecker, 2002).

Active cathodic protection is a process whereby the steel reinforcement is kept cathodic by the application of a current from an external power source, and a transformer-rectifier supplies the current between the reinforcement and an anode, which is placed on the surface of the concrete. These anodes may range from conductive paint overlays to titanium mesh embedded in shotcrete. According to Mackechnie and Alexander (2001), these types of anode systems are designed for a minimum service life of twenty years, but may last more than fifty years.

Passive cathodic protection is where there is no externally applied electric current to promote the movement of ions, but the anode that is placed at the surface of the concrete is a metal that is higher than steel in the electrochemical series, such as zinc. It will therefore corrode instead of the reinforcement steel, and supply electrons to the cathodic steel surface. The use of sacrificial anodes to prevent steel corrosion has been used for over a hundred years in applications where the steel is wet, such as in the hulls of ships. In concrete, however, the resistance of the concrete to the movement is usually too high for the use of these systems without an external power source (Pullar-Strecker, 2002). These types of sacrificial anode systems are therefore most successful in temperatures above 20^0C and where the concrete is wet and thus resistivity is low (Mackechnie and Alexander, 2001).

Figure 2-13 shows a typical impressed current cathodic protection system:

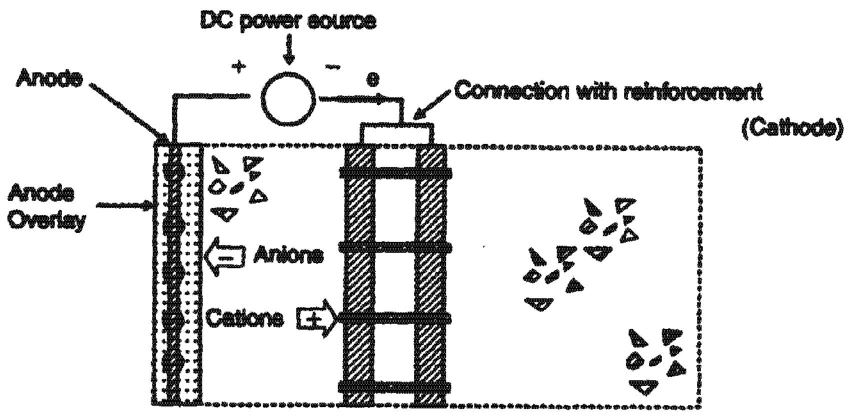


Figure 2-13: Cathodic Protection (Pullar-Strecker, 2002)

Of all of the repair options, cathodic protection is the most successful in practice in that it directly affects the corrosion process, and provides a much longer lasting solution with a reduced likelihood of corrosion occurring again. The cost is, however, relatively high compared to other treatment options. Besides this initial cost consideration, there are other factors that would influence the choice of this treatment method:

- The reinforcement must be electrically continuous.
- As with ECE, the cover must be uniform and must not contain any major defects such as delamination.
- Special cares must be taken when working with prestressed members and alkali reactive aggregates.
- There must be a permanent power supply available to power the operation.

(Mackechnie and Alexander, 2001)

It is important that the correct amount of current is used in the system, because if there is too much negative potential in the system it could result in negative side effects such as hydrogen embrittlement of the reinforcing steel or reinforcement bond loss.

Cathodic protection has been shown to be successful in the Western Cape, with a jetty in the Simonstown Harbour having been successfully rehabilitated to full

2.8.1 Service Life

Mehta (1991) notes that it is the current assumption at the design stage that many, if not all, concrete structures will go through their service life maintenance free. Current repair practice shows that this is not a valid assumption to make. A study of marine structures in the Western Cape revealed that in most of the structures investigated will require major repairs before their design life is reached (Strohmeier 1994). In the design phase, it is expected of the engineers involved that they design the structure to withstand the expected deterioration processes safely until a specified age.

The engineer who has to design the repair works for a deteriorated structure must make key decisions as to how much money to spend and to what level the structure must be restored. Figure 2-14 is a model of a typical life span of a structure that does not undergo repair, from construction to failure by corrosion damage:

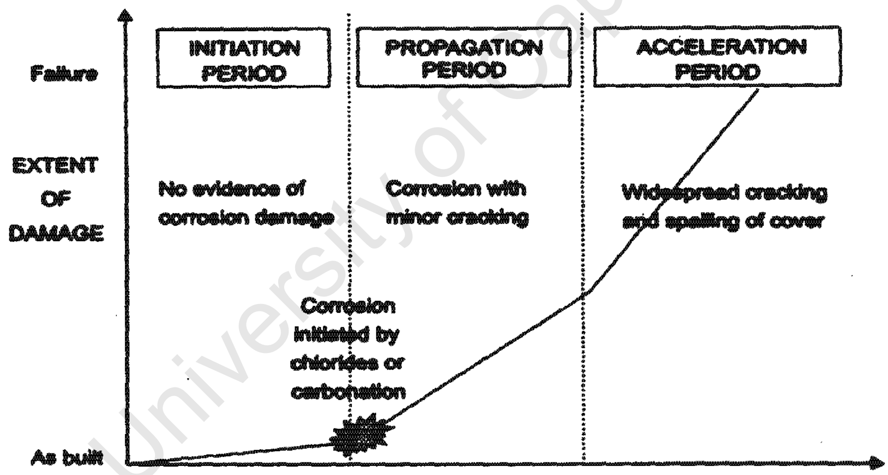


Figure 2-14: Tutti Diagram (Mackechnie and Alexander, 2001)

This model represents the typical life span of a reinforced concrete structure. It is divided into three periods, namely the initiation period, where the corrosion causing elements have not yet penetrated to the level of the steel, the propagation period, where corrosion has been initiated and the protective layer covering the steel has been broken down, and the acceleration period, where corrosion causes cracking and spalling, and so the access of corrosion-causing chlorides and carbonates into the material is improved.

A rapid increase in the level of deterioration with increasing time is evident in Figure 2-14. It is therefore expected that the cost of repair will also increase with the level of deterioration. Strohmeier's research confirmed this, and he further stated that repair work should be performed as soon as distress is noted (1996). This is difficult to adopt as a rule, however, because in practice the level of distress might have different implications for different engineers and owners. Thus the decision has to be made as to what practical level of deterioration the structure is allowed to reach before rehabilitation. It has already been seen in the previous section on repair options how different repair methods are suited to different levels of damage.

The following Table 2-12 is taken from a study done by Scott (1997), which shows the typical extension of service life that is available through certain repair measures.

Table 2-12: Life extension for various repairs (Scott, 1997)

Repair Methods	Possible Useful Life
Surface treatment	*Approximately 10 years (based on European data)*
Reconstruction of damaged areas (patch repairs)	*Variable, depending on rate of penetration of contaminants:* • *Complete removal of contaminated concrete – 10 years or more* • *Partial removal of contaminated concrete – fairly short (a few years)*
Cathodic Protection	*Variable, depending upon components of the system. Minimum of 20 years*

The following graphs show what impact these repair methods would then have on the basis of a Tuutti type model, showing deterioration plotted against age.

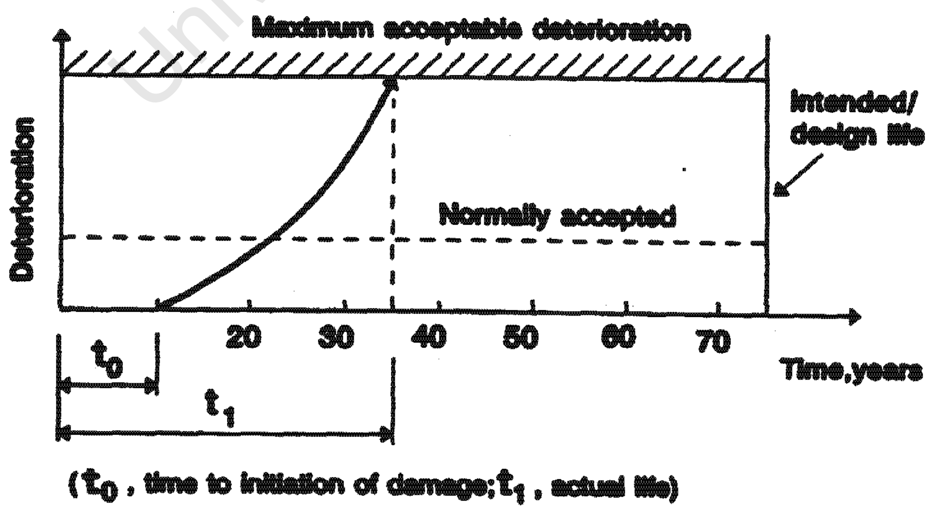


Figure 2-15: Do nothing option (Strohmeier, 1994)

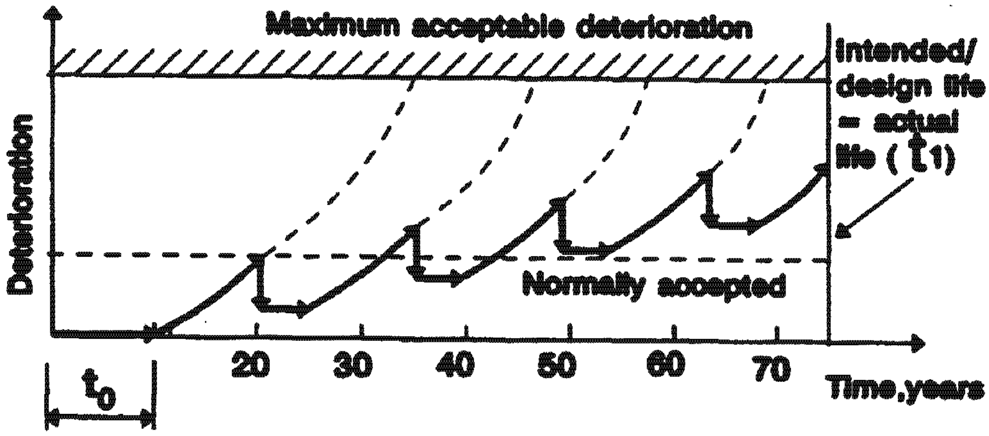


Figure 2-16: Regular minor repairs, for example patch repairs (Strohmeier, 1995))

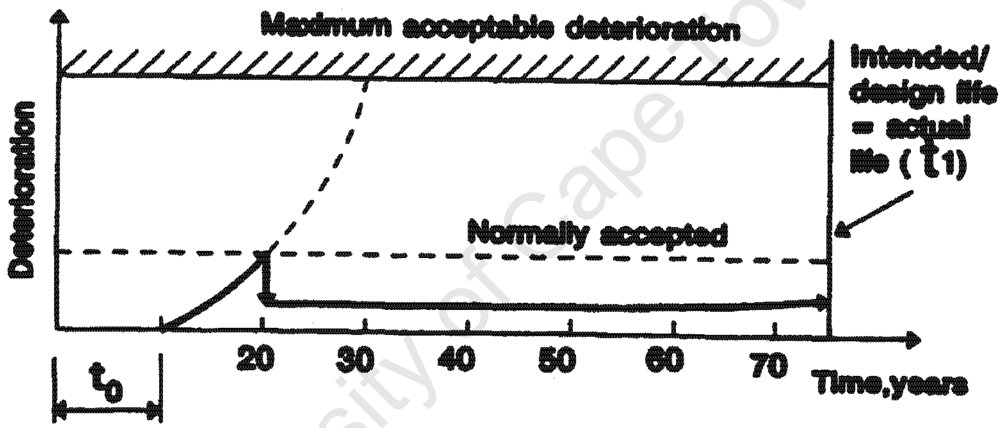


Figure 2-17: Cathodic Protection (Strohmeier, 1995)

Whilst the above three diagrams are purely for illustrative purposes, the effect that the different options have on extending the service life of a structure is clear. The third diagram, which represents cathodic protection, suggests that it is possible to indefinitely prolong the service life of a structure, but the practicality of this system is not that simple in that there are many factors, including the high cost and permanent current requirements that influence the implementation of this method. In the South African context, there have only been a few cases where this has been successfully adopted, for example at the Simonstown Jetty (a historical site).

The main parameters that govern the previous figures from Strohmeier are the required service life and the acceptable level of deterioration. Every structure will encounter some deleterious mechanisms that will degrade the structure to a point of failure. At the repair stage, if these mechanisms have resulted in a need for repair at a stage that is earlier than predicted, which is the standard case as concrete structure are designed for 'maintenance free service lives', it is clear that applying the same approach in repair will result in failure. Lessons can be learnt from structures that prematurely show damage.

2.8.2 The basic 'rules' of repair

According to Mackechnie and Alexander, in developing such repair strategies there are three basic rules that should be addressed (2001):

- The level of deterioration of the structure that is to be repaired will indicate the performance that is to be expected from the structure after repair. In other words, the repair must be able to withstand the deterioration processes that cause the necessity for repair. This may seem like an obvious requirement, but it can be the case that a standard method of repair treatment that is applied without the full understanding of the damage-causing mechanism involved can result in a detrimental effect on the structure instead of a beneficial effect.
- Corrosion rates and damage increase dramatically with time, and thus the timing as well as the method of repair is crucial to the successful rehabilitation of a structure. Structures that are left for too long without rehabilitation may not be able to be restored to full serviceability.
- Different treatments are able to repair and restore the structures to different levels. Some repair methods, if instituted incorrectly (such as patch repairs causing incipient anodes), will increase the rate of deterioration of the structure as opposed to improving it.

Unfortunately it is often not possible to make the above three considerations freely without the limitations of timing and budget, or even the lack of available necessary information to guide decision processes. Mackechnie and Alexander state that the reasons that repairs often perform poorly include the following five aspects (2001):

- There is a lack of understanding on the part of the engineers involved of the corrosion and deterioration processes. If the fundamentals are not clearly understood, it will not be possible to adequately repair the structure in question.
- Proper investigation and testing are not made prior to the important decisions regarding the state of the structure. Thus the accurate state of the structure is not fully assessed and the engineer might propose remedial works that are inadequate are even unnecessary.
- As mentioned earlier, there might not be sufficient funds available to fully repair the structure.
- Inappropriate repairs could lead to poor performance, where the specified method of repair is not the most suitable for the rehabilitation of the structure.
- The actual institution of the repairs on site could be defective in procedure. If proper site practices are not ensured, it is unlikely that the repair will be successful.

Thus there are financial, forensic, procedural and fundamental considerations to be made and balanced against each other. It is shown through common practice, however, that the economic considerations of such repair have the greatest impact and they will dictate the timing and scale of repairs (Mackechnie and Alexander, 2001). The most common practice mistakenly looks only at the short-term cost associated with one single set of repairs at a single stage of the structure's design life, instead of adopting a more holistic view of how cost can be minimised throughout the entire life of a structure.

2.8.3 Life Cycle Costing

Very few clients (i.e. the owners of the structures) are capable of effectively budgeting for future repairs, and this means that the repair works are commonly very poorly funded (Mackechnie, 2001). The ideal scenario is to construct structures that do not require any future repair works, but the high cost upfront is the major hurdle in this regard, and it can be hard to persuade clients that money will be saved over an extended period of time. In work done by Scott, the above was shown to be true in that it is cheaper to spend more money upfront than even to defer funds for repair at a later stage.

Table 2-13: Total Life cycle costs of typical beam members exposed to marine environment (Scott, 1999)

Option	1	2	3	4	5
Original Design	*60 MPa 30% fly ash 55 mm cover*	*60 MPa 30% fly ash 30 mm cover*	*60 MPa 30% fly ash 40 mm cover*	*60 MPa 30% fly ash 40 mm cover*	*60 MPa 100% PC 75 mm cover*
Repairs/maintenance	*None*	*Surface treatment at 10-year intervals*	*Patch repairs after 20 and 35 years*	*Cathodic protection after 20 years*	*Patch repairs after 15, 25 and 35 years*
Relative Costs	*1.0*	*2.0*	*2.3*	*3.0*	*3.5*

Notes on repair options:-
Option 1. Durability design for maintenance free 40 year service life
Option 2. Based on anticipated life of surface treatment
Options 3-5. Based on the likely stage at which spalling damage becomes excessive
Option 5. Design required by SABS 0100:1992

Within the scope of current repair projects and new structures, however, engineers are able to make use of the advantages that have been in the fields of forensic analysis, repair methodology development and serve life predictions in order to increase the reliability with which they can predict the future performance of a structure. This is not just a consideration to be made in new structures, but also in repair contracts as the exponential nature of corrosion damage escalation results in exponentially increasing costs associated with delaying concrete repair.

2.9 Conclusion

A review of the aspects concerning the repair of reinforced marine concrete structures has been presented. This progressed from the fundamentals, looking at an understanding of deterioration processes in marine concrete, focussing specifically on corrosion damage as induced by chlorides, to the assessment of such damage, the options for repair and what the necessary considerations should be in making decisions regarding concrete repair and rehabilitation. The research to be presented in the following chapters will investigate repair contracts for local reinforced concrete structures that are situated in the marine environment, and that already show clear signs of corrosion. An assessment of the contract documentation for these repair works is presented, followed by a description of the repair strategies that are being implemented by the engineers on site. In chapter 6 some forensic work that was performed at two of the locations is presented and chapter 7 critiques the works from a number of viewpoints pertaining to concrete material repair and rehabilitation.

3 Methods of Investigation

3.1 Introduction

This chapter discusses the methods of investigation that were used, detailing what each investigation entailed and their relevance to the research conducted in this thesis. These investigations serve as tools by which the repair methodologies used in the four contracts can be assessed and quantified, and then in turn compared and critically reviewed.

The purpose of this chapter is to help with understanding the results that will be presented later, and to give some idea of the limitations and fundamental mechanisms of each investigation. The following are the methods used in this work:

- Review of Contract Documentation
    - Project Specifications
    - Construction Drawings
- Discussions and Meetings with Consulting Engineers
- Visual Condition Surveys
- Invasive Techniques (Selected Sites)
    - Chloride Profiling
    - Penetrating Corrosion Inhibitor Test

3.2 Review of Contract Documentation

3.2.1 Project Specifications

Chapter 4 reviews the project specifications for the repair works at the various harbours. It is evident that these documents are copied from project to project, and that the real instructions for repair methods exists either in the form of drawings or on-site supervision.

The specifications given in these documents are commented on in terms of suitability, practicality and likelihood of success.

3.2.2 Construction Drawings

The most prevalent form of instruction between the consulting engineers and the contractors on site is in the form of construction drawings, which give details of the work that is to be carried out. The level of detail contained in these drawings and the clarity of the instructions given therein are important in ensuring quality workmanship.

The drawings are commented on for each location, and in some instances certain details have been reproduced to show the details of the repair works that have been carried out.

3.3 Discussions and Meetings with Consulting Engineers

During the course of this investigative work it was also felt necessary to communicate with the consulting engineers involved at each location. The purpose of this was two-fold: Firstly, it served as an introduction to the various sites and works, but secondly, it also gave an opportunity to ascertain the repair philosophy that was being used, and how that was affecting the chosen methodologies. In addition to this, an attempt was made to gauge the 'attitudes' of the respective engineers to the various repair options.

3.4 Visual Condition Surveys

3.4.1 Objectives

Visits were made to all four contract sites. The objective of the various site visits was to gain familiarity with the structures that were being repaired. In this way it was possible to do two things: Firstly to relate the locations in order to contrast the adopted repair strategies, and secondly to be able to relate the level of deterioration at each site/structure to the methodologies that were implemented. These investigations form the basis of much of the discussion at the end of the thesis. Unlike the particular forensic tests that were performed, the investigations made in these condition surveys were fairly simplistic, relying mainly on visual surveys for data.

3.4.2 Limitations

3.4.2.1 Specific Limitations

A number of different types of structure were investigated: these were constructed at different dates, using different materials and methods, and thus the performance of each is not expected to be similar. Not all of this 'base' information is available, and many of the structures show signs of previous repair measures, which will also have influenced the performance over time.

As these structures are all in the marine environment, accessibility was often a problem, and it was not possible to survey some of the more inaccessible places such as underwater and underneath some jetties and quays.

Another limitation of this survey was the timing of the investigations, which were performed as soon as possible within the timeframe of this thesis, but often not at the most suitable time for investigations – most were performed whilst the repair contracts were underway, and this should be considered in perusing the collected data.

3.4.3 Methodology

As far as possible with the available information, the performance of a structure (whether showing visible signs of deterioration or not) is defined in terms of:

Properties of the structure

This includes material properties such as the concrete type, type of member/structure, the age of the structure, possible repair history, and location relative to deleterious mechanisms.

Damage indicators

Visible damage, for example in the form of rust staining, cracking, and spalling.

Cause

Whether or not estimation can be made from the visual survey as to the cause of the deterioration, for example whether the damage is caused by mechanical means or by reinforcement corrosion.

3.4.4 Focus of Investigation

Chloride induced corrosion damage

The damage of reinforced concrete as a result of chloride-induced corrosion is a theme that runs throughout the entirety of the thesis, and so it is this damage mechanism that will be highlighted as the most important to address. It is also the most common cause of damage in the structures that are being investigated.

Performance of previously performed repair work

The performance of previous repair measures can also give insight as to how the current measures will perform in the future, and investigations will be made into these.

Current repair works

Current repair works are identified and investigated in order to make a comparison of the differences in repair strategy between the four contracts.

3.5 Invasive Techniques

3.5.1 Chloride Profiling

This procedure quantifies a chloride profile from the surface into a concrete, representing the percentage chloride present (expressed as a percentage by mass of binder) in a concrete sample, plotted against depth. The results are typically shown in graphical format, and are useful in understanding how chlorides are able to move through concrete.

The typical chloride profile follows a roughly hyperbolic decreasing shape, moving from a high surface concentration and tending to either a zero concentration or the 'cast in' chloride concentration. The properties of the concrete will dictate the ability of chlorides to be transported, and thus this test will also give an indication of the 'quality' of the concrete in relation to the environment and time.

3.5.1.1 Sampling

A common method of sampling is to extract cores from the in-situ concrete member that is being investigated. These cores are drilled to a specific depth, which is state that the susceptibility of a structure to corrode and to show signs of corrosion is influenced by the following factors:

- The size of the reinforcement will affect how easily the cover concrete will be able to delaminate. Also the steel bar spacing will impact on the likelihood of delamination.
- The depth of cover, in that if the steel is at a large enough cover depth, full oxidisation will not be possible.
- The resistivity of the concrete, connected to the moisture content will determine the development of anodic and cathodic sites.
- Cracks provide pathways for chlorides to ingress to the level of the steel, and promote corrosion. The crack may not necessarily have been caused by the corrosion itself, for examples cracks may appear under working loads.
- Connected to the above is the possibility of corrosion being promoted in highly stressed members, i.e. stress-induced corrosion.

2.6 Assessment of Corrosion Damaged Concrete

2.6.1 Introduction

This section refers to the inspection and assessment of the level of damage that has occurred and could potentially occur in reinforced concrete. According to Pullar-Strecker, such inspections are typically required at the change of ownership of a structure, through a routine health check or possibly as a result of visible corrosion damage (2002). For infrastructure such as bridge, harbours etc., this would entail regular maintenance inspections by the authority.

Options for corrosion assessment cover a wide range of different types of investigation, from general visual inspection to more detailed testing methods that require time, expense, and specialised expertise. The option that is chosen would be set out by the client at the start of the work, and would usually move from general to specific over the course of the repair contract. Baker (in Mays, 1992) notes the importance of a correct and comprehensive diagnosis at an early age, stating that in order to successfully repair a structure, the engineer developing the repair dependant on the requirements of the investigation, most commonly at least to the depth of the reinforcement.

The core is taken to a laboratory and sliced into increments, and each increment is pulverised in order to prepare it for solution. This is shown in Figure 3-1.

Figure 3-1: Sliced cores ready for Chloride Testing

An alternative method to this approach is shown in the Figure 3-2 below, whereby the concrete member to be investigated is drilled and the powder is collected. Care is taken to ensure the correct depth measurement of the drill bit, and this could save time and money in the lab by resulting in an already pulverised sample.

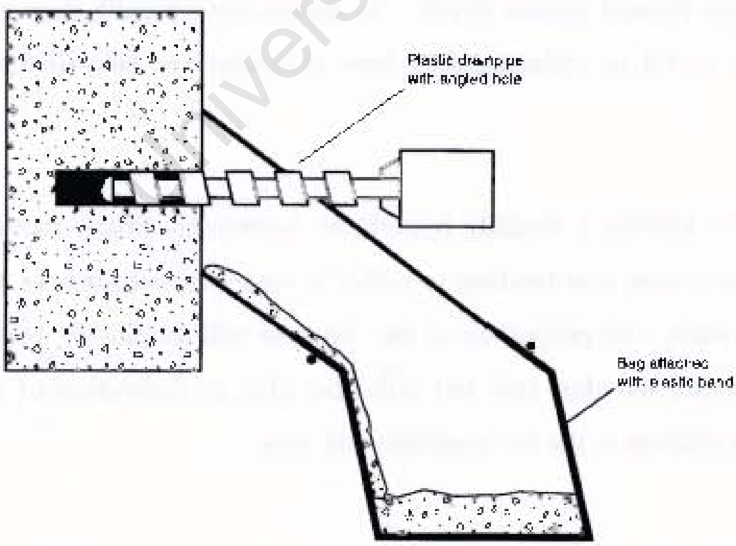


Figure 3-2: Dust collection for Chloride Testing with drilling

3.4.4 Focus of Investigation

Chloride induced corrosion damage

The damage of reinforced concrete as a result of chloride-induced corrosion is a theme that runs throughout the entirety of the thesis, and so it is this damage mechanism that will be highlighted as the most important to address. It is also the most common cause of damage in the structures that are being investigated.

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The performance of previous repair measures can also give insight as to how the current measures will perform in the future, and investigations will be made into these.

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Current repair works are identified and investigated in order to make a comparison of the differences in repair strategy between the four contracts.

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The typical chloride profile follows a roughly hyperbolic decreasing shape, moving from a high surface concentration and tending to either a zero concentration or the 'cast in' chloride concentration. The properties of the concrete will dictate the ability of chlorides to be transported, and thus this test will also give an indication of the 'quality' of the concrete in relation to the environment and time.

3.5.1.1 Sampling

A common method of sampling is to extract cores from the in-situ concrete member that is being investigated. These cores are drilled to a specific depth, which is

Figure 3-3: Corrosion Inhibitor Test Kit

3.5.2.1 Sampling

The method of sampling adopted in this thesis was to extract cores from a structure to which the migrating inhibitor had already been applied. This was performed at a number of different locations in terms of depth relative to mean sea level and also to different types of concrete member.

3.5.2.2 Sample Preparation

The cores were cut in half longitudinally, and one half was sliced into increments of 10 mm. The other half was retained for confirmatory testing if deemed necessary. A 'control' was tested, i.e. performing the same test on an untreated sample of concrete. Each incremental disc was crushed into a fine powder of particle size 1mm or less using the tools provided in the test kit (hammer and plate). 1g of this powder is mixed with 1 ml of distilled water, mixed vigorously and left to settle for at least half an hour. The supernatant was then removed and filtered through a syringe to produce the liquid specimen that was tested. This was tested alongside the control sample, as well as against a standard solution, which represented the minimum amount of inhibitor that the test can detect.

The pulverised sample is then digested in concentrated nitric acid in order to release all chlorides.

3.5.1.2 Analysis

The samples are then analysed using potentiometric titration in accordance with BS 1881: Part 124, titrating the digested solution against silver nitrate. A result is produced in terms of percentage chloride by mass of binder, and it is noted that this is an overestimate of the durability threat as it represents the total chlorides present and not only the corrosion-inducing free chlorides.

3.5.1.3 Modelling

Each depth increment will have a chloride content value, and from this data a profile can be drawn to interpolate or extrapolate other values. Using Ficks 2^{nd} law of diffusion to model the profile, future chloride levels can also be predicted.

From this, values for the Diffusion coefficient and Surface concentration of the concrete can be extracted, and the depth of the 0.4 % Chloride threshold can be calculated.

3.5.2 Penetrating Corrosion Inhibitor Test

This test kit is provided by the manufacturer of the penetrating corrosion inhibitor, and is designed to be a qualitative assessment of the ability of the inhibitor to penetrate to suitable depths. The test kit is suited to on-site use, and does not require large expense, time or expertise to produce results. As with the chloride concentration test, it is possible to use a series of results at different depths to produce a 'profile', although the typical goal of such a test is to ascertain whether or not the inhibitor has penetrated in sufficient quantity to the depth of the reinforcement.

3.5.2.3 *Test Procedure*

The samples are extracted from their containers using a micro pipette and applied to a white chromatography plate in the form of a circular dot, not greater than 5mm diameter. This was performed in 5-6 layers, drying the plates with a hot air blower between each application. Chemical solutions were applied and dried for 1-3 minutes.

3.5.2.4 *Analysis*

After the above is performed, the sample will change colour to a reddish brown if it contains the inhibitor. The control sample should remain white, but may contain a slight discoloration due to the pollution of the concrete or due to slight contamination during the test. The standard test solution should discolour to light brown.

The test specimens should discolour to a shade of light brown-red. If not, it does not necessarily mean that the sample does not contain traces of the inhibitor, but rather that more precise methods should be used for further analysis.

Figure 3-4: Chromatography plate reading for inhibitor test (Heiyantuduwa, 2001)

An example of a plate showing the test colours is given above in Figure 3-4. This shows the difference between a good (i.e dark) brick red reading (A-C) and a paler 'trace' reading (D).

The qualitative rating that was given to the results in Heiyantuduwa's thesis is shown below, and alongside is the breakdown given to the tests that were performed under this thesis.

Table 3-1: Qualitative Rating of Inhibitor Testing (Heiyantuduwa, 2001)

Test kit colour	Qualitative rating
Brick Red	*Excellent*
Pink	*Good*
Pale Pink	*Adequate*
Trace	*Inadequate*

Table 3-2: Qualitative Rating used in this thesis

Test Kit Colour	Rating
No Colour	*No Trace*
Circular Pink stain	*Light Trace*
Solid Pink	*Trace*
Pink to Red	*Good Trace(Medium)*
Brick Red to Purple	*Good*

3.5.2.5 Limitations

This test is capable of providing insight as to how far the inhibitor has ingressed into a concrete within a reasonably short space of time. The major limitation is that unless corrosion rate monitoring is performed before and at a long time after application, the engineer has to rely on the manufacturer's claims for the success of the product.

3.6 Discussion

These investigations are not an attempt at accurately studying every single structure investigated from a forensic point of view. Instead, they operate at two different levels:

Firstly the inhibitor and chloride tests give particular results that have direct application to the repair methodologies adopted at the relevant sites, and will be discussed further in the form of case studies.

Secondly the visual surveys and contract documentation review will inform discussion of repair strategies and philosophy four all four sites as an indicator of current practice.

4 Contract Documentation: Project Specifications and Drawings

This chapter presents the project specifications for the four repair contracts, moving from standardized specifications to the more particular requirements as prescribed in contract drawings. Particular focus is made on concrete durability aspects.

4.1 Standardized Project Specifications (SABS 1200G)

The RAMP programme adopted the SABS 1200G guidelines for standardized specification of construction works. Thus for all four contracts the requirements were the same and reference is made to SABS 1200G in particular in terms of concrete requirements.

4.1.1 Exposure Conditions

The classification of exposure condition, 'very severe conditions', was assigned to all four locations because of their close proximity to seawater. Scope was also given to allow for a more severe classification in the specification if needed.

4.1.2 Materials

Table 4-1: shows the cements that were specified for the RAMP contracts, and includes the allowance of the use of blended cements. This is beneficial in terms of durability and is common practice in current construction contracts.

Table 4-1: Cements allowed in the RAMP contracts

Cement	Description
CEM I 42,5	*Portland Cement*
CEM I 42,5R	*Portland Cement, rapid hardening*
CEM II/B-V	*Portland fly ash cement*
CEM II/B-W	*Portland fly ash cement*
CEMIII/A	*Blast furnace cement*

Allowance is also made for the on-site blending of cements, conditional on compliance with the relevant standards as shown in Table 4-2.

Table 4-2: Cement extenders and the governing SABS standards

SABS EN 197-1	*Cement - Part 1: Composition, specification and conformity criteria for common cements*
SABS 1491-1	*Portland cement extenders - Part 1: Ground granulated blast furnace slag*
SABS 1491-2	*Portland cement extenders - Part 2: Fly ash*
SABS 1491-3	*Portland cement extenders - Part 3: Condensed silica fume*

4.1.3 Construction

4.1.3.1 Cover

For three of the four contracts, reference was made to the contract drawings for cover information. For the St Helena Bay contract, however, a 'blanket' governing value of 75 mm was specified as minimum cover. This is noteworthy as it appears that the engineers on the St Helena contract have opted to specify a relatively high cover in order to conform to a more conservative design.

4.1.3.2 Concrete (Structural)

The common concrete material specifications are given in Table 4-3, and show the options that are available to the engineer for concrete choice.

Table 4-3: Typical Concrete Mixes Recommended by Contract Documents

Concrete type	Cement type and % content	Extender type and % content	Minimum cement + extender content kg/m^3	Maximum water/cement ratio
Steel reinforced	*CEM 1 50%-60%*	*GGBS 40%-50%*	*420*	*0.40*
	CEM 1 70%-75%	*FA 25%-30%*	*420*	*0.40*
Plain	*CEM 1 100%*	*Nil*	*340*	*0.50*
	CEM 1 75%	*FA ≤25%*	*340*	*0.50*
	CEM 1 35%-65%	*GGBS 35%-65%*	*340*	*0.50*
	CEM 1 65%-74%	*FA 26%-35%*	*300*	*0.55*

Note:

1) CEM I may be CEM I 42,5N or 42,5R
2) GGBS – Ground Granulated Blast Furnace Slag
3) FA – Fly Ash
4) Factory blended cements (CEM II/B-V, CEM II/B-W or CEM III/A) will be accepted provided that they conform to one of the blends specified in the table. The Contractor shall supply certification thereof.
5) Water-reducing admixtures may be used to improve workability. The water cement ratio shall include the water content of admixtures.

The options given for selection above, especially those for steel reinforced concrete, show acceptance of current practice regarding the use of cement extenders. The use of Pozzolans such as Fly Ash or latent hydraulic binders such as Ground Granulated Blast-Furnace Slag have been proven to increase the durability of reinforced concrete in the marine environment.

4.1.3.3 Curing and Protection

The curing requirements given below are standard recommendations for projects of this nature, but in practice these are very rarely fully adopted, or adequately performed on site.

Table 4-4: RAMP Curing Requirements

Curing Requirements:	
For plain concrete	**For steel reinforced concrete:**
i) Retaining forms in place on vertical surfaces provided they are made with non-absorbent facing materials.	i) Covering with burlap or hessian or similar moisture retaining materials and keeping the concrete continuously wet.
ii) Ponding of water on horizontal surfaces.	ii) Continuous spraying with water.
iii) Covering with sand, earth, straw, sawdust, cotton, jute, burlap or hessian or similar moisture retaining materials.	iii) Releasing the forms slightly and allowing a flow of water between the form and the concrete by continuous spraying with water.
iv) Continuous spraying with water to ensure that the concrete surface remains continuously moist and is not allowed to dry out.	iv) Curing methods using sealing materials such as plastic or liquid membrane forming compounds shall not be used for steel reinforced concrete structures due to the low W/C ratio of the concrete mix. The water provided by the moist curing is required for completion of the hydration of the concrete in the cover layer.
v) Covering with plastic sheeting, waterproof or other curing paper.	
vi) Liquid membrane-forming curing compounds may be used. Only resin type compounds will be permitted.	
The curing period for concrete containing CEM I only shall be 7 days. The curing period for concrete's containing CEM I plus cement extenders (GGBS, FA) shall be 10 days.	
All water for curing shall be clean, fresh water and under no circumstances shall seawater be used.	

4.1.4 Comment on Project Specification

While the more detailed discussion on the use of project specifications will follow in Chapter 7, it is important to note that it is the specifications on the drawings themselves that will usually take priority for a given project. What results is that the 'theory' given in these specifications often does not correlate to what is performed on site. The point here is that onsite practice does not necessarily correlate to the written specification. Also important to note is that these are very common specifications, and so a comparison of only these specifications across the investigated sites on this basis would not prove insightful.

4.2 Particular Specifications

The particular specifications used in the four contracts cover methods used in the rehabilitation of concrete structures that are not covered in SABS 1200G. It is clear that while some editing and minor alteration was performed, essentially the same document was used throughout the four contracts. The key elements as they pertain to concrete durability and the works performed in these contracts have been identified and critiqued in this section.

4.2.1 Materials

Table 4-5 shows the repair materials that were specific for the RAMP contracts under the particular specifications.

Table 4-5: Repair materials as specified in RAMP contracts

Repair Materials		
Large Crack Sealing:		
Temporary surface sealing:	*Sikadur 31 Thixotropic Epoxy Resin Adhesive*	*Epoxy Based*
Crack injection and grouting:	*Sikadur 52 Low Viscosity Epoxy Injection Resin for cracks less than 5 mm width*	
	Sikadur 42 Epoxy resin based flowable grout for cracks not less than 5 mm width	
Cracked, spalled and delaminated areas:		
Reinforcement corrosion protection and bonding coat:	*Sika Monotop 610 Bonding slurry and anticorrosion primer or Sika Top Armatec Epocem*	*Barrier Protective Coating*
Repair mortar - overhead and vertical	*Sika Monotop 615 HB Repair Mortar*	*Cement Based Mortar*
Repair mortar - horizontal	*Sika Grout 212 expanding cementitious grout for repairs less than 60 mm thickness*	*Cement Based Grout*
	Sikacrete 214 structural repair concrete, for repairs greater than 60 mm thickness	*Cement Based Grout*
Dowel bar grouting:		
In vertical holes and horizontal slots:	*Sika Grout 212 Expanding Cementitious Grout*	*Cement Based Grout*
In horizontal holes:	*Sikadur 31 Thixotropic Epoxy Resin Adhesive*	*Epoxy Based*
Protecting concrete surfaces:		
Carbonation protective coating:	*SikaTop - Seal 107 Protective and Waterproof Coating*	*Barrier Protective Coating*
Penetrating corrosion protective coating:	*Sikagard 903 Corrosion Inhibitor*	*Corrosion Inhibitor*

Three general repair methods are described here: the first is that of crack injection, using low viscosity epoxy to fill cracks that have formed in concrete. This not only restores strength to the concrete, but also helps in preventing the ingress of deleterious agents.

The second used cementitious grouts and repair mortars to 'patch' spalled or badly cracked concrete. This method is widely used within these contracts, and the potential for failure of such methods was presented in 2.7.3.2, where it was discussed that if the neighboring concrete to a patch of repaired concrete is chloride contaminated, 'incipient' anodes will form adjacent to the patch site and corrosion will continue.

This method of repair is used on concretes that have spalled due to chloride induced reinforcement corrosion. The specific method for repair is as follows:

- Defective concrete to be removed and reinforcement exposed to a minimum depth of 25 mm behind the reinforcing bar
- Reinforcing bars corroded beyond usable extent are to be replaced by welded or lapped new steel bars
- Retained steel to be cleaned of rust and dirt by sandblasting, and two coats of anticorrosive coating are to be applied
- Repair mortar is to be used in areas with a depth of less than 60 mm. In areas of greater depth, a suitable structural concrete is to be used.

The specified method for protecting concrete surfaces comprises a barrier coating and penetrating corrosion inhibitor. Comment has been made in 2.7.3.3 that with concretes that have already been exposed to significant amounts of chlorides, these methods will most likely fail.

4.2.2 Trial Repairs

The contract documents clearly state that repair methods are to be tested before full scale work is implemented. This is to be performed in accordance with the manufacturer's instructions under observation of the engineer.

4.3 Project Construction Drawings

Table 4-6 below gives a summary of the information that is presented in the contract drawings for the four sites. It evaluates not only the information given in the drawings, but also the level of detail of many of the descriptive elements.

Table 4-6: Evaluation of Project Drawings

Contract	Laaiplek	Hout Bay	Saldanha Bay	St Helena Bay
Concrete Material Mix Information (Concrete Strength - MPa/Stone Size)	*45/19*	*50/12 or 50/19*	*Referred to project specifications*	*40/19*
Repair Material Information	*Products specified repeatedly*	*Products specified repeatedly*	*Very Detailed, number of varied products specified*	*None*
Cover	*Standard 60 mm*	*Standard 60 mm*	*Standard 60 mm*	*60 mm for precast elements, 75 mm elsewhere*
Repair Methodology Information	*Minimal detail and description shown*	*Minimal detail and description shown*	*Very detailed, processes explained clearly and concisely*	*None*
Level of drawing detail	*Clear and relatively basic*	*Clear and relatively basic*	*Very comprehensive, including construction sequencing etc.*	*Good, adequate for the construction works*
Other		*Included alternative options for repair*		*Included detail of past construction and repair works*

4.3.1 Laaiplek and Hout Bay

The same consulting engineers had been appointed at Laaiplek and Hout Bay, and the drawing standards for the two were similar. The concrete strength requirements shown on the drawings were 45 MPa and 50 MPa, and a standard cover of 60 mm was specified. Some repair products were specified repeatedly, but in general the repair methodology information was sparse or non-existent. In comparison with the other contracts, the drawings were simplistic and limited in detail.

4.3.2 Saldanha Bay

The drawings from the Saldanha Bay contract also showed a cover of 60 mm, and referred the reader to the standard project specification for concrete strength requirements. The repair methodology information shown in these drawings was very detailed, and included the use of various repair products. Construction phasing was also detailed in the drawings.

4.3.3 St Helena Bay

The drawings from the St Helena Bay contract specified the lowest concrete strength classification (40 MPa), but showed the highest prescribed cover. While detail on the structures was of a high standard, very little repair information was given and no repair products, such as surface treatments, were incorporated into the drawings. This was the only site, however, to include drawings that detailed the previous construction and repair methods.

These results are reviewed more critically in Chapter 7.

5 Descriptions of Repair Works

5.1 Introduction

Table 5-1 gives a summary of the repair works that were investigated within the four contracts presented in this thesis. The four contracts have been separated and the repair works are described in terms of location, type of repair and a brief description of the damage at each. It can be seen that the types of repair range from 'do nothing' options to complete reconstruction.

This chapter is divided into four sections, describing the repair works of each contract in more detail, giving material information and descriptions of repair works and methodologies. Important aspects of each are highlighted and commented on, while comparative discussion will be presented in chapter 7.

Table 5-1: Summary of Repair Works

Hout Bay			Laaiplek			Saldanha Bay			St Helena Bay		
Location	Type of Repair	Description of Damage	Location	Type of Repair	Description of Damage	Location	Type of Repair	Description of Damage	Location	Type of Repair	Description of Damage
Turning Bay North Mole (Alt. Design)	**Do Nothing**	Collapse in sections	West and East Breakwater	**Patch Repair**	Cracking and delamination present, settlements	Government Jetty	**Surface Treatment**	Corrosion staining, some cracking and spalling	Jetty 3 and 4	**Do Nothing**	Cracking, rust staining present
Jetty 1	**Surface Treatment**	Cracking, rust staining present	End of East Breakwater	**Partial Rebuild**	Settlement of structure	Trawler Quay	**Surface Treatment**	Cracking and spalling, large deflections	Gunning Jetty 2	**Patch Repair**	Cracking and delamination, rust staining
Original Design, Turning Bay North Mole	**Partial Rebuild**	Collapse in sections	Reinstated slabs West Breakwater	**Complete Rebuild**	Large settlements, steel loss due to corrosion	Government Jetty edge beams	**Patch Repair**	Cracking and spalling, mechanical damage	Quay 3	**Partial Rebuild**	Rust staining, high chloride levels
Turning Bay North Mole (Final Design)	**Complete Rebuild**	Total collapse				Pepper Bay	**Partial Rebuild**	Cracking and rust staining present	Deck of Jetty 1	**Complete Rebuild**	Cracking, moderate chloride levels
New Jetty	**Complete Rebuild**	Cracking and spalling				Repair Jetty	**Partial Rebuild**	Cracking and spalling to piles near cap	Supporting Beams Jetty 1	**Complete Rebuild**	Cracking and delamination, severe staining
						Trawler Quay Columns	**Complete Rebuild**	Total deterioration, capacity loss			

5.2 Laaiplek Harbour

5.2.1 Description of site

Laaiplek harbour is located at the mouth of the Berg River, and comprises two breakwaters on either side of the river mouth. It is a proclaimed harbour located adjacent to an environmentally sensitive wetland.

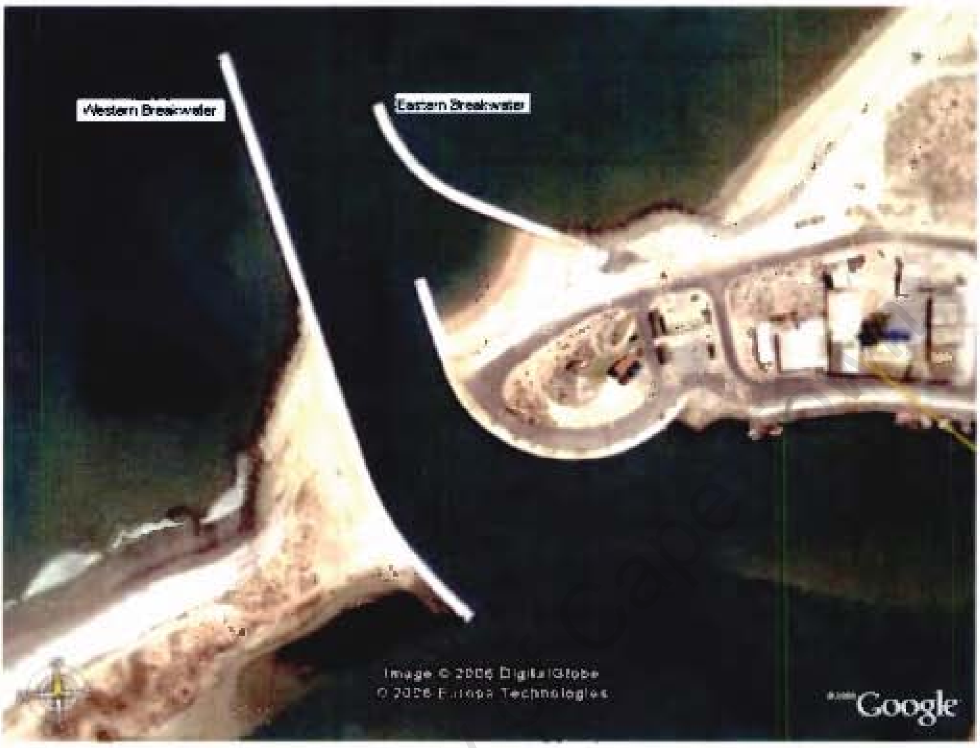

Figure 5-1: Aerial View of Laaiplek Harbour (Google Earth, 2006)

The majority of the work that was performed here comprised patch repairs to the long edge beams of each breakwater, and an example of this is shown in Figure 5-2.

Figure 5-2: Spalling to Western Breakwater (Typical across site)

Table 5-2: Summary of Repair Works: Laaiplek

Location	Type of Repair	Description of Damage	Description of Repair Works	Specified Cover	Concrete Mix (Strength, Aggr. Size)	Use of Specialist Repair Products
West and East Breakwater	**Patch Repair**	*Cracking and delamination present*	*Large quantities of patch repairs performed, concrete cut out and replaced with patch mortar, steel cleaned*	*60 mm*	*45/19 (OPC+30%FA or 50%GGBS)*	*SIKA Monotop Protective Coating, SIKAGARD 70*
End of East Breakwater	**Partial Rebuild**	*Major settlement of structure*	*New ring beam instated to support settled structure*	*60 mm*	*45/19 (OPC+30%FA or 50%GGBS)*	*SIKA Monotop Protective Coating, SIKAGARD 70, Grout also used*
Reinstated slabs – West Breakwater	**Complete Rebuild**	*Major settlements, steel loss due to corrosion*	*Reinforced concrete slabs replaced entirely*	*60 mm*	*45/19 (OPC+30%FA or 50%GGBS)*	*N/A*

5.2.2 Discussion of Laaiplek Harbour

A visual inspection of the Laaiplek Harbour site was performed by the author towards the completion of the repair works, and it was possible even at this late stage to identify many locations of damage due to chloride induced corrosion. The most predominant manner in which damage could be identified was through the cracking and spalling of the cover concrete along the long pile cap beams. Figure 5-3 shows such cracking, and further inspection showed that a large piece of the concrete had fully delaminated.

Figure 5-3: Cracking to Edge Beam

Upon removal of the concrete at such locations, as shown in Figure 5-4, the reinforcing steel was found to be at an advanced stage of corrosion. The two smaller photographs (Figure 5-5), show steel that was found at similar locations and the loss of cross-sectional area can be seen. This was estimated at 50% loss, which suggested an advanced level of corrosion present.

Figure 5-4: Spalling to Edge Beam

Figure 5-5: Severe loss of section due to corrosion

The survey showed the pernicious nature of chloride attack, with seemingly 'untouched' reinforced concrete being located adjacent to severely damaged concrete.

Figure 5-6: Use of new and existing elements

Another point that was observed during the investigations was the combination of new structural elements, such as the slab shown in Figure 5-6, with existing elements, in this case the pile capping beam. The difficulty with which certain elements can be repaired or replaced is clearly an important factor in the repair methodology. The long quay wall was being repaired on a patch basis, where the worst locations of damage were being identified and treated. With the slabs, however, a decision was taken to replace all of them as this would most likely have been the more cost-effective option for repair.

5.2.3 West Breakwater

Figure 5-7 below shows the slabs on the western breakwater that were replaced. It was decided that the damage to these elements was too great for rehabilitation, and that the demolition and the reconstruction of the surface slabs was a more suitable option.

Figure 5-7: Reinstating of new slabs on Western Breakwater (Note existing capping beams)

5.2.4 East Breakwater

There were numerous instances of patch repairs along the edges of the eastern breakwater too, but the main work at this location concerned the breakwater head. This had settled due the corrosion damage and the settled structure was linked to the breakwater using a new reinforced concrete edge beam.

Figure 5-8: Settled Slab at Lighthouse end of Eastern Breakwater

The literature review in chapter 2 notes the possibility of the failure of patch repairs due to the formation of 'incipient anodes' on either side of the new patch, restarting the corrosion process. Also key to the success of patch repairs is adequate surface preparations, something which is far from guaranteed in an environment that is difficult to work in such as this.

5.3 Hout Bay Harbour

5.3.1 Description of site

Hout Bay harbour is a proclaimed fishery harbour located on the west side of the Cape Peninsula. The harbour is protected by two large 'moles' and it was these moles along with a number of other reinforced concrete structures that were in need of varying levels of concrete repair.

Figure 5-9 Aerial View of Hout Bay Harbour (Google Earth, 2006)

The structures that involved corrosion damage due to chloride penetration were highlighted and investigated further. The consultants that were appointed to specify the repair works at this harbour were the same as the consultants for Laaiplek, but as can be seen in the summary Table 5-3, different specifications and approaches were used. This was because the different engineers within the consultancy were appointed to perform the work.

Table 5-3: Summary of Repair Works: Hout Bay

Location	Type of Repair	Description of Damage	Description of Repair Works	Specified Cover	Concrete Mix (Strength, Aggr. Size)	Use of Specialist Repair Products
Turning Bay North Mole (Alt. Design)	**Do Nothing**	*Major collapse in sections*	*Structure not repaired/rebuilt, left for later assessment*	*N/A*	*N/A*	*N/A*
Jetty 1	**Surface Treatment**	*Minor cracking, rust staining present*	*Surface applications of barrier coating or corrosion inhibitors*	*N/A*	*N/A*	*SIKA Corrosion inhibitor and Armatec Barrier Coating*
Original Design, Turning Bay North Mole	**Partial Rebuild**	*Major collapse in sections*	*Reinstate new span to structure. Approach discarded upon condemnation of entire structure*	*60 mm*	*50/12 (OPC+30%FA or 50%GGBS)*	*SIKADUR Epoxy, Monotop protective coating*
Turning Bay North Mole (Final Design)	**Complete Rebuild**	*Total collapse*	*New structure rebuilt at same location, using similar design concepts*	*60 mm*	*50/19 (OPC+30%FA or 50%GGBS)*	*N/A*
New Jetty	**Complete Rebuild**	*Major cracking and spalling*	*Reconstruction of entire jetty, using modern concrete standards and materials*	*60 mm*	*50/19 (OPC+30%FA or 50%GGBS)*	*N/A*

5.3.3 Localised repair

Figure 5-11: General Corrosion damage

The two photos in Figure 5-11 show corrosion damage along a joint that had been cut out, showing the discontinuity of the steel across this support, something that the original engineers would not have desired.

5.3.4 Structural Strengthening

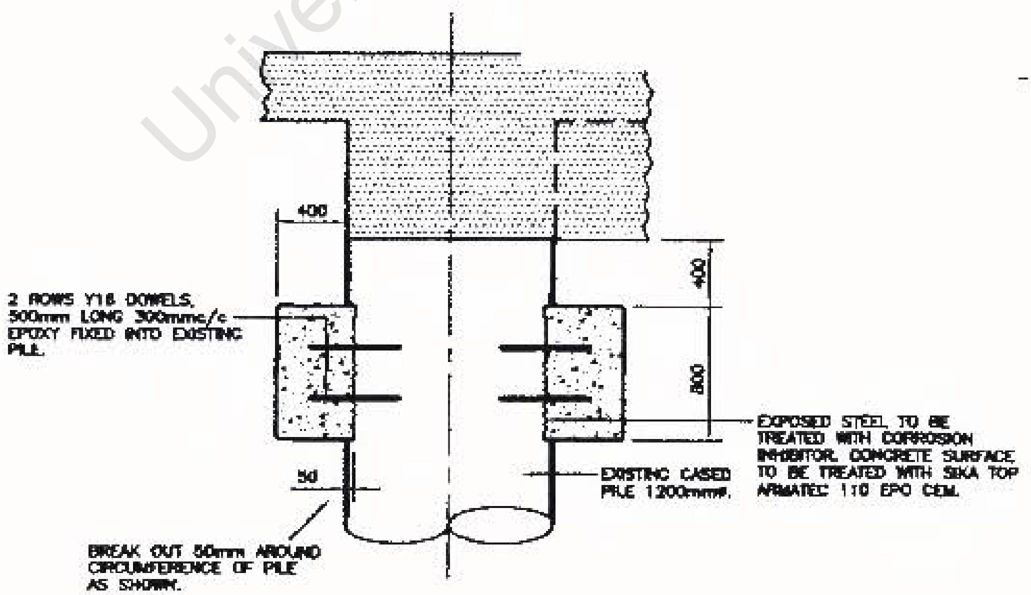

Figure 5-12: Jetty 1 Hout Bay

The above Figure 5-12 shows the rehabilitation of Jetty 1 at Hout Bay Harbour, by placing a new supporting ring beam underneath the structure. The use of surface treatments as is shown in the diagram is noteworthy.

Figure 5-13: Crack sealing and crack grouting

The use of crack sealing and grouting with mortar and/or epoxy is evident in Figure 5-13.

5.3.5 Discussion of Hout Bay Harbour

The turning bay structure on the North Mole was of particular interest because it was a structure that had reached the point of ultimate structural failure. Figure 5-14 shows the structure at a stage of partial collapse, where the central spanning slab had failed in an assumed shear mechanism. The exposed steel revealed minimal reinforcement and the minimal visible steel (assumed to be designed to minimum code requirements) was found to be in an active state of corrosion, despite showing good cover depths (>50mm).

Figure 5-14: Collapsed bay of Turning Mole, showing damaged steel reinforcement

According to the resident engineer, the structure was previously used to manoeuvre boats coming into the harbour, thereby exerting large horizontal forces on to the piled structure, which would almost certainly not have been designed to withstand this load. Thus it could be said that from a concrete material perspective the failure of the structure as a whole comes not from a concrete material or structural shortcoming, but rather from inadequate design assumptions or the change of use of the structure. The methodologies used in the repair/rehabilitation process of the structure are still of interest to the research, however.

As Figure 5-14 shows, the use of steel 'I' beam braces were necessary to maintain the stability of the structure during investigation. Initial investigations led the consultants to adopt a 'partial replace' approach to the structure, and a new central slab was designed and included in the original tender. A detailed investigation including structural analysis of the bay revealed that the best option would be to demolish the structure. This was done on site, and as yet the structure has not been replaced. Thus, it was the structural requirements that governed the repair strategy at this location. This was confirmed through discussion with the engineers involved.

Throughout the harbour, locations of repair works incorporated existing but aged reinforced concrete elements with newly-cast works. The performance of such 'composite' members from a durability perspective remains to be seen, but it is noted that the engineers in charge were willing to adopt a total replacement for the North Mole turning bay. This practice of total replacement as opposed to combining old and new structures will be discussed further in chapter 7.2, as will the use of a slightly higher cement content, as shown by the higher required strength requirement (50/19 as opposed to 45/19).

5.4 Saldanha Bay Harbour

5.4.1 Description of site

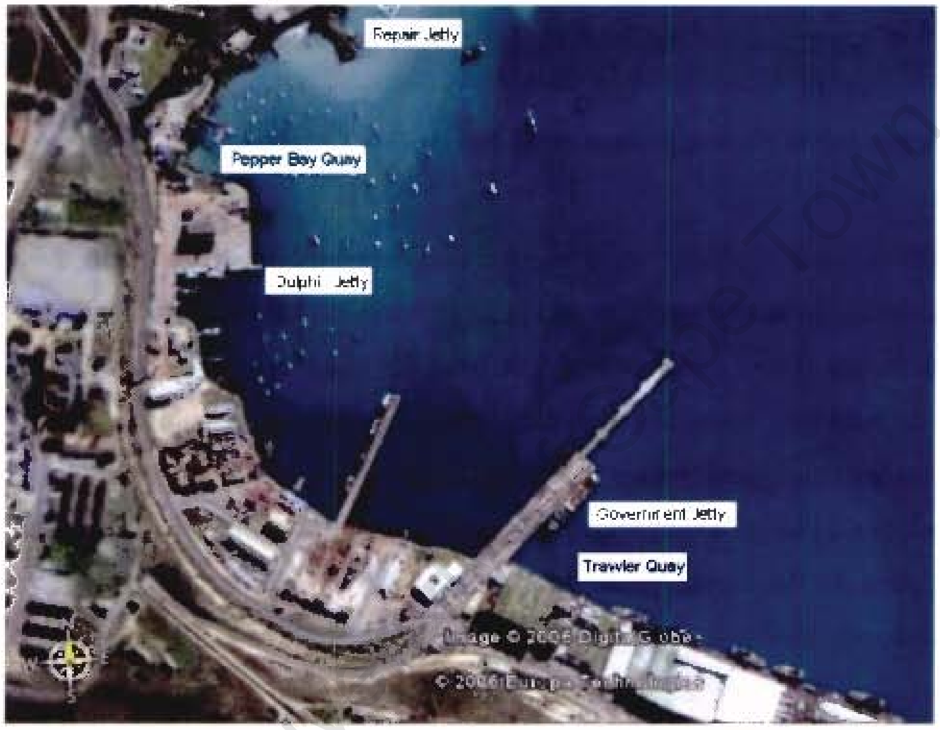

Figure 5-15: Aerial plan of Saldanha Bay (Google Earth, 2006)

Saldanha Bay harbour is a heavily utilised proclaimed harbour located approximately 100 km north of Cape Town. Of the four selected in this research, this was the port most in need of repair works, and the majority of the repair focus was on concrete repair to structures that had deteriorated due to reinforcement corrosion.

As is seen in the summary Table 5-4, the consulting engineers at this harbour adopted an approach that heavily favoured the use of surface treatments, either on their own or in conjunction with other repair works. The specified concrete mix refers the reader to the project specifications discussed in 4.1.3.2.

5.5.2 Jetty 1

One of the main focus points of the rehabilitation contract at St Helena bay was the identification of major corrosion damage at Jetty 1. Investigation of the underneath of the jetty showed severe cracking to the longitudinal and transverse beams supporting the main deck, and the decision was made to demolish the deck along with the support beams, but to retain the columns. The new design used both precast elements and in situ cast members.

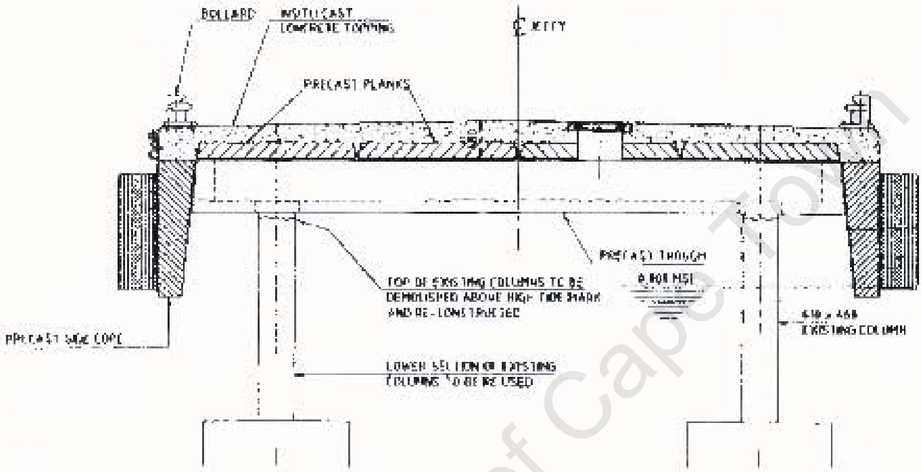

Figure 5-26: Concrete Repairs at St Helena Bay, Jetty 1

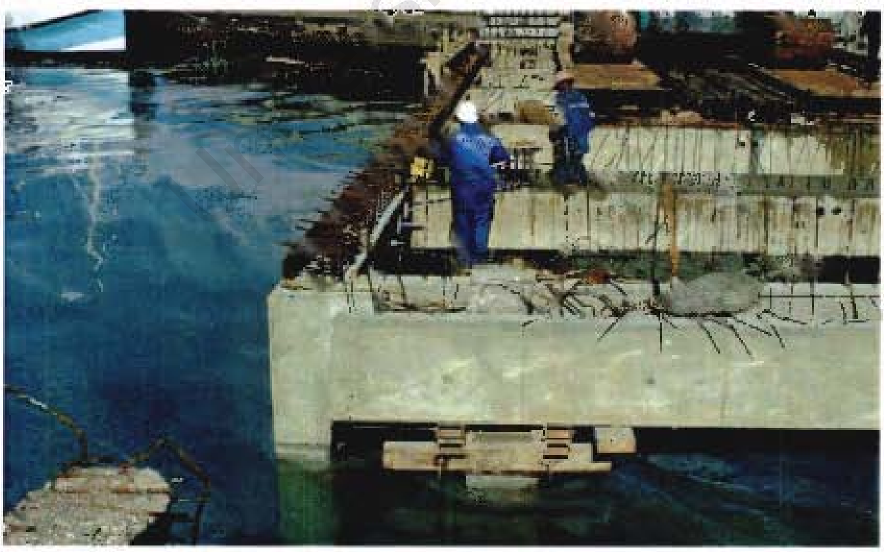

Figure 5-27: New works at Jetty 1, showing Precast Beam shuttering

Table 5-4: Summary of Repair Works: Saldanha Bay

Location	Type of Repair	Description of Damage	Description of Repair Works	Specified Cover	Concrete Mix (Strength, Aggr. Size)	Use of Specialist Repair Products
Government Jetty	**Surface Treatment**	*Major cracking and spalling*	*Penetrating corrosion inhibitor applied in conjunction with porous hydrophobic coating*	*N/A*	*N/A*	*SIKA Corrosion Inhibitor and Hydrophobic Barrier Coating*
Trawler Quay	**Surface Treatment**	*Major cracking and spalling, large deflections*	*Penetrating corrosion inhibitor applied in conjunction with porous hydrophobic coating*	*N/A*	*N/A*	*SIKA Corrosion Inhibitor and Hydrophobic Barrier Coating*
Government Jetty edge beams	**Patch Repair**	*Major cracking and spalling, mechanical damage*	*Large quantities of patch repairs performed, concrete chopped out and replaced with patch mortar, steel cleaned*	*60 mm*	*Class A or B, with 13 mm coarse aggregate*	*SIKA Monotop*
Pepper Bay	**Partial Rebuild**	*Major cracking and rust staining present*	*Piles and supporting beams retained and a new deck cast in situ*	*60 mm*	*Class A or B, with 26.5 mm coarse aggregate*	*SIKADUR Epoxy Resin, ABE Dura Grout*
Repair Jetty	**Partial Rebuild**	*Cracking and spalling to piles near cap*	*New reinforced concrete 'jackets' cast at top of piles*	*60 mm*	*Class A or B, with 26.5 mm coarse aggregate*	*SIKADUR Epoxy Resin, SIKA Topseal Coating*
Trawler Quay Columns	**Complete Rebuild**	*Total deterioration capacity loss*	*New precast columns installed and edge beam rebuilt*	*60 mm*	*Class A or B, with 26.5 mm coarse aggregate*	*SIKA Monotop*

5.4.2 Government Jetty

Government Jetty comprises a beam and slab superstructure supported on reinforced concrete piles, constructed in 1944.

Many of these piles had been damaged either due to mechanical impact or corrosion and spalling. These were to be repaired as shown in Figure 5-16, with the damaged concrete being removed and new jackets cast around the top of the piles.

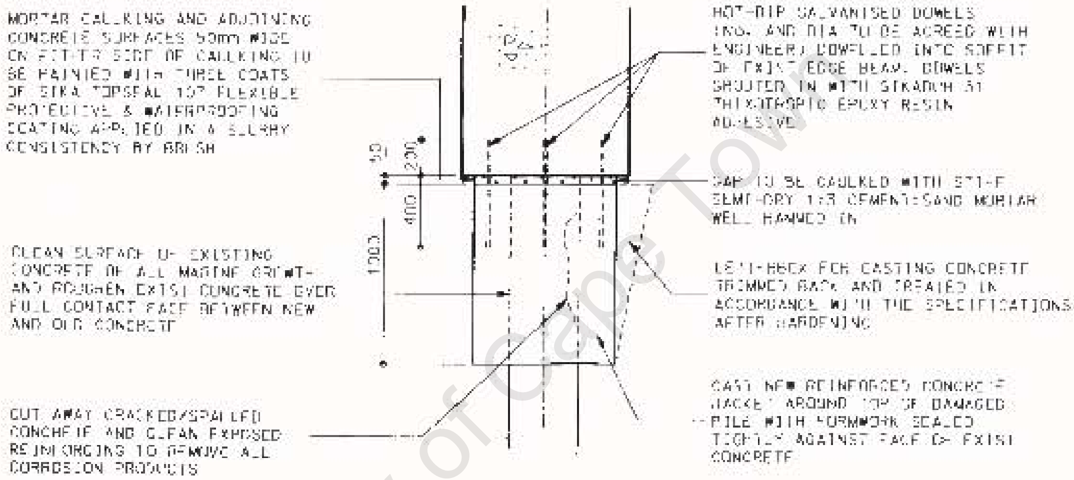

Figure 5-16: Repairs to damaged piles

There had been much mechanical damage along the edge coping beam of the jetty, and the adopted repair method involved cutting back the concrete to beyond the level of the steel, and recasting the edge beam. While the damage may have been attributed to mechanical impact of ships docking at the jetty, the removal of the surface concrete clearly showed the presence of steel corrosion. Also noticeable in the photo on the right is the presence of previously performed repair works, showing at least three different concretes in one edge beam.

Figure 5-17: Previously performed patch repairs

Figure 5-18: Edge beam of Government Jetty

Figure 5-18 and the following diagram (Figure 5-19) show the superstructure of the Government jetty, to which surface treatment of barrier coatings and penetrating corrosion inhibitors was applied. This repair method will be given more focus at a later stage in the thesis, where the forensic work leading to this decision will be analysed. This treatment was also applied beneath the jetty, and much care had to be taken in the application of the treatments on account of two considerations. Firstly, some of the application was within the tidal range and needed a specified dry time for effective use, and secondly the jetty was to remain in service for the duration of the contract, and the application of the treatments required the temporary removal of the furniture fixed to the coping beams.

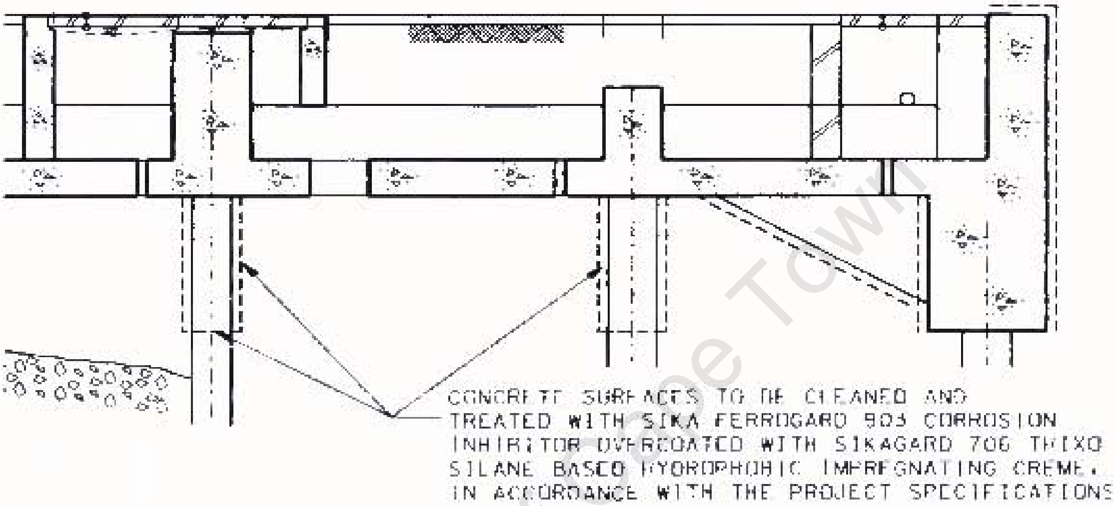

Figure 5-19: Repair details to Government Jetty substructure

5.4.3 Trawler Quay (Phase 1)

The Trawler quay was constructed in approximately 1965, and the main damaged area at the commencement of the repair contract was the supporting reinforced concrete columns. Upon investigation it was found that three of the fourteen columns had failed completely, with a number of other piers showing similar advanced levels of damage. A temporary measure was instituted to prop up the edge of the quay with two 203x203x52 steel H-sections. Tests carried out on the quay superstructure showed the deck elements to be of a reasonably sound structural condition, and the decision was made to rehabilitate the columns and the edge beam only.

A 'strong-back' steel girder was used as a temporary stay to prevent the deck collapsing completely while the columns where being demolished. The deck was then jacked to its correct line and level before new columns were cast. A new edge beam was cast upon completion of the columns.

Figure 5-20: Severe damage to Trawler Quay beams and columns

Figure 5-21 shows the prescribed surface treatments to the Trawler Quay:

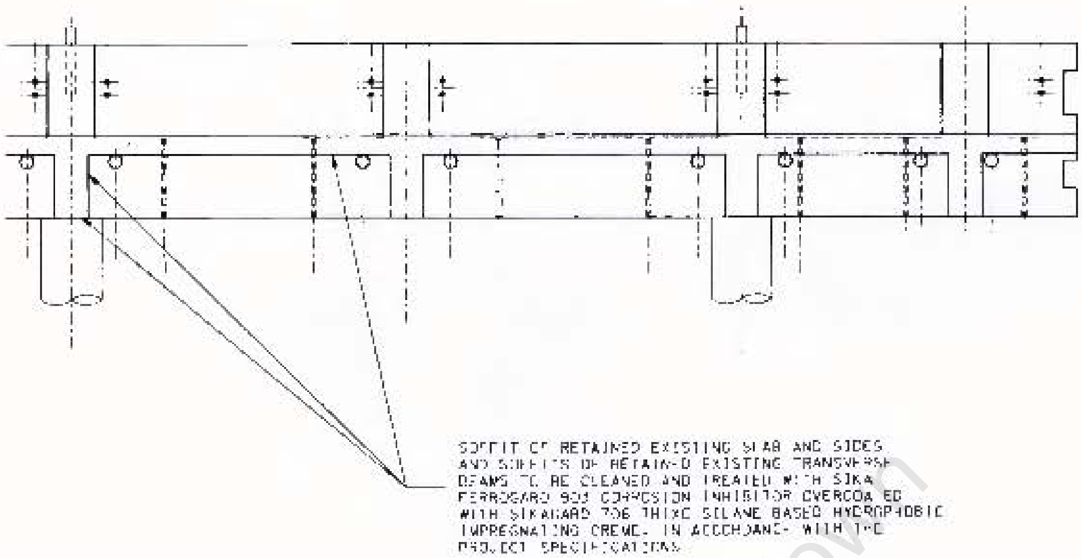

Figure 5-21: Trawler beam surface treatments

As with the Government Jetty, the use of a penetrating corrosion inhibitor in conjunction with a hydrophobic surface treatment was adopted for treatment to the underneath of the superstructure.

Figure 5-22: Trawler Quay showing 'strongback' girder

The photo above shows clearly the deflection that had occurred along the edge of the Trawler Quay, and the strong-back girder that was being used for temporary support. Also noticeable is the presence of previous repair measures.

5.4.4 Trawler Quay (Phase 2)

Phase 2 of the Trawler key rehabilitation involved the repair of the edge beam located further along the deck, which had been previously damaged due to mechanical impact from ships.

5.4.5 Repair Jetty

The Repair Jetty is a beam and slab superstructure supported on reinforced concrete piles, and it is likely that its original construction date was sometime between 1960 and 1970. At the top of the piles there was longitudinal cracking evident, and according to the contract is likely that the damage has been caused by reinforcement corrosion. Rehabilitation required the removal of cracked and spalled concrete and the casting of a new jacket around the top of the piles, as is shown in Figure 5-23.

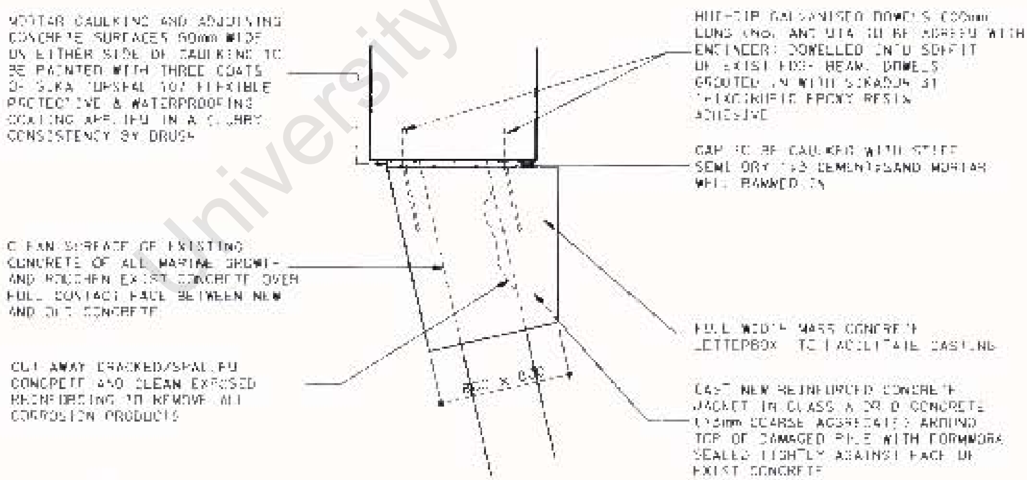

Figure 5-23: Cracked pile repair detail

5.4.6 Pepper Bay Concrete Quay

The Pepper Bay Concrete Quay is a reinforced concrete structure comprising a system of in-situ cast beams supporting a deck made up of precast planks, all supported on concrete piles. It is estimated that it was constructed in the late 1960's. Severe spalling had occurred on the precast planks and the decision was made that the deck would be replaced in its entirety, due to the impracticality of repairing the spalled concrete.

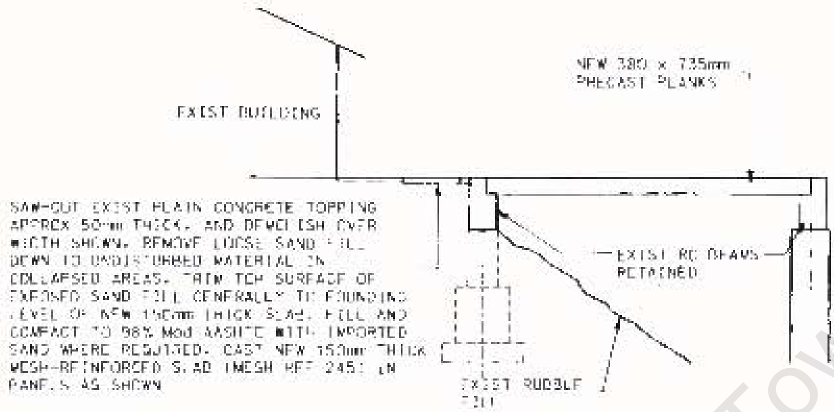

Figure 5-24: Pepper Bay repair details

5.5 St Helena Bay Harbour

5.5.1 Description of site

Figure 5-25: Aerial View of St Helena Bay Harbour (Google Earth, 2006)

St Helena Bay harbour comprises a number of jetties and quays, some of which have had previous repair works performed upon them. The repair works, shown in summary form in Table 5-5, range from 'do nothing' to complete reconstruction. The approach at this location, as gleaned from the table seems not to rely on repair products, but rather on a more conservative approach regarding concrete cover. This conservatism is again noted with the reconstruction of beam elements, as opposed to rehabilitation. In contrast to this is the use of a relatively low concrete strength requirement of 40 MPa. This is discussed and analysed with the other harbour sites in chapter 7.

Table 5-5: Summary of Repair Works: St Helena Bay

Location	Type of Repair	Description of Damage	Description of Repair Works	Specified Cover	Concrete Mix (Strength, Aggr. Size)	Use of Specialist Repair Products
Jetty 3 and 4	**Do Nothing**	*Minor cracking, rust staining present*	*Initially disregarded as too minor for repair*	*N/A*	*N/A*	*N/A*
Guniting – Jetty 2	**Patch Repair**	*Major cracking and delamination, rust staining*	*Old contaminated concrete chopped out and replaced with a gunited concrete*	*75 mm*	*40/19 (OPC+30%FA or 50%GGBS)*	*SIKA 212 Grout*
Quay 3	**Partial Rebuild**	*Major Rust staining, high chloride levels*	*Beams replaced with new prestressed precast beams*	*60 mm*	*40/19 (OPC+30%FA or 50%GGBS)*	*SIKA 212 Grout*
Deck of Jetty 1	**Complete Rebuild**	*Minor cracking, moderate chloride levels*	*Old deck condemned and demolished, new deck constructed*	*75 mm*	*40/19 (OPC+30%FA or 50%GGBS)*	*None*
Central Span Beam Jetty 1	**Complete Rebuild**	*Major cracking and delamination, severe staining*	*Beams chopped out and replaced with new beams cast in precast shutters*	*75 mm*	*40/19 (OPC+30%FA or 50%GGBS)*	*None*

Figure 5-28: Failure of previously performed repairs at Jetty 1, St Helena Bay

Previously performed repair works were also identified, and the failure of the one of the patch repair locations is visible in Figure 5-28. The reinforcing steel has again become exposed due to the spalling of the patched concrete.

5.5.3 Jetty 2

Figure 5-29: Typical reinforcement at St Helena Bay Jetty 2

Jetty 2 is a lead-in jetty for the main slipway at St Helena Bay. It comprises a deck supported on transverse and longitudinal beams, which are in turn supported on concrete piles. The reinforcement in the piles and beams are Rolled Steel Joist beams, and in the beams especially, major spalling had occurred exposing the bottom flange of these beams. The method adopted in the repair of these elements was to the remove the damaged concrete and shotblast the corroded steel to rid it of corrosion products. An anti-corrosive coating was then applied to the steel before the member was gunited to a minimum cover of 75 mm, as is shown in Figure 5-30 below.

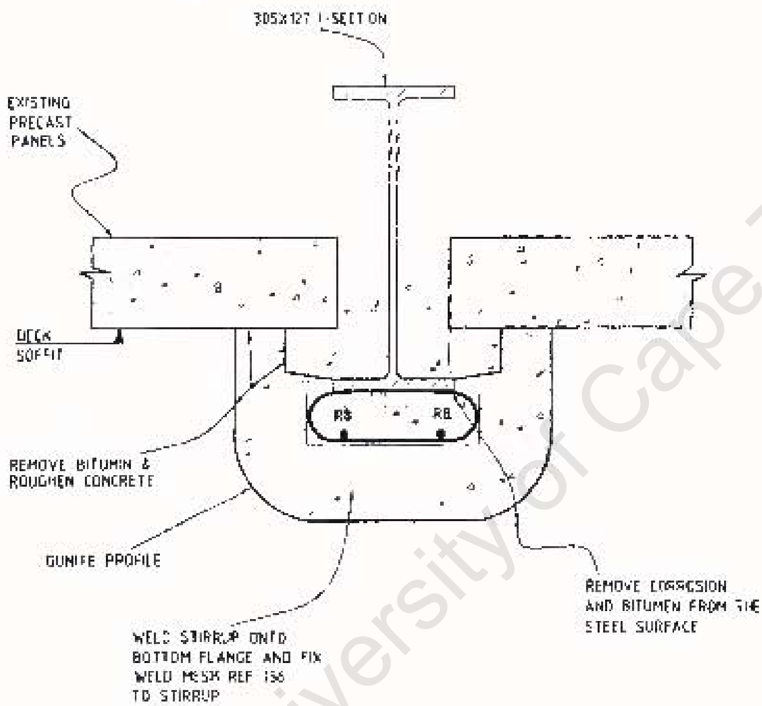

Figure 5-30: Guniting detail for Jetty 2, St Helena Bay

It should be noted that previous repair measures were also present in these elements. Details of these are shown in the following drawing, noting the date (1977), and the similarity between this method and the current method of repair.

The major difference is the level of cover provided, and it is likely that the failure of the previous method of repair contributed to this increased cover specification.

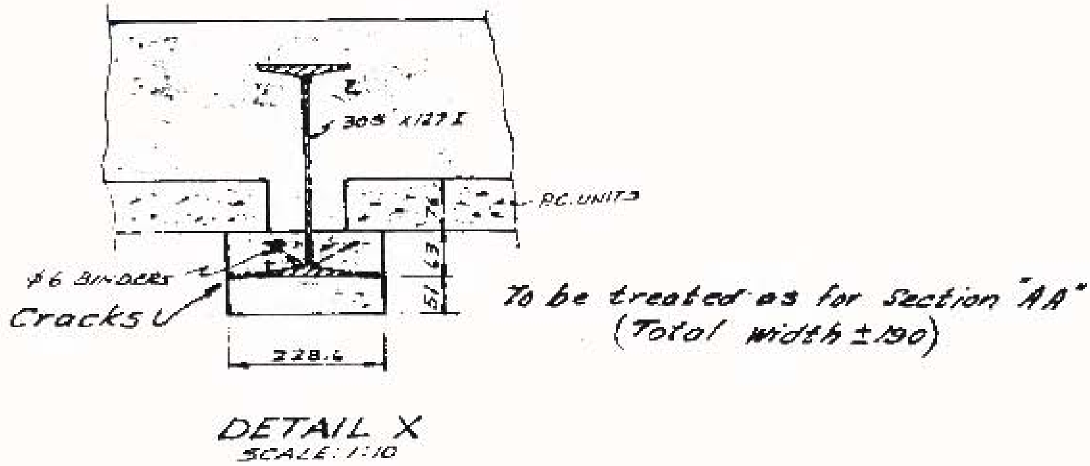

Figure 5-31: Details of previously performed repair methods to Jetty 2

Major repairs were being performed along the surface of the deck, ranging from areas such as shown below where breaking out and recasting is required to crack injection and sealing.

Figure 5-32: New reinforcement to Jetty 2 at St Helena bay

5.5.4 Quay 3

The deck of the quay was found to be in very poor condition with most of the post-tensioned transverse beams having spalled so severely that the structure had been condemned. The piles and the pile cap beam, however, were in good condition and were retained for the new construction. Shown in Figure 5-33 are the new prestressed transverse beams sitting on the original pile cap beam.

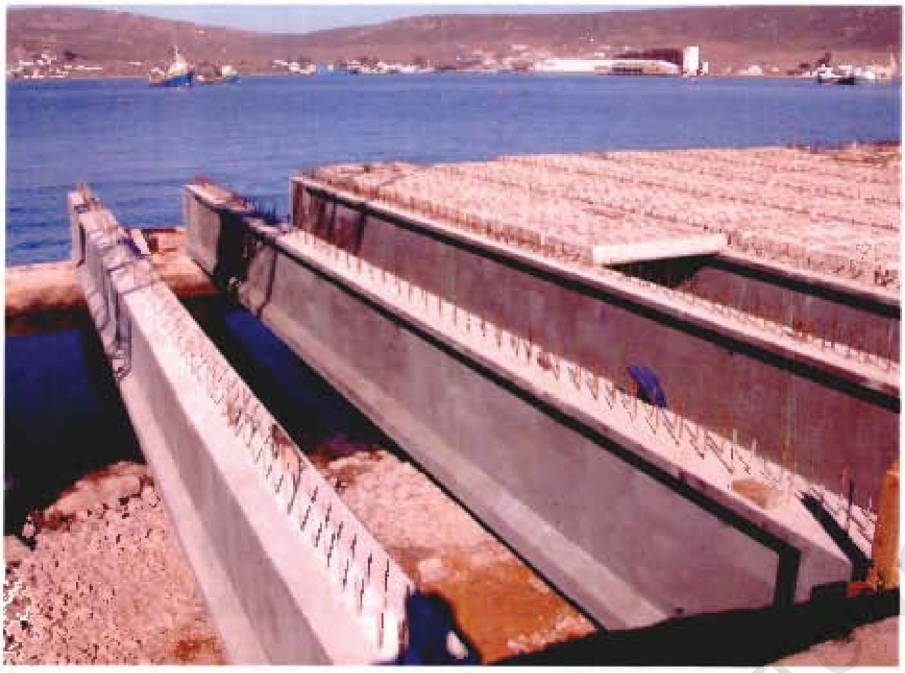

Figure 5-33: New transverse beams for Quay 3, St Helena Bay

5.5.5 Jetty 3 and 4

At the commencement of the contract, Jetty 3 and 4 did not show enough damage to warrant inclusion in the original repair works. However, as the contract progressed it was seen that there could be funds available for certain remedial measures to be applied to the locations, and testing was performed at UCT to ascertain the appropriateness of such measures (7.2.4).

6 Forensic Testing

6.1 Introduction

During the course of this research, the Concrete Materials Research Group (CMRG) at the University of Cape Town (UCT) was contracted to perform certain forensic testing at two of the harbours involved in the RAMP programme. The two tests used in particular were chloride profiling and penetrating corrosion inhibitor testing, both described in chapter 3. Presented in this chapter are the results of these tests. The discussion and critical evaluation of these results is contained in chapter 7.

6.2 Saldanha Bay Harbour

6.2.1 Government Jetty Chloride profiling results

Two locations were investigated in detail at the Saldanha Bay Harbour, namely the Government Jetty and Trawler Quay. The CMRG at UCT was contracted to analyse twenty-four cores that were extracted from these structures in order to assess chloride ingress into the concrete. Both structures had been showing severe signs of corrosion damage, and the investigation was to inform appropriate remedial recommendations (UCT, 2002).

The locations of the cores were as follows:

Government Jetty	
Slab soffit (including two samples through reinforcing)	6 No.*(1-6)*
Beam side above high water mark	2 No.*(7-8)*
Beam side below high water mark	2 No.*(9-10)*
Column above high water mark	2 No.*(11-12)*
Column below high water mark	2 No.*(13-14)*
Trawler Quay	
Beam side above high water mark	3 No.*(15-17)*
Beam side below high water mark	3 No.*(18-20)*
Slab soffit	4 No.*(21-24)*

Using the chloride profiling technique discussed in chapter 3, chloride profile analysis was performed on the cores; in some instances cores were combined to produce results. The results are shown in Figure 6-1and Figure 6-2:

Table 6-1: Summary of results for chloride contents at level of reinforcing steel

Government Jetty

Location	Cover Depth (mm)	Cl Level (% mass cem)	Comments	
1	(40-65); 30 & 45 from co	> 1.5	At level of 30 mm steel.	
			Substantial rusting and pitting of steel	
2	(20-60), 50	> 2.0	At level of 50 mm steel. (No Staining?)	
3/4	(40-65), 45	> 2.0	At level of 45 mm steel.	
5/6	(40-60), 50	> 1.5	At level of 50 mm steel.	
7	50 assumed	> 1.0	Above highwater mark	
8	50 assumed	> 3.0	Below highwater mark	
9	(50-65), 50	> 1.0	At level of 50 mm steel	
10	(50-65), 50	> 3.0	At level of 50 mm steel	
11	(55-60)	= 1.0	Above highwater mark	
12	(55-60)	> 2.0	Below highwater mark	
13	(55-60)	= 0.5	Above highwater mark	
14	(55-60)	> 2.0	Below highwater mark	

Trawler Quay

Location	Cover Depth (mm)	Cl Level (% mass cem)	Comments	
15	50	> 0.2	Above highwater mark	
16	50	> 0.5	Below highwater mark	
17/19	(55-65), 60	< 0.2	Above highwater mark	
18/20	(55-65). 60	< 1.0	Below highwater mark	
21/22		<< 0.5	Depth assumed to be greater	
23/24		< 0.5	than 80 mm.	

(NOTE: Cover depth given in second column in brackets is an approximation)

The summary of the chloride results is given below:

Government Jetty

Slab soffit. *In excess of 1,5% (by mass of cement). Relatively high chloride level, confirms damage observed on site.*

Beams above high-water mark. *In excess of 1% but not greater than 1,5%, not yet considered excessive.*

Beams below high water mark. *In excess of 3%. Very high chloride level, as expected in permanently saturated elements.*

Columns above high-water mark. *Between 0,5 and 1%, moderate.*

Columns below high-water mark. *In excess of 2%.*

The authors noted that the high chloride levels in the slab had led to major corrosion damage, despite the lack of major visible damage in the most contaminated areas. This was because the chloride levels above the high water mark were not yet at

excessively high levels, while below the high water mark, the permanently saturated nature of the concrete had resulted in little damage.
The authors also noted that the column elements were of a higher quality than the beam elements.

Trawler Quay

Beam elements above high water mark. *Chloride levels generally 0,2%. This is a low chloride content, below the generally accepted threshold level of 0,4%.*
Beams below high-water mark. *Chloride levels between 0,5 and 1%. Moderate.*
Slab soffits. *Chloride contents generally less than 0,5%, which also indicated a relatively passive steel condition.*

These results indicated that the beam quality was higher than that of the slabs and suggested the possibility of the use of different binders, such as Sorel Cement.

While the permanently saturated elements showed high chloride values, they were not deemed to be at a great risk of corrosion because of the starvation of oxygen at the steel. For the elements that were partially saturated, particularly those located within the tidal zone, the following categorization was devised for recommendations for repairs (UCT, 2002):

Cl level	
< 0,5%	*Structure to be protected from further chloride ingress in non-permanently saturated areas.*
0,5-1%	*Structure to be protected from further chloride ingress; also, use of a corrosion inhibitor to be considered.*
1-1,5%	*Structure to be protected from further chloride ingress; also, use of a corrosion inhibitor to be considered, possibly at a higher dosage rate.*
> 1,5%	*Conventional remediation measures such as patch repairs or application of corrosion inhibitors are unlikely to be successful. Cathodic protection (impressed current or sacrificial anode types) is the only proven method of halting corrosion in such areas.*

The use of a penetrating Corrosion Inhibitor (CI) was suggested in order to slow the rates of corrosion at the level of the steel, but in conjunction with a surface treatment to promote the migration of the inhibitor as well as to reduce further chloride ingress.

It was recommended that field trials be implemented on site before any full scale work was attempted.

6.2.2 Corrosion Inhibitor field trials

In order to gain some assurance of what combination of surface treatment and corrosion inhibitor (CI) would be most suitable, three different combinations were applied to sections of the Government jetty columns and beams as follows:

- A: CI only
- B: CI plus a surface coating(barrier coating to physically prevent penetration)
- C: CI plus a silane coating (hydrophobic coating allowing gaseous penetration but limiting moisture ingress)

The columns and beams were cored a month after application and the penetration levels of the CI were tested. The surface treatments were also inspected.

Seventeen cores in total were extracted, and Figure 6-3 shows the locations of each:

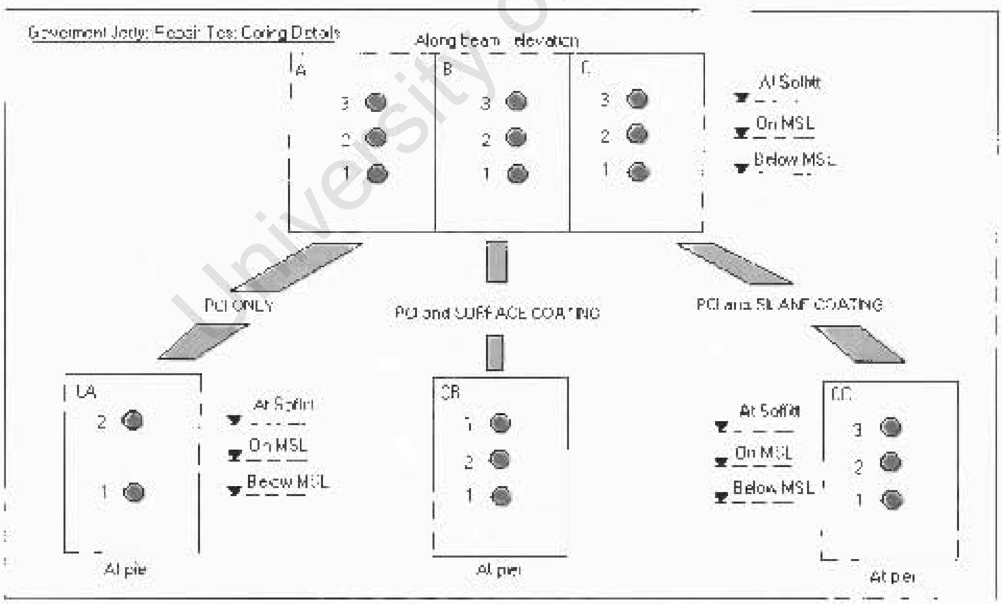

Figure 6-3: Locations of Cores extracted from Government Jetty for Inhibitor Testing

The following observations were made of the cores and locations.

- The cores were 52 mm in diameter, ranging in length from 125 – 150 mm.
- Large aggregate sizes present. 50 mm and larger.
- Granite coarse aggregate used, some weathered.
- Isolated presences of Alkali-Silica reactions visible.
- Good compaction in general, few voids present.
- Some cores showed steel at 40-60 mm cover.
- Organic deposits/growth still present on many surfaces, e.g. barnacle traces.
- Samples taken from below sea level had characteristic darker first 5 mm.
- Surfaces of the cores visually not impacted by the application of the treatments, except for grey Surface Barrier Coating. On these cores (the 'B' set) it was possible to remove the coating easily; appeared to be as a result of the presence of the organic residue. Residue could be removed with better surface preparation.

6.2.2.1 Test Results:

Figure 6-4 below shows the results of the corrosion inhibitor combination testing, on beam and column elements. The darker the colour, the stronger the presence of inhibitor at that depth (shown on left).

Test results: Presence of Ferrogard at core depths

		Beam			Column		Beam			Column			Beam			Column		
	Core Depth	A1	A2	A3	CA	CA2	B1	B2	B3	CB1	CB2	CB3	C1	C2	C3	CC1	CC2	CC3
	0-10	N	N	N	N	N	LTR	N	N	N	N	N/A	N	N	TR	LTR	TR	TR
	10-20	TR	N	TR	N	LTR	N	N	N	N	N	LTR	N	N	LTR	N	TR	G
	20-30	TR	N	N	N	TR	N	N	N	N	TR	TR	N	N	TR	N	GTR	G
	30-40	TR	N	TR	TR	TR	N	N	N	LTR	TR	GTR	N	N	TR	N	G	G
Cover Range	40-50	TR	LTR	TR	TR	G	LTR	N	LTR	TR	GTR	GTR	TR	LTR	G	N	G	G
	50-60	TR	TR	TR	TR	TR	LTR	LTR	LTR	GTR	TR	LTR	LTR	TR	G	TR	GTR	G
	60-70	TR	TR	TR	N	TR	LTR	LTR	TR	TR	TR	TR	LTR	GTR	G	TR	G	G
		PCI only					PCI and Barrier						PCI and Silane					

Interpretation		
N	No colour	Inadequate
LTR	Light Trace	
TR	Trace	Adequate
GTR	Good Trace	
G	Good	

X3	Top @ Soffit – Beam
X2	Approx MSL - Beam
X1	Below MSL - Beam
CX3	Top @ Soffit - Column
CX2	Approx MSL - Column
CX1	Below MSL - Column

Figure 6-4: Results of Inhibitor Testing

Notable results are discussed briefly in bullet form below:

- The CI penetrated to an adequate level (70 mm plus). The majority of the samples showed at least a trace CI presence.
- In terms of the quantity of CI that is found in the concrete, in general the cores showed fairly low concentrations of CI present. This can be seen in the majority of the results showing 'trace' levels of the CI. The fact that this is not a quantitative test as such must be kept in mind, as the test is designed to show the level of penetration.
- The best results were obtained using a coating of Silane in combination with a CI. The Silane appears to act well as a hydrophobic barrier. This barrier prevents water from moving into the material and diluting the CI out of the concrete, which can be seen in the increased presence of CI in the 'C' samples.
- The first 20-30 mm of concrete typically showed no CI presence. The reason for this is the dilution and extraction of CI due to the constant water action on the surface of the concrete.
- The best performing location was above MSL at the soffit, especially for column elements as opposed to beam elements, and this was expected because it has relatively little direct contact with water. The worst location was on the mean sea level where there is constant sea action against the concrete.
- In general the columns appear to have performed better than the beams. This could be ascribed to a difference in quality of concrete between the two, noting that while a lesser-grade concrete offers less resistance to the inhibitor ingress, a higher grade concrete will be able to hold more of the inhibitor and thus be more resistant to the surface dilution effect.
- The barrier coating, despite retaining its bond with the surface of the concrete adequately, had little if any positive effect on the performance of the inhibitor in terms of ingress and retention.

Other comments:

- The concrete contains large aggregates, and it is possible that these aggregates provide a barrier to penetration, increasing the distance that the CI would have to travel to get to the level of the reinforcing steel. This also impacted the test procedure in that the cores that were drilled have a lower cement paste content, because of the large aggregate sizes.

- Organic deposits (barnacles etc.) were found on the surface, even beneath the barrier coating. This indicates poor surface preparation prior to the application of the treatments. The effect of this on the migration of the inhibitor is unknown, but it was seen that it directly affected the ability of the barrier coating to bond to the surface of the concrete.
- The applications were applied with only a few hours before the surfaces were in contact with the rising tide. The 'drying' time of this treatment is stated as 6 hours, with a time of 5 hours required between each coating. In terms of overcoating procedure, the CI coat should be allowed to dry for 3 days before a surface overcoat is applied. The fact that these requirements are very difficult to meet in harbour structures has negative impacts on the performance of the penetration of the inhibitor.

The report showed that the CI was most successful when used in conjunction with the Silane surface coating. It also showed that it was possible, with the correct surface preparation, to achieve a penetration of up to 70 mm into the concrete. Noted was the present of CI at the most vulnerable areas to corrosion, that being above the MSL (UCT, 2003).

6.2.2.2 Recommendations

The report produced from these findings made the recommendation of full scale implementation of a CI/Silane system to inhibit reinforcement corrosion. The trial testing had shown that this combination was likely to produce the best results. It was noted that the accurate prediction of service life for the structure was difficult, and thus an estimate of a service life extension of between 5 and 10 years was given, at which point more serious investigations and repairs of a more permanent nature would be necessary.

Because of the possibility of problems with surface preparation and application of such materials, a recommendation was also made for the manufacturers of the products to produce a Method Statement. The Method Statement was to address the most appropriate manner in which to apply the materials for the particular location.

6.2.2.3 Retest

Remnants of four of the original cores were retested three months later to give confidence in the original results – Results are shown in Figure 6-5:

Retest

	New	Orig	New	Orig	New	Orig	New	Orig
Core Depth	CA2	CA2	B3	B3	C3	C3	CC3	CC3
0-10	N	N	N	N	LTR	TR	G	TR
10-20	N	LTR	LTR	N	TR	LTR	G	G
20-30	LTR	TR	LTR	N	TR	TR	G	G
30-40	TR	TR	TR	N	LTR	TR	G	G
40-50	GTR	G	TR	LTR	TR	G	G	G
50-60	GTR	TR	TR	LTR	GTR	G	G	G

N	No colour
LTR	Light Trace
TR	Trace
GTR	Good Trace
G	Good

Figure 6-5: Retest results for Inhibitor Testing, comparing Original and New Test Results

CA2: On column, at MSL, CI application only

B3: On beam, above MSL, CI and Surface Coating application

C3: On beam, above MSL, CI and Silane Coating

CC3: On column, above MSL, CI and Silane Coating

The above figure shows the comparison between the results of the retest and the original set. The majority of the results confirm the original findings, with the CI/Silane combination again providing the best performance. The re-test of core B3 showed a better result for the CI/Surface Coating, but this is still inferior to the performance of core CC3.

6.2.3 Trawler Quay: Chloride profiling of vertical cores through deck

100 mm cores were extracted vertically through the 500 mm Trawler Quay deck. These cores were tested using chloride analysis to produce a chloride profile. It was possible to see the chloride level (expressed as a percentage of binder) from both the 'top' (deck surface) and the 'bottom' (deck soffit).

As this deck is in such close proximity to the sea, the results for the chloride levels were expected to be high (See Figure 6-6). These are compared to the results obtained for the deck cores extracted from St Helena bay in the following chapter.

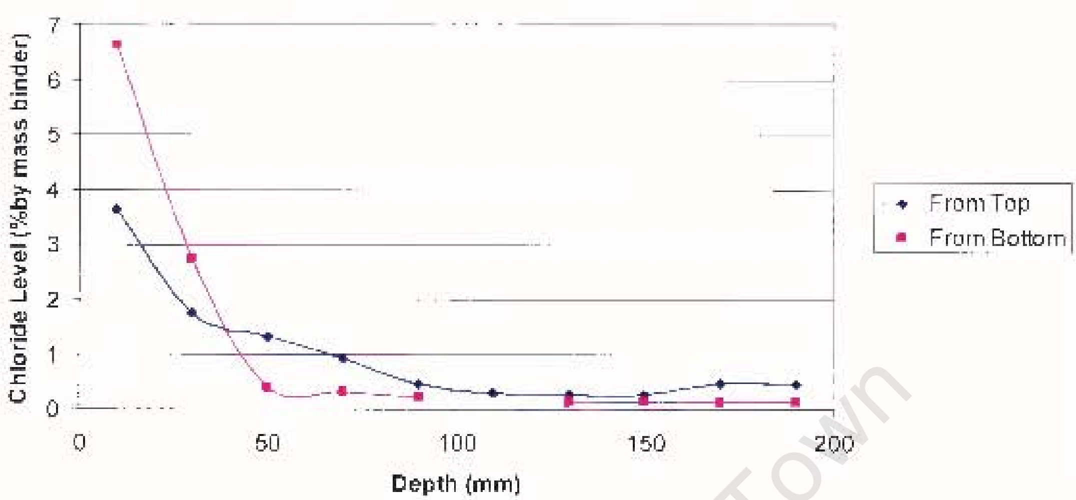

Figure 6-6: Average chloride profiles for Trawler Quay Vertical cores

The results show a typical chloride profile, with a high surface concentration value (in the order of 5-8% by mass of binder each side). Beyond a depth of 100 mm into the concrete, the chloride levels drop below the 0.4% 'threshold.

It is interesting to note that the core from the bottom has a higher surface chloride value but quickly reduces to a lower value. The core that was extracted from the top, however, shows a lower initial value but the rate at which this value decreases into the concrete is less. This would be consistent with the physical arrangement, with the lower face being in more regular contract with seawater, whereas gravity would encourage the transport of the chlorides from top to bottom but hinder the movement from bottom to top. There is also the possibility of the top surface chlorides being 'flushed out' by rain.

6.2.4 Trawler Quay: Chloride profiling of horizontal cores into wall

Three 100 mm diameter cores were extracted from the Trawler Quay edge beam and tested for chlorides. These cores were drilled horizontally, core 2 from the seaward side whilst cores 1 and 3 were extracted from the sheltered side.

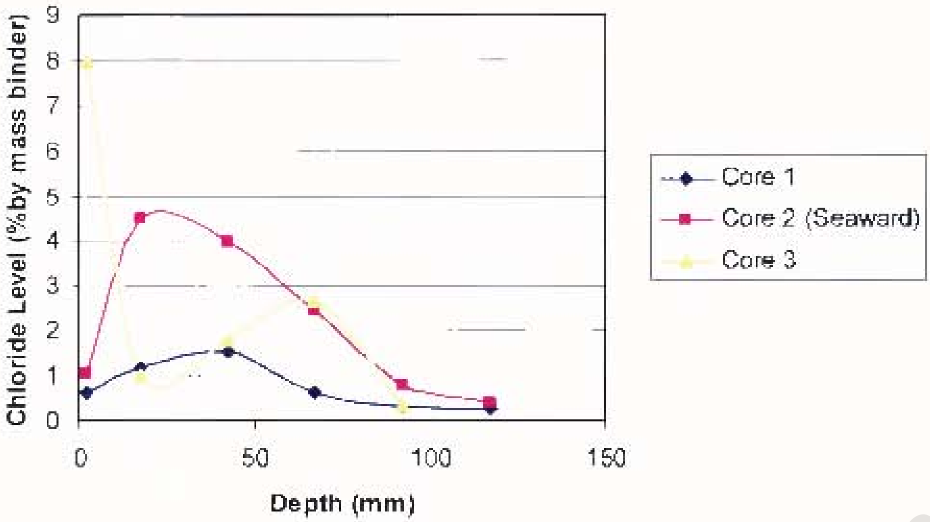

Figure 6-7: Chloride Profile for Trawler Quay Horizontal Cores

Whilst the results show considerable scatter, there are some points that can be made from the profiles. There is definite 'leaching' present, accounting for the low surface values of cores 1 and 2. Also notable is the high level of chlorides (>2%) at considerable depth, reaching up to 80 mm and beyond. The beams/wall from which these cores were extracted showed high levels of corrosion damage (cracking and spalling) and were subsequently demolished and replaced.

In the case of core 3, Figure 6-8 shows evidence that patch repairs had been performed on the vertical wall elements. The difference in the two concretes is striking: with the older concrete (Core 1) using a smaller stone whereas the newer patch repair mix (Core 3) uses a much larger stone. Whilst this mix may achieve its desired properties in terms of shrinkage, the transport properties of chlorides through the material will be very different as a result of the different microstructure and this has direct implications for corrosion.

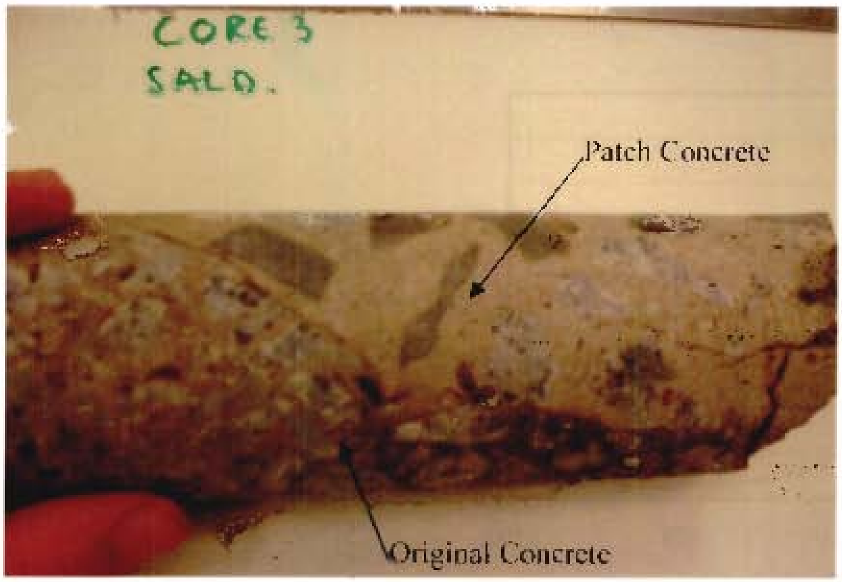

Figure 6-8: Core through Trawler Quay showing original concrete (below left) and patch repair (above right).

6.3 St Helena Bay Harbour

6.3.1 Jetty 1: Chloride profiling of beams

Jetty 1 showed major signs of reinforcement corrosion damage and it was decided to investigate further the level to which chlorides had moved into the structure.

Drilled powder samples were taken from eleven transverse beams underneath the deck. At each beam, four locations were drilled to ensure a large enough sample. The drilled powder was collected in 20 mm increments to a depth of 60 mm. Testing was not done at beams where corrosion staining was present, assuming that these beams were already beyond repair. Contact was made with reinforcement at one location at a depth of approximately 50 mm. The longitudinal beams that crossed the eleven transverse beams showed signs of major corrosion with large scale cracking and spalling present, and had been condemned.

Chloride contents were determined and the results are displayed below in Figure 6-9.

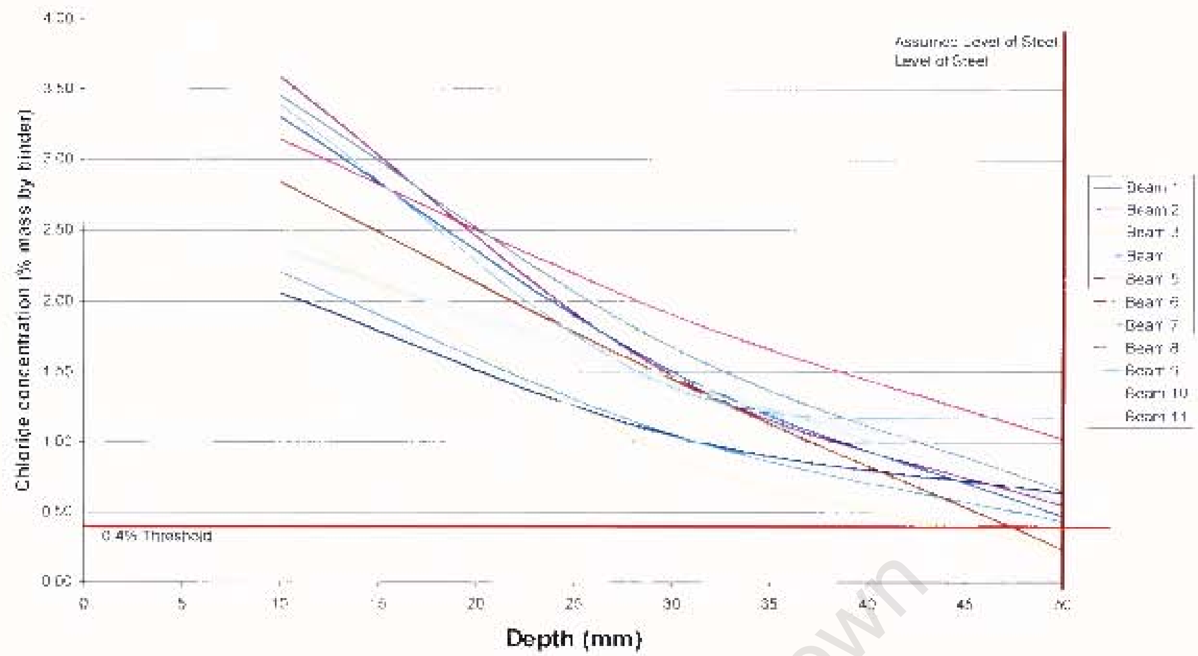

Figure 6-9: 11 Chloride profiles for St Helena Jetty 1

By approximating the curves in Figure 6-9 to a Ficks 2[nd] law model based on the diffusion properties of the concrete, it is possible to ascertain the depths of an assumed 0.4% threshold for each beam. These are represented in Figure 6-10:

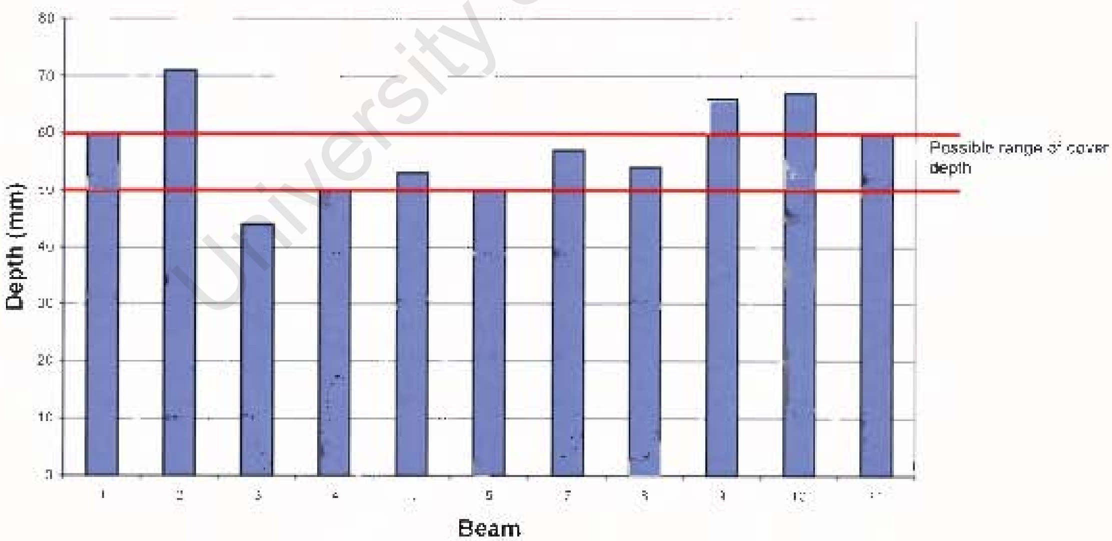

Figure 6-10: Depths of the 0.4% threshold at which corrosion risk becomes imminent

This figure shows that at present, the threshold value at which corrosion risk becomes imminent (0.4% chlorides by mass of cement) is in many instances (2, 9, 10, 11) at a depth that is greater than the suspected range of cover (50-60 mm). The possibility of corrosion being active in many of these beams is therefore high.

6.3.1.1 Conclusions

The results of the chloride profile testing showed that there is a high chloride presence in these beams. Using chloride profiling techniques it can be seen that the depth of the 0.4% threshold is well into the estimated level of cover in many beams, in fact all bar one. This indicates that corrosion is likely to be active and the beams are being steadily deteriorated. As the structure is 32 years of age, it would be expected to be corroding under such a harsh marine environment.

6.3.2 Jetty 1: Chloride profiling of deck

The demolition of the deck was already part of the works programme, but it was decided for the sake of confirmation that two cores were to be extracted from the deck. These cores were drilled vertically through the 400 mm thick deck and chloride analysis was performed on slices extracted from the cores in 20 mm increments. Results are shown as follows:

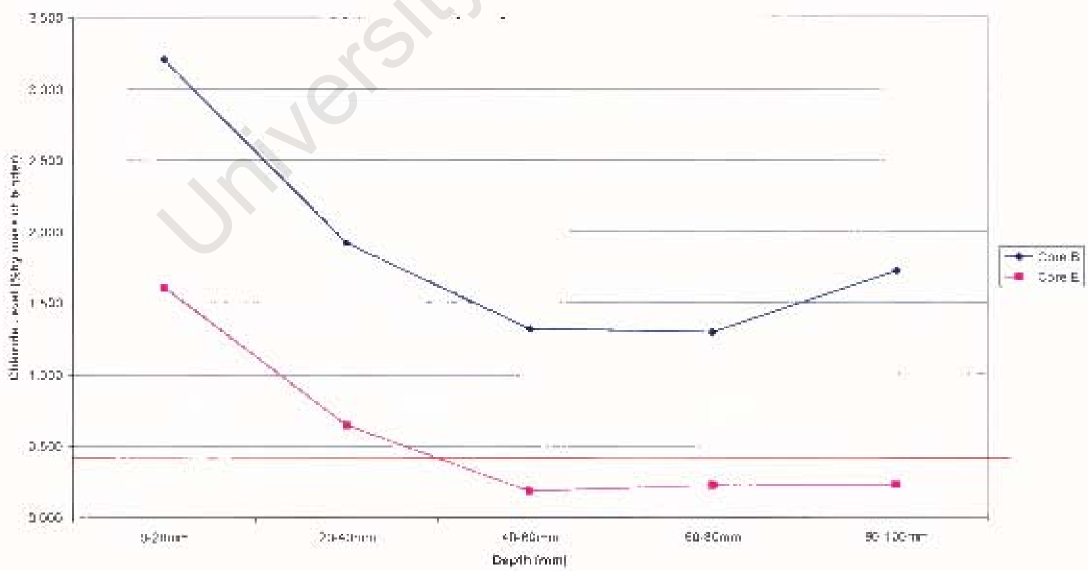

Figure 6-11: Chloride profiles for Jetty 1 from Soffit

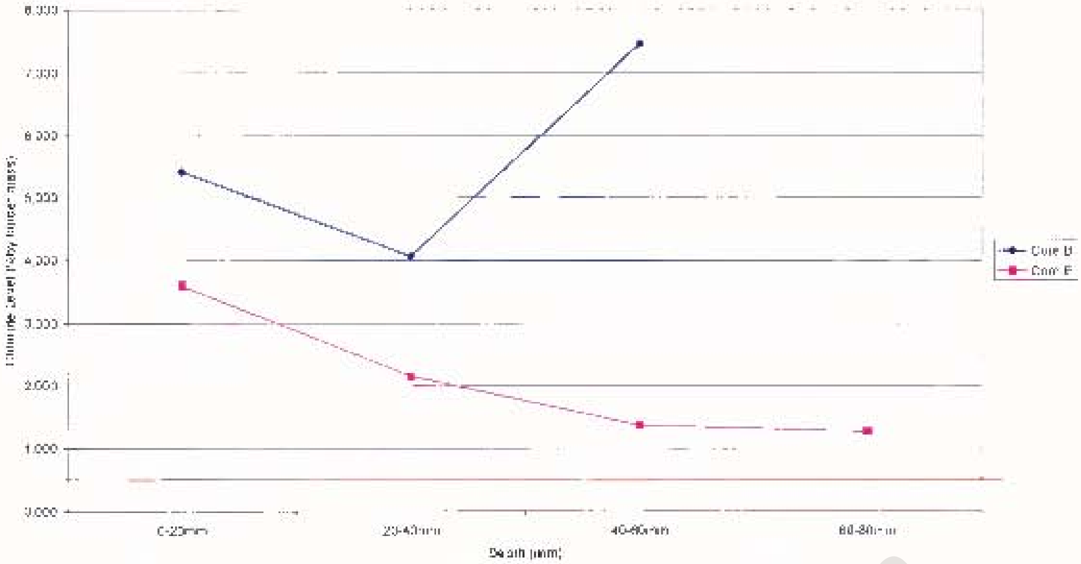

Figure 6-12: Chloride profiles for Jetty 1 from Top of Deck

The results for Core E are in both cases (top and bottom profiles) lower than for Core B. This could be as a result of a difference in location (Core E was extracted from a more sheltered location than Core B). The key result to notice, however, is that the chloride values are very high at great depth into the deck. It is clear that even in the case of high cover to reinforcement (>100mm), it is very likely that active corrosion is present throughout the deck. The red line in the graph shows the chloride threshold value of 0.4% chloride by mass of binder, and it is only in Core E at a depth of greater than 40mm from the soffit, that the level is lower than this.

6.3.3 Jetty 3 and 4 Chloride profiling of deck

In the repair works programme, it was found that finances were possible for more repairs than were initially budgeted for. It was decided that the decks of jetties 3 and 4 should be investigated to see to what level of chlorides had penetrated into the cover concrete, and then to make remedial recommendations.

Three cores were extracted from each deck, and were labelled as follows:
3.1, 3.2, 3.3 and 4.1, 4.2, 4.3

Cores 3.1, 3.2, 3.3 and 4.1 were continuous through the deck, ranging in length from 450 mm to 465 mm. Cores 4.2 and 4.3 were incomplete. In each core steel was present with top cover depths varying from 30 mm (Core 3.1) to >100 mm (Core 3.3). Bottom cover depths ranged from 50 mm (Core 4.2) to 90 mm (Core 4.3).

The cover depths that were obtained for the six cores are shown below in Table 6-2:

Table 6-2: Cover depths as obtained from cores.

(Stated cover is 60mm)
**Steel found at two depths*

Core	Top Cover Depth (mm)	Bottom Cover Depth (mm)
3.1	*30 + 75**	*85*
3.2	*70 + 105**	*Not present in core*
3.3	*120*	*60*
4.1	*80*	*65*
4.2	*110*	*50*
4.3	*95 + 130**	*90*

At a cover depth of 30 mm in Core 3.1 the top steel had begun to corrode heavily and there was some visible loss of cross sectional area.

The visible concrete properties showed the cores to have good compaction, but there were some isolated instances of major voids and large air hole sizes (>2mm diameter).

6.3.3.1 Results of Chloride testing

The results for the 12 profiles are shown in Figure 6-13 below. It should be noted that not all these results are shown in the following graphs, as certain 'outliers' (results so far out of the range to be considered as errata) have been removed to produce more uniform curves.

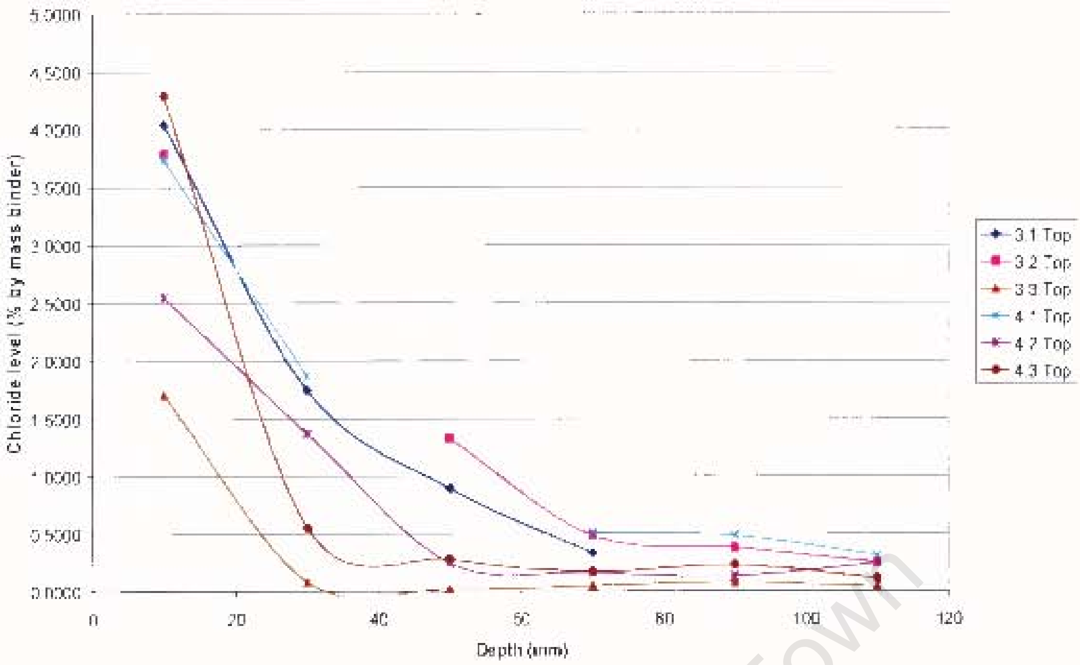

Figure 6-13: Chloride profiles from top of deck

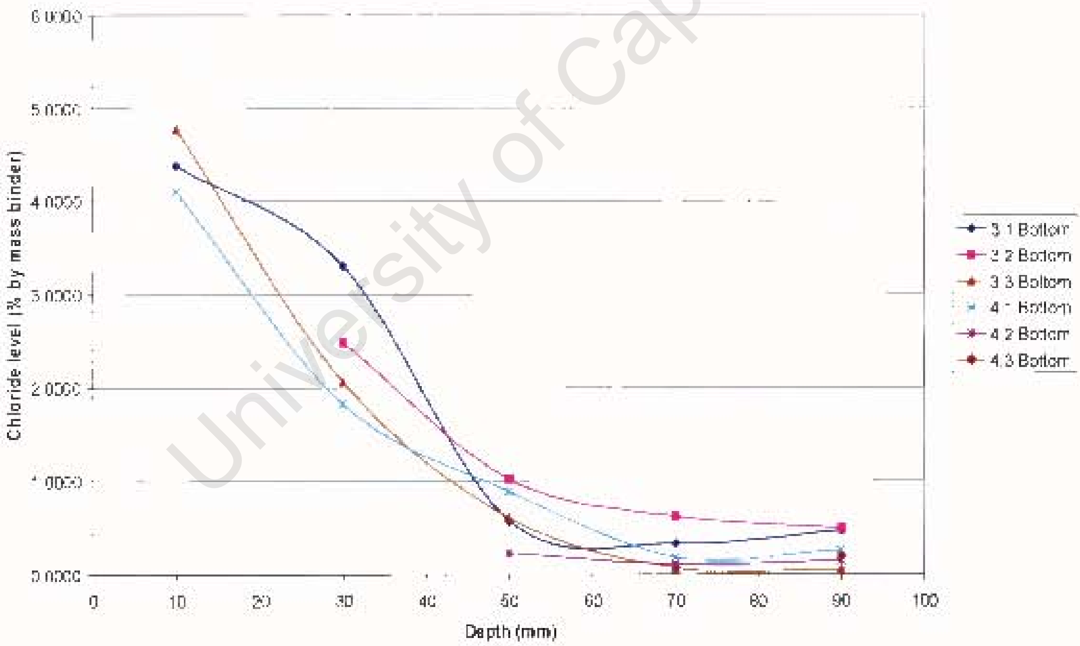

Figure 6-14: Chloride profiles from bottom of deck

6.3.4 Discussion of St Helena Bay Harbour results

The results of the chloride testing showed that for regions where steel reinforcement is to be expected (at 60 mm), the majority of the measured chloride levels were below the 'threshold' level of 0.4% by mass binder. However, the scatter of the results in

Figure 6-13 and Figure 6-14 showed that there were instances where the measured chloride value at a depth greater than 60 mm is relatively high i.e. 0.4%-1.0% chloride concentration by mass binder, for example in Cores 3.1 and 3.2. This would indicate that certain remedial measures could be appropriate for the state of the structure at present. The scatter of the results could be attributed to the variable porosity as evidenced by the presence of some large pores. The variability of the measured cover values present in the cores should also be noted.

On site, it was these results that led the engineers to investigate the opportunities for use of a penetrating corrosion inhibitor to prevent further corrosion damage in the existing structure, based on the success of the testing performed at the Saldanha Bay site (UCT, 2004).

6.4 Closure

The forensic tests presented in this chapter all gave valuable insight into the performance of the concrete materially. The expense and time consumed by these tests are negligible when compared to the positive effect that the results have on decision making and repair specification.

The chloride profiling performed at the Government Jetty at Saldanha Bay Harbour was intended to provide more information to the consultants about the state of deterioration of the structure. The results confirmed the original assumption of a relatively high chloride content throughout, but also lead to the recommendation of specific repair methods, which were tested for suitability in 6.2.2.. These trial repairs gave clear guidance as to what was likely to be the most successful combination out of three surface treatment and corrosion inhibitor permutations. It also showed that is was possible even in such harsh conditions to achieve penetration to a depth of 50-70 mm, something that may well have been considered unlikely without such tests. The chloride analysis of cores extracted from vertical and horizontal elements of the adjacent Trawler Quay also showed high chloride levels, which correlated to the site exposure conditions.

At St Helena Bay Harbour, Jetty 1 was investigated using chloride profiling to determine that state of chloride migration onto the deck and beam substructure. Results from 11 beam locations were obtained and they indicated a high likelihood of active corrosion as the chloride values at depths greater than 50 mm were tending to be larger than the 'threshold' of 0.4 % chlorides by mass binder. Cores through the deck of the same structure revealed even higher values, which served as confirmation to a decision to demolish and replace.

At Jetty 3 and 4, the chloride profiles for cores through the deck showed that at depths greater than 50 mm (the likely depth of reinforcement), the level of chloride contamination was of a range suited to the use migrating corrosion inhibitors as a repair methods. This was recommended as an option for further repair works, being dependant on the allowed budget for the remainder of the contract.

In Chapter 7.1 the results of this chapter are critically reviewed. Special focus is made on the difference in chloride results and profiles between different types of concrete elements, and similar elements with different orientations (such as deck profiles from top and bottom/soffit faces). Contrast is made across sites as well, for example the discussion of deck chloride profiles from both the Trawler Quay at Saldanha Bay Harbour as well and Jetties 1,3 and 4 at St Helena Bay Harbour.

More generally, the value of these tests in gaining knowledge about the deterioration of the structure and informing repair is shown repeatedly.

7 Evaluation and Critical Comparison

A critique of the research presented in the previous chapters can be divided into two categories. The first is specific forensic testing of chloride levels that was performed at Saldanha Bay and St Helena Bay, and this is discussed with a comparison being made between different localities. The findings of the corrosion inhibitor (CI) testing are also discussed further. The second discussion uses examples from all four sites to discuss 'Repair Methodology'. The phases of the repair process, from initial inspection to repair implementation are investigated and key issues for consideration in rehabilitation projects are presented.

As an introduction, Table 7-1 summarises the relevant types of repair and rehabilitation that were investigated during the course of this work. With each location is the method of investigation, labelled the 'Indicator', being the manner in which the damage was assessed. An estimation of the degree of damage is given for each.

The majority of these structures were damaged by reinforcement corrosion, and the level of damage (assessed visually by the author) varied from minor to total collapse. It is evident that more complex and specialised methods of repair such as Chloride Extraction or Cathodic Protection have not been used, while methods that require patch repairs and new construction, either partial or complete, have been favoured. There is a moderate correlation between the degree of damage and the chosen option for repair, but the factors that will be discussed in this chapter (such as relevance, extent) result in a more detailed approach being necessary for repair choice. This will be discussed further in section 7.2. As is common with repair projects in current practice, the main indicator that informs the damage assessment is a visual survey. The need, of course, for an appropriate methodology to move from assessing damage to applying repair methods is also discussed further. Even within the range of localities and repair methods that were adopted in these contracts, the need for a sound repair strategy that takes into account a holistic view of the repair process is vital.

Table 7-1: Summary of Repair Works investigated in this Thesis

Type of Repair	Location	Indicator	Description of Damage
Do Nothing	Jetty 3 and 4, St Helena Bay	Chloride testing	Minor cracking, rust staining present
	Alt. Design Turning Bay North Mole, Hout Bay	Visual, Collapse evident	Major collapse in sections
Surface Treatment	Government Jetty, Saldanha Bay	Visual, Chloride	Major cracking and spalling
	Trawler Quay, Saldanha Bay	Visual, Chloride	Major cracking and spalling, large deflections
	Jetty 1, Hout Bay	Visual	Minor cracking, rust staining present
Patch Repair	Guniting – Jetty 2, St Helena	Visual	Major cracking and delamination, rust staining
	West and East Breakwater, Laaiplek	Visual	Cracking and delamination present, settlements
	Government Jetty edge beams, Saldanha Bay	Visual	Major cracking and spalling, mechanical damage
Partial Rebuild	Jetty 1, Quay 3, St Helena Bay	Chloride testing	Major Rust staining, high chloride levels
	Pepper Bay, Saldanha Bay	Visual	Major cracking and rust staing present
	Repair Jetty, Saldanha Bay	Visual	Unknown
	Original Design, Turning Bay North Mole, Hout Bay	Visual, Partial collapse	Major collapse in sections
	End of East Breakwater, Laaiplek	Visual, Major settlement	Major settlement of structure
Complete Rebuild	Final Design Turning Bay North Mole, Hout Bay	Complete collapse	Total collapse
	New Jetty, Hout Bay	Major deterioration	Major cracking and spalling
	Trawler Quay Columns, Saldanha Bay	Collapse	Total deterioration capacity loss
	Deck of Jetty 1, St Helena Bay	Chloride testing	Minor cracking, moderate chloride levels
	Central Span Beam Jetty 1, St Helena Bay	Visual, Major cracking	Major cracking and delamination, severe staining
	Reinstated slabs – West Breakwater, Laaiplek	Visual, Major settlement	Major settlements, steel loss due to corrosion

7.1 Discussion on Forensic Investigations

This thesis made a study of reinforced concrete structures that were undergoing concrete repair because of damage from chloride-induced corrosion. Coring and chloride profiling was used to assess the level of chloride contamination of elements in order to further establish corrosion risk. This section discusses these results. contrasting them across the different sites.

7.1.1 Chloride Profiling at Saldanha Bay

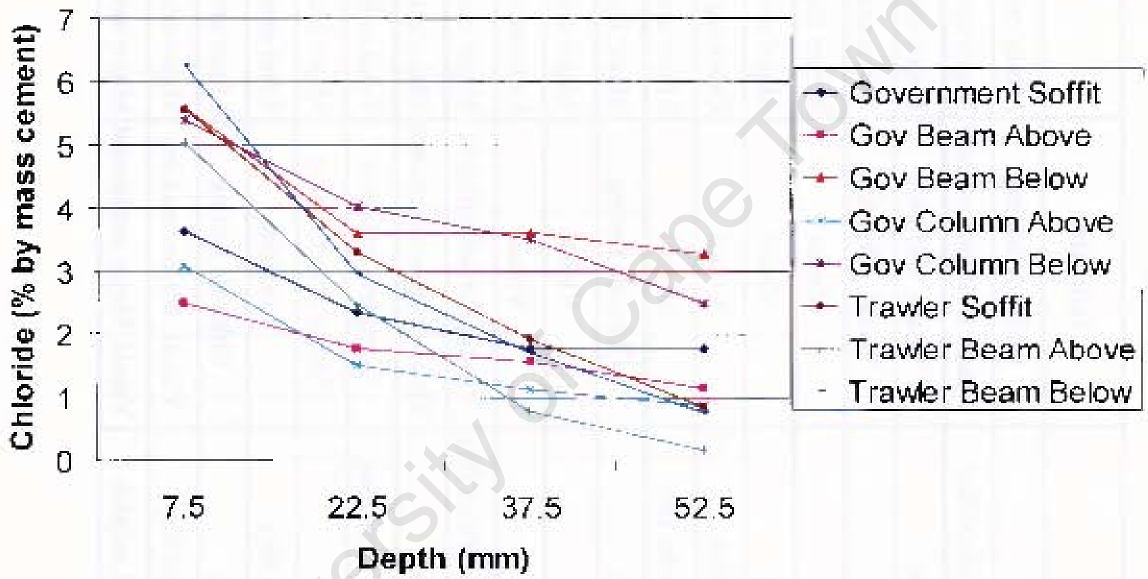

Figure 7-1: Summary of Chloride Results for Saldanha Bay

Figure 7-1 shows a summary of the results obtained from the Government Jetty and Trawler Quay chloride investigations. The results of the tests are presented fully in Chapter 6.2.1, but a key point that led to the recommendation of repair methods was the high chloride levels that were found at large depths (> 55 mm). The values were high enough to recommend that suitable long-term measures be investigated/budgeted for. as the recommended measures (Corrosion Inhibitors and Surface Treatments) would only be suitable to 'buy time'.

The results also revealed an interesting point. shown in Figure 7-2. that chloride levels below the mean sea level (MSL) were significantly higher in general than the than those for the samples extracted above MSL. This is perhaps to be expected with the areas below MSL being in constant contact with sea water, but the visible damage on site did not correlate with the level of chlorides. This was attributed to the fact that the corrosion process operates at a slower rate in saturated conditions, i.e. it is starved of oxygen. Thus it was not deemed necessary to specify repair techniques to elements that were permanently submerged.

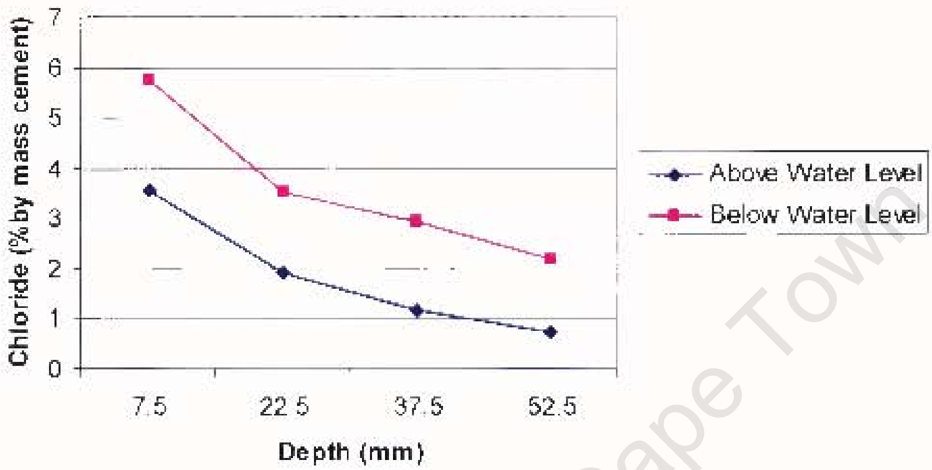

Figure 7-2: Saldanha Bay Chloride results showing influence of mean sea level

7.1.2 Corrosion Inhibitor testing at Saldanha Bay

The chloride testing described above led to repair recommendations and the on-site testing of the remedial measures. A summary is given below (See chapter 6.2.2 for more detail):

- *Corrosion Inhibitor alone*: Moderate penetration
- *Corrosion Inhibitor with Barrier Coating*: Additional coating appeared ineffective
- *Corrosion Inhibitor with Hydrophobic Surface Coating*: Performed best of the three combinations – providing inhibitor concentrations at depth

The limited trials of a penetrating corrosion inhibitor at the Saldanha Bay Government Jetty showed that it was possible for a repair trial to be implemented and produce results that could affect full-scale implementation. Thus, wherever possible, repair trials should be implemented on site and monitored for effectiveness prior to full scale repair. The problem with such a recommendation is the fact that many repair procedures use methodologies that are such that effective monitoring is not only time consuming, but costly as well (for example using in-situ corrosion rate measurement to monitor chloride extraction or sacrificial anode systems). The results of a trial repair need to be available within an acceptable timeframe for the particular project.

The trials performed at Saldanha Bay showed that the corrosion inhibitor was able to reach sufficient depth when used in conjunction with a hydrophobic surface treatment, despite the high chloride level and relatively saturated concrete. It was not possible to measure the effect that this inhibitor actually has on the corrosion rates of the reinforcement in the concrete, as the specific circumstances of the project required immediate action. The predicted success of these works is then partly based on the guidance given by the forensic testing done at the location, but also on information gleaned from inhibitor tests under similar conditions such as the tests performed by Heiyantuduwa and Rylands (UCT Monograph No 7, 2004, also see 2.7.3.1). It is likely that these methods will slow corrosion rates, but as indicated in the repair trial reports, this will only 'buy time' before further repairs are necessary.

The combination of a hydrophobic coating and a penetrating corrosion inhibitor was shown to be the most effective in obtaining a good penetration depth, which is necessary for reinforced concrete elements with deep cover.

7.1.3 Chloride Profiling at St Helena Bay

Eleven transverse beams were drilled and tested for chlorides at Jetty 1 at St Helena Bay. The purpose of this was to assess the condition of the beams, which were showing signs of corrosion damage (See Chapter 6.3.1). Several adjacent beams had already spalled and cracked to such an extent that demolition and replacement was required, but more insight was required as to the possible future performance of the remaining eleven.

The results are summarised in Figure 7-3, with good consistency obtained across the eleven samples. An average value for chloride concentration at 50 mm depth (the depth at which steel was found in a sample) was approximately 0.6% by mass of cement, which is above the 'threshold' value of 0.4% where corrosion is suspected to commence. This was reported to the consulting engineers for the project and the decision was taken to replace all the beams in the structure, as well as the deck which was also found to be severely contaminated (See next section). The piled columns, which were not investigated, were retained and the new beam and slab system was cast upon them.

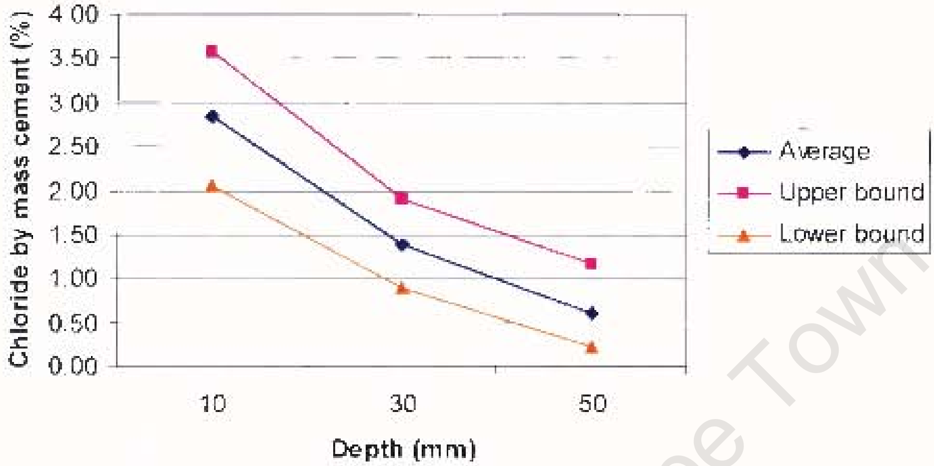

Figure 7-3: Summary of Jetty 1 Beam Chloride Profiles – St Helena Bay

The implications of this decision are noteworthy. The recorded chloride levels were within the range that would potentially be suitable for the application of corrosion inhibition, but instead of opting for this method (as was implemented at Saldanha Bay), the decision was made to demolish and reconstruct the beam and deck elements. The reason for this decision was that the central spanning beam had already deteriorated beyond the point of repair, and thus it was impractical to retain the rest of the structure whilst replacing that element. Where this not the case, it is likely that remedial measures such as surface treatments and corrosion inhibitors would have been more suited for the repair.

7.1.4 Deck Coring Results

The chloride profiling of four reinforced concrete jetty decks (Jetty 1, 3 and 4 at St Helena bay and Trawler Quay at Saldanha bay) showed the difference in chloride profiles between the top and bottom faces of reinforced concrete decks. Jetty 1 at St Helena Bay was tested prior to demolition, whilst the other decks are still in use. The combined profiles are shown in Figure 7-4, expressing chloride level versus depth from top face (t) or bottom/soffit (b).

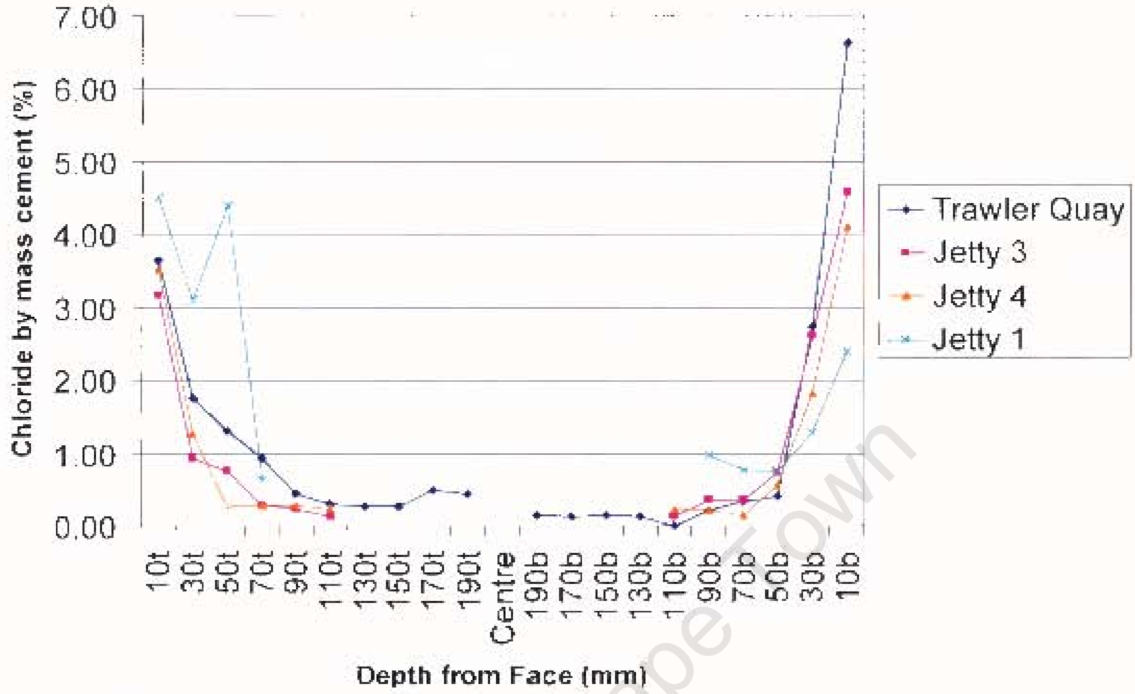

Figure 7-4: Chloride profiles for deck cores

7.1.4.1 Chloride penetrated to depth from both sides

It was found in the chloride profiling results that the chlorides had been able to penetrate to a significant level (>50 mm) from both the top surface as well as the soffit of the deck slabs. It could be expected that the close proximity of seawater to the soffit would result in increased levels of chlorides, relative to the top surface. This is shown to be true for the soffit, which shows higher surface chloride values than the top of the deck. A factor that may influence the migration of chlorides from the deck surface could be the ponding of seawater on top of the deck for appreciable periods of time. This would create a constant driving pressure head as well as a constant source of chlorides, and would results in more chlorides penetrating to greater depth. The likelihood of such an occurrence is debatable, and was not observed on site.

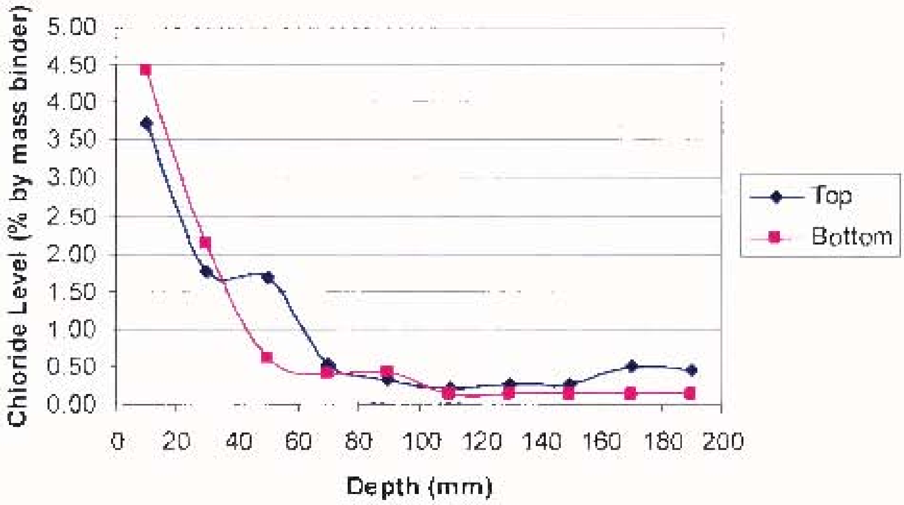

Figure 7-5: Average chloride profiles for deck cores from top and bottom

Figure 7-5 above shows the averaged profiles from Figure 7-4. Beyond a depth of 100 mm, the differences between the profiles are minimal, but there is a moderate difference shown in the near surface results. For the top surface, the chloride transport mechanisms of sorption and diffusion account for a higher result at depth (50mm), while flushing during rain could potentially account for the lower surface concentration. For the bottom surface, the more frequent contact with seawater explains the higher surface concentration while the mechanism of diffusion only would link to a lower concentration at depth.

7.1.4.2 High surface levels

Both surfaces showed very high surface chloride concentrations. Also notable was the lack of any presence of 'leaching' or removal of surface chlorides through mechanical means such as wave action, abrasion, etc.. The fact that that both surfaces are horizontal would affect the impact of wave action on 'washing' the surfaces of chlorides, in that the surface would be in less contact with splashing water than if it were vertical.

7.1.4.3 Critical Depth of Chlorides

The profiles of the cores extracted from all jetties except the excessively damaged Jetty 1 show an increase in chloride concentration at depths shallower than 40-60 mm. While the profiles for Jetty 1 indicate that chlorides will most likely move through the cover concrete in larger quantities, it is suggested that an initial value that should be adopted as 'adequate' for cover should be larger than 40-60 mm, more likely 75 mm, assuming OPC concrete (as would have been originally typically specified for these works) is used. Chapter 5 shows that the majority of the contracts investigated adopted a 60 mm cover depth. These results show that this depth is perhaps not conservative enough for life spans of more than 30 years, unless a more suitable binder material (such as blended binders with extenders e.g. Fly Ash, Slag). The results also show the very rapid chloride level increase in the cover of the concrete.

7.1.4.4 Jetty 1, St Helena Bay

The results of the chloride profiling of the Jetty 1 deck stand out amongst the others, showing higher chloride concentrations throughout the deck. This was found to be consistent with the appearance and visible damage indicators on site. Extensive cracking and spalling had occurred in the jetty, and the visual indicators alone were enough in the mind of the engineer upon which to base a 'demolish' decision. It was decided that it was impractical and would be too costly to endeavour to salvage certain elements of the structure, while replacing the deck. A new beam and deck system was opted for, built upon the existing piles. A correlation between chloride testing and actual performance is shown.

At Saldanha bay, surface washing/leaching and the effect of gravity was deemed to have influenced the surface concentrations of both chlorides and penetrating corrosion inhibitors. Current specifications for new structures, such as the specifications discussed in 3.2, do not specify different levels of cover for top faces and soffit faces, but possible differences or correlations in their 'chloride behaviour' may prove relevant in the sphere of repair. They could inform different specifications for repairs to deck soffits and surfaces, but also help in an understanding of the influence of surface washing and gravity in the transport of chlorides through concrete.

7.1.5 Conclusions

Forensic tests on concrete elements in the RAMP programme have been presented and discussed. The use of forensic tests provides the engineer/consultant with quantifiable data upon which to base his/her repair decisions. The ability of chloride profiling to show corrosion risk is used to gain insight into the performance of the elements, and to establish the potential future performance as well. Certain levels of chloride are also more suited to certain repair techniques, as was seen in Saldanha Bay Government Jetty, where 'high' to 'moderate' chloride levels permitted the use of corrosion inhibitors in order to 'buy time'. At St Helena Bay, however, the chloride levels in the deck of Jetty 1, coupled with the existing damage, led the engineers to recommend that the structure be replaced.

Repair decisions/processes adopted at those locations have been contrasted. While the most common method of assessment throughout the numerous investigated works was on a visual basis, it is clear that the relatively minimal expense of performing forensic tests where possible proved valuable in understanding the performance of these structures/elements from a durability viewpoint. The outputs of the chloride tests in particular, such as surface concentration, critical chloride depths, rates of chloride movement and relative concrete material performance all have direct bearing on material performance, and help to understand how reinforced concrete behaves in a severe marine environment.

7.2 Discussion on Repair Methodologies

The route taken by a consulting engineer in moving from initial damage detection and assessment to repair method implementation and monitoring is discussed in the following section. It is divided into four parts:

- **Damage Detection**
  - Accuracy of damage assessment methods
  - Forensic testing
- **Decision Making/ Detailed Design**
  - Problems in relating degree of damage to extent of repair
  - Repeating older designs in new construction
  - Combination of new and aged reinforced concrete elements
  - Link between binder type and durability
  - Cover to reinforcement
  - Use of a repair philosophy
- **Implementation/ Execution**
  - Guidance and direction from Contract Documents
  - Quality control of concrete on site from a durability perspective
  - Trial Repairs
- **Monitoring/Assessment**
  - Future performance of repair measures
  - Future maintenance planning
  - Cost effectiveness in repair contracts

Key questions are asked in each section, and the projects that have been presented in this thesis will be discussed and evaluated. The objective is to use this framework to assess current practice using the four RAMP contracts investigated in this work. Reference will be made to sections of the literature review presented in chapter 2 for more detail where necessary. The chapter will conclude with a presentation of a revised breakdown of repair methodology to be considered in new repair contracts.

7.2.1 Damage Detection

7.2.1.1 Accuracy of damage assessment methods

Before discussing the appropriate specification for damaged concrete, it is important to investigate the accuracy of current damage assessment tools, i.e. the correlation between damage measurement and actual damage recorded within the structure. The method of investigation that is used in the assessment of damaged reinforced concrete structures, especially that of visual inspection, is reliant on human factors and so a large degree of variance is expected. Simple reliance on one method (such as visual inspection) is not recommended and rather an approach that incorporates different methods of assessing and quantifying damage should be used. The results produced by the forensic testing at St Helena Bay and Saldanha Bay reinforced this point in that they influenced the repair choice directly (less to the prescription of surface treatments as a suitable repair method. Had the choice been based solely on visual inspection methods, it is likely that a less appropriate repair method would have been adopted, such as isolated patch repairs.

As was discussed in chapter 2, it is important for a consulting engineer to have a good understanding of what the mechanism of deterioration at hand is, and especially so in reinforced concrete structures, as tell-tale signals of reinforcement corrosion are often present. It is possible to misread the extent to which the structure has been damaged as well as the relevance of such damage.

An example of this in practice is the investigation of repair works at Laaiplek Harbour. The findings in Chapter 5 showed corrosion damage in the form of exposed reinforcing steel elements that had lost cross sectional area to a great extent. The relevance of such damage, however, is lessened by the mass concrete nature of the quay/breakwater. The steel was not critical and was most probably included in the design of the structure for other considerations (crack control, handling etc) rather than performing a structural function. The need for repair might thus be different from a structural viewpoint, i.e. no danger of structural failure present.

7.2.1.2 Forensic testing

Table 7-2 below summarises the forensic testing performed at the four sites. Two of the four contracts used forensic testing as a means to gain further insight into the level of deterioration, and also to help inform repair methods.

Table 7-2: Forensic testing

Location	Laaiplek	Hout Bay	St Helena Bay	Saldanha Bay
Forensic Testing	*None*	*None*	*Chloride analysis, Cement content determination*	*Chloride analysis, Corrosion inhibitor testing*

The chloride analyses that were performed at Saldanha Bay on Government Jetty (Chapter 6) showed very high chloride levels, and this corresponded to the visible damage, confirming the assumption of a high corrosion risk. The results were also able to indicate that certain repair measures would only be satisfactory if followed up by more long term measures at a later stage. The high chloride levels did not always correlate to corrosion damage, as was seen in elements below the water level where the corrosion process was starved of sufficient oxygen for there to be a corrosion risk.

The recommendation of using a migrating/penetrating corrosion inhibitor, and further trial repair testing, produced the recommendation of a specific combination of inhibitor and silane-based surface treatment. The use of further forensic testing in the form of repair trials provided valuable information, relevant to the particular location and repair type in terms of showing the suitable treatment combinations for repair.

The repair works at St Helena Bay (Chapter 6) were also guided by the results obtained from chloride profiling. The results of chloride profiling of the beams at Jetty 1 confirmed the likelihood of active corrosion, while the cores through the deck produced very high chloride levels, enforcing the decision to replace the beams and deck. Cores through the decks of Jetty 3 and 4, however, did not produce as high levels of chlorides and thus did not cause the consulting engineer to investigate any further repair methods. Again the ability of forensic testing to guide decisions is shown. In the absence of this, it is possible that expensive and unnecessary repairs may have been performed on the structures.

One clear point emerging from the forensic investigations is that it is possible for relatively 'low cost' investigations to produce good results that can impact positively upon the decision process. In particular, the use of chloride analysis in reinforced concrete elements in the marine environment has been shown to produce results that have real value in not only assessing the level of damage and corrosion risk, but also in prescribing appropriate remedial measures. The costs of the investigations, when compared to the project as a whole (Approx R10 million per site), are typically minimal (unlikely to be more than a few percent), and it does not take long to obtain results. While the exact cost of the repairs was not available to this research, it is possible that these investigations might help save money from a long term point of view.

Certain locations may limit practical forensic testing, but its value has been shown and thus it is recommended that it should be addressed as mandatory wherever possible.

7.2.2 Decision Making/ Detailed Design

7.2.2.1 Problems in relating degree of damage to extent of repair

Reinforced concrete gives signs of corrosion damage (such as staining, spalling) before ultimate failure, but unfortunately this relationship is not always clear or predictable. As was the case in certain of the structures that were investigated in this thesis, for example the deck of Jetty 1 of St Helena Bay Harbour (Chapter 7), elements showed the expected high levels of damage on demolition of the structure. However, the lack of any damage to the permanently saturated columns at Saldanha Bay (which showed high chloride levels in Chapter 6) showed that one cannot rely on these results as indicators without understanding the relevant deterioration mechanisms. The discrepancy between the predicted and the actual in reinforced concrete should also be noted, and an example of this in this work could be the relatively moderate chloride values found at Jetty 3 and 4 at St Helena Bay, despite the presence of corrosion staining. Reasons for this discrepancy could include unknown information pertaining to the structure age, binder type, cover depth, density, etc. Thus information gained about a structure (as has been shown by forensic testing especially) will only benefit the repair process and improve practice.

Also key to the correct application of remedial measures is the understanding of the full consequence of ultimate structural failure. The 'relevance' of the loss of use of the structure invariably has an impact on repair philosophy, in that in may be deemed unnecessary to repair something that is not utilised, whereas a heavily utilised structure may seem more in need of urgent repair. Table 7-3 shows examples from this work.

Table 7-3: Examples of structures with different usage requirements.

Location	**Laaiplek**	**Hout Bay**	**St Helena Bay**	**Saldanha Bay**
Structure	*Western Breakwater*	*Northern Mole Turning Bay*	*Jetty 1*	*Government Jetty*
Usage	*Low to moderate*	*Low*	*Moderate*	*High*
Reason	*Long, isolated structure, predominantly mass concrete*	*Structure had collapsed without any change to harbour usage*	*Recreational jetty, not heavily utilised*	*Commercial jetty, heavily utilised*

The repair needs of a heavily utilized Jetty, such as Government Jetty in Saldanha Bay, are different to those of the Western Breakwater at Laaiplek, because of the difference in day-to-day use, the likelihood of encountering full loading as well as the risk to human life if structural failure were to occur. The repair of the surface concrete damage at the Laaiplek locations is more of an aesthetic function than one that has serious structural significance.

The various factors that impact upon repair choice, such as the consequences of failure and the available budget would affect the relationship between the level of damage that is recorded and the actual repair level that is implemented. Thus the application of remedial measures cannot be simply reduced to a linear function whereby the greater the assessed damaged, the greater level of repair that is implemented on site. The approach used by the engineer should incorporate all relevant factors.

In some damage detection systems (such as the method proposed by Andrade – Chapter 2.6.4) there is the opportunity for the engineer to rate the relevance of the assessed damage in a matrix system to ascertain which structures are most in need of repair.

7.2.2.2 *Repeating 'older' designs in new construction*

An interesting example of 'repeat' construction, in fact 'repeat' repair method, is the repair of Jetty 2 at St Helena Bay (Chapter 7.1.2). Original repair works in 1977 involved the removal of cracked concrete areas and the coating of the 'reinforcement' steel (large RSJ sections) with epoxy tar, and then covering with a layer of new concrete. Corrosion of these large sections had resulted in large scale cracking and spalling of the cover concrete, and it is interesting to note that the new repair methods involved the application of fresh concrete in the form of gunite/shotcrete as a protective layer (implying that the most efficient protection to reinforcement in reinforced concrete is in fact more concrete). The point here is that the original repair works are being mimicked by the engineers, with minimal incorporation of new products and methods.

Unfortunately there exists a mindset at the core of some engineering philosophies that in repair projects, it is simply enough to replace what was initially constructed and not to include advancements in knowledge that may have occurred since the original conception of the particular structure. Whilst it is acknowledged that perhaps the brief for the repair project may require no more than a repeat of the original construction, the failure on the part of the engineer to be able to adequately incorporate modern methods and materials could unnecessarily disadvantage the future performance of the structure.

7.2.2.3 *Combination of new and aged reinforced concrete elements*

Table 7-4 shows examples of the use of new and aged reinforced concrete elements in the four contracts presented in this thesis.

Table 7-4: The combination of new and existing elements

Location	Laaiplek	Hout Bay	St Helena Bay	Saldanha Bay
Description	*Multiple patch repairs to edge beams. Replacement of top slabs to western breakwater*	*Original designs required a new span to be built into the existing northern mole*	*Guniting works to existing structures at Jetty 2. Construction of a new deck upon existing piles at Jetty 1*	*Piles heads re-jacketed at Pepper Bay and Repair jetties. Patch repairs to Government Jetty*

The nature of the development of infrastructure such as ports and harbours is that at any given time there will be a large range of structures of different ages. The varying demand of the shipping industry requires structures to be adapted and altered, and with this change comes the combination of new and existing reinforced concrete elements.

From a durability perspective, this can create confusion as sampling methods are used to assess the structures/locations as a whole. At Saldanha Bay, cores extracted from an edge beam on top of the Trawler Quay revealed two very different types of concrete resulting from a patch repair that had previously been performed on the structure. Different materials in different elements lead to different rates of deterioration, and caution must then be taken to ensure that an accurate assessment of the entire structure is used to inform rehabilitation decisions.

On a smaller scale, the implementation of patch/partial repairs to elements results in similar 'composite' structures.

The patch repairs to the breakwaters of the Laaiplek harbour provide a good example of this, where the worst locations of damage are being replaced with new 'patches' of fresh concrete. The likelihood of failure of such methods is noted in the first chapter of this work, owing to the probability that 'incipient' anodes will form adjacent to the new concrete, and corrosion cells could then be formed at an early age post-repair.

The works that were implemented at St Helena Bay Harbour at Jetty 1 also provide an interesting example. Upon initial investigation, it was clear that previous repair methods, such as patch repairs had failed. The deck and beams were removed from the jetty, demolished and replaced with a decking system comprising precast elements as well as in situ cast elements, built upon the original piles/columns (See Chapter 7.1.1). The future performance of the structure as a whole will be more complex to assess as not only will the different elements of the structure be at different ages, the likely difference in concrete quality between precast and in situ elements will affect the performance in terms of durability (as was suspected with the Government Jetty at Saldanha Bay). Of course, this situation would be even more critical had the engineer decided to retain some of the beams that were tested for chlorides.

7.2.2.4 *Link between binder type and durability*

The choice of binder type is important for marine concrete and bears some discussion. In chapter 4 the concrete specifications detailed in the contract documentation required an extender (either Fly Ash – FA or Ground Granulated Blast-furnace Slag – GGBS) to be used in conjunction with Ordinary Portland Cement (OPC) for reinforced concretes. The detail shown on the contract drawings, however, stated a minimum compressive strength and an aggregate size (eg. 45/19). It is common practice to base concrete specifications on these values alone, i.e. an engineer will be satisfied with the concrete mix provided it has the prescribed strength at 28 days. Thus the actual concrete properties, based on prescriptive specifications, are assumed and not measured (e.g. the engineer has little proof that FA or GGBS are actually used on site).

Table 7-5: Binder type/compressive strength requirements

Location	Laaiplek	Hout Bay	St Helena Bay	Saldanha Bay
Cement content/compressive strength requirements	*45/19*	*50/12 or 50/19*	*40/19*	*Reference made to standard project specifications*

The specified concrete strength requirements varied across the contracts, ranging from 40 MPa to 50 MPa. This relatively high strength requirement comes from a durability rather than a structural requirement. Traditionally the expected performance of a reinforced concrete structure in terms of durability has largely been based on the cement content of the concrete mix. This corresponds to a high compressive strength, and thus a traditionally 'acceptable' method of specifying for durability has been by requiring a minimum cement content, or compressive strength that must be attained.

In the existing structures, the Government Jetty that was investigated at Saldanha bay gave good support for such an approach, where the chloride penetration levels of members suspected to be precast were lower than those deemed to be in-situ members. Whilst full compressive strength testing was not performed, it would be expected that the precast elements would have a higher cement content than in-situ placed concrete. The age of the structure would indicate that it is likely that Ordinary Portland Cement would have been used as a binder. However, it should be noted that in at least one structure it was suspected that a blended binder was being used.

At St Helena Bay, the cores extracted from Jetties 3 and 4 were sent for analysis to determine cement content, and the results were higher than the usually assumed value (360 kg/m^3 and 400 kg/m^3 compared to 300 kg/m^3). The chloride results for these cores were also better than expected, showing the influence that cement content can have on the durability of reinforced concrete. It is to be expected that more recent structures using a minimal cement content could potentially perform worse than older structures, thereby ruling out age as the main factor governing repair.

As advances in concrete technology have been made it is clear that there is more involved in achieving durable concrete than simply including a high cement content. Factors such as binder type, curing, porosity, exposure conditions and cover contribute greatly to the durability of a concrete, and thereby discount the method of specifying for durability from a strength only perspective. With the choice of binder type especially, concretes can be made significantly more durable with minimal extra expense, but it seems that this is given little prominence is current practice.

7.2.2.5 Cover to reinforcement

The cover specifications for the RAMP contracts investigated in this thesis are shown in Table 7-6, and while three sites use a standard of 60 mm, St Helena adopts a 75 mm cover as standard.

Table 7-6: Level of cover

Location	Laaiplek	Hout Bay	St Helena Bay	Saldanha Bay
Cover requirements	*60 mm*	*60 mm*	*60 mm – precast* *75 mm – other*	*60mm*

The most economical protection of steel in concrete is often regarded as being more concrete. The practicality is, however, that it is often difficult during construction to achieve good cover consistently. The need for large cover depths becomes less important as the concrete quality is improved.

The various tests on existing structures uncovered some serious fluctuations in cover in use across these sites. Locations such as the Jetties at St Helena Bay revealed cover depths of less than 30 mm on occasion. In comparison, the specified cover in use within the repair works programme at that locality is in excess of 75 mm.

The chloride profiles from both St Helena and Saldanha Bay indicate that cover depths of less than 50 mm should not be used for marine concretes, unless specific binders are used to address durability considerations. A more conservative value of 75mm is recommended, as it seems unlikely that this will incur much extra expense when factoring in future repairs into the budgeting system. As an example, a 30 mm increase in cover depth to a 500 mm slab increases the required concrete by a mere 12%. However, it must be noted that simply adopting a larger cover depth and ignoring concrete material properties such as appropriate binder types will not result in a durable structure.

7.2.2.6 Use of a repair philosophy

Table 7-7 gives a summary of the repair philosophies that have been gleaned from this work. The philosophy of repair at Laaiplek Harbour was such that little forensic work was performed and standard repair works were specified. Similarly at Hout Bay, it is apparent that the repairs that were performed were driven by a structural nature, as opposed to concrete material considerations.

Table 7-7: Description of general repair philosophies

Location	Laaiplek	Hout Bay	St Helena Bay	Saldanha Bay
Description of general repair philosophy	*Combination of reconstruction and patch repairs; minimal repair performed in a very exposed site. Some use of repair products for surface treatment.*	*Standard 'run of the mill' repair methods adopted, focus made on new construction based on structural requirements as opposed to concrete material repair*	*In depth investigations, more conservative designs specified. Little if any use of repair products***	*Effort made to do both forensic and repair testing, some full reconstruction, but also some 'time buying' methods adopted. Extensive reliance on repair products*

** Repair Products: Surface Treatments, Grouts, Repair Mortars as shown in Table 4-5.

At St Helena Bay effort was made to understand the material characteristics of the structures that were being repaired. At this site there was also the tendency for conservatism in design, as they specified the highest level of cover to new structures and did not incorporate the use of concrete repair products or specifically recommend particular binders.

The Saldanha Bay contract spent time and money on forensic investigations, and utilised the latest repair products in their specifications. Account was made for the future performance of the structures. Methods applied at Government Jetty, for example, would suspend deterioration by ten years, when it was suggested that more repair works would be needed.

The adoption of a repair philosophy is suggested as a means of assessing and implementing repair projects from a holistic perspective. There is a danger in individually dealing with damaged areas in that it is possible to lead to a situation whereby parts of a location or structure are repaired to a much higher level than others. Such unevenness in approach, while maybe sometimes necessary or expected, is not suggested as a sustainable solution.

Connected to this is the question of whether it is better for a group of structures to all require maintenance at the same time, or rather to be able to spread the maintenance cost over a number of years. This would depend on what system (if any) is put in place by the owners of the structures to meet the budget requirements of a maintenance programme. It is the opinion of the author that it is very rare that such maintenance considerations are made when the structure is designed. Typically, if based on anything, design philosophies are time based; i.e. it would be required that the particular structures require no maintenance for the specified number of years after the repair works are performed.

7.2.3 Implementation/ Execution

7.2.3.1 Guidance and direction from Contract Documentation

Table 7-8: Contract documentation evaluation

Location	Laaiplek	Hout Bay	St Helena Bay	Saldanha Bay
Project Specifications	*Standard SABS 1200*	*Standard SABS 1200*	*Standard SABS 1200*	*Standard SABS 1200*
Particular specification	*The same for all contracts with minor editing*	*The same for all contracts with minor editing*	*The same for all contracts with minor editing*	*The same for all contracts with minor editing*
Construction drawings	*Clear and relatively basic*	*Clear and relatively basic, included alternative options*	*Detailed, processes explained clearly and concisely. Included construction sequencing*	*Good, adequate for the construction works. Included detail of past construction and repair works*

The guidance and information that is relayed to the people performing the repair works is critical. In the form of drawings or contract documents, it is crucial that enough relevant information be given to allow the people responsible for the physical application of the works to do an adequate job. Repair methods such as patch repairs require specific attention to detail for the preparation of the existing concrete, and incorrect procedures will result in the failure of the repair system.

It is seen in Table 7-8 that many of the specifications and descriptions of works that are given to the contractors are simply copied from contract to contract and drawing to drawing without being given proper evaluation. It is likely that the contractors on-site, and especially the artisans putting the work to hand, will not have access to the full set of project specifications. Thus the communication of relevant information and instructions are critical to the success of the repair works, especially those that involve very specific procedures.

The difference in level of detail between the contract drawings (at St Helena Bay and Saldanha Bay especially) is notable. While the drawings from the Hout Bay and Laaiplek contracts (both produced by the same engineering firm) are of a relatively basic nature, the drawings at the other two locations show more detail and relevant information, showing that difference in engineering standard and opinion can even exist between two project teams within the same consultancy. This shows the individual nature of each contract, and how personal style and preference can impact a repair contract.

7.2.3.2 Quality control of concrete on site from a durability perspective

Table 7-9: Site supervision

Location	Laaiplek	Hout Bay	St Helena Bay	Saldanha Bay
Description of site supervision	*Regular visits by Technician in charge*	*Qualified graduate Engineer on site*	*Regular visits by Engineer in charge*	*Qualified and experience senior Engineer on site*

There is a noticeable trend in current practice (reflected at the St Helena Bay and Laaiplek sites) to move away from having permanent engineering consultant representatives based on-site. At Hout Bay and Saldanha, however, it was decided to have resident engineers on-site permanently.

Another method of improving quality control on site, a method not present in these RAMP contracts, is to introduce performance-based specifications into the contract. At present it is not standard practice to include criteria in the contracts in which concrete can be assessed from a durability perspective, as opposed to strength criteria. It is performed occasionally by the contractors at the request of the consultant, but until it is enforced in regular specifications, there is little control of what the properties actually are of the concrete that is being produced in these repair contracts. A performance-based specification, perhaps even incentivised as a payment item, would contribute greatly to ensuring a high quality product.

7.2.3.3 Trial Repairs

The only instance of trial repair evident within these contracts was the trial corrosion inhibitor testing at Saldanha Bay, despite it stating in the contract documents that the testing of repair methods is mandatory. Such 'short cuts' by contractors will only save them marginal time and expense, but will cost the consulting engineer invaluable information. It is strongly recommended that these trials be enforced on site.

7.2.4 Monitoring/Assessment

7.2.4.1 Future performance of repair measures

Repair contracts typically have very little provision for the future monitoring of the repair works that have been implemented. By revisiting the sites at some future date, lessons can be learnt as to how the various repair measures have performed, and this can in turn help to guide the future application of such repair technology. In many technologies, for example with cathodic protection, elements that allow for future monitoring of the structure can be built into the repair works. The point here is that with proper design and execution, the repair works should have some sort of guarantee of performance.

Along with such a provision comes the opportunity for the consulting engineer/specifier, manufacturers and contractors to be held accountable for the performance of the repair technology. If performance is measured, and found to be of an unacceptable quality, payment can be withheld.

7.2.4.2 Future maintenance planning

The RAMP project came about as a result of the little or no maintenance to the harbours for nearly twenty years. It would then follow that future repair/maintenance requirements would be part of the repair methodologies used by the relevant consults. However, the projects and methodologies that have been investigated in this work do no show such provision, except perhaps at Saldanha Bay. It is unfortunately then likely that this situation will re-occur.

7.2.4.3 Cost effectiveness in repair contracts

Incorrect repair specification and improper execution of even correctly chosen methods will result in situations where not only will the work have to be redone, but this will have to happen at an even greater expense to the client, as it is likely that damage will have progressed even further by that stage. It is a commonly held belief that more money spent at an earlier stage on a higher 'level' of repair will result in a higher quality end product which will last longer. This is not always true, and simply allocating funds to a repair project will not necessarily improve its chances of success.

Common practice gleaned from these four contracts tends to the conservative side, favouring 'familiar' repair methods as opposed to more expensive risky technologies, especially those that do not have a history of use in the surrounding areas.

Whilst the full costs and budgets of these contracts were not available to this research, what was evident was that finance was the major driving factor in the decision process for repair. From the outset, the RAMP programme was initiated as a result of the majority of the structures that were identified not having had any significant maintenance work in almost 20 years. It is surprising then, that with these large contract values (R8 million to R13 million), more money isn't being spent on adequate forensic investigation and repair trial testing.

Even on a smaller scale, many of the decisions that were made in the individual contracts were financially based – an example is Jetty 3 and 4 where remedial

7.3.3 St Helena Bay

At St Helena Bay it appears the approach was more conservative in nature. While this contract had the lowest concrete strength requirements of the four, the adopted cover for new structures was above average as well as the tendency to replace versus repair. The repair works also benefited directly from the information given by forensic tests of specific structures. Very little if any repair products were used, and the information and instruction shown in the construction drawings was of a high standard in comparison to the other three contracts.

7.4 The Anatomy of a Concrete Repair Contract

Successful repair contracts require inputs and decisions at various levels throughout their timeframe, and ignoring or overlooking relevant factors will lead to poor repairs. The flow chart that is presented below represents the 'structure' of a repair contract, from inception to implementation.

STAGE	CONSIDERATIONS
IDENTIFICATION OF REPAIR NEED	*AVAILABLE BUDGET*
	GUIDED BY INDICATORS - MOSTLY VISUAL
	USE OF INSPECTION PROGRAMMES
ASSESSMENT OF DAMAGE	*FORENSIC TESTS, E.G. CHLORIDE PROFILING, CORROSION RATE MEASUREMENT*
	UNDERSTANDING OF DETERIORATING PROCESSES
	UNDERSTANDING OF EXPOSURE CONDITIONS
	RELEVANCE AND EXTENT OF DAMAGE
DESIGN	*REQUIRED LEVEL OF PERFORMANCE*
	CONSEQUENCES OF ALL OPTIONS - INCLUDING DO NOTHING
	LIKELY FUTURE PERFORMANCE OF REPAIRS
	CONFIDENCE/TRACK RECORD OF CHOSEN REPAIRS
	AVAILABLE BUDGET FOR/COST OF REPAIRS
	FUTURE USE OF STRUCTURE
	FUTURE REPAIR/MAINTENANCE PLANNING
IMPLEMENTATION	*ON-SITE QUALITY CONTROL*
	REPAIR TRIALS/ EARLY TESTING OF REPAIR QUALITY
MONITORING	*EVALUATION OF LONG TERM REPAIR PERFORMANCE*
	ADEQUATE MAINTENANCE PLANNING
	FUTURE ASSESSMENT FOR SPECIFIC DAMAGE

Figure 7-6: Repair Philosophy

7.4.1 Identification of Repair Need

The identification of the repair needs will most often be guided by the available budget for inspection programmes, and these programmes are based mainly on visual indicators. This was the case for the structures that were repaired under the RAMP programme. Key to this stage is the level of expertise of the person responsible this investigation. An understanding of common damage mechanisms is vital.

7.4.2 Assessment of Damage

Once these structures have been identified, a fuller assessment of damage must be performed: it is not enough to rely on the initial visual survey alone. The research in this thesis has shown the value of forensic testing in understanding the damage to structures, and also informing repair methodology. Where necessary external expertise (from researchers, product manufacturers, etc,) should be consulted in order to ensure a full understanding of the deleterious mechanisms. The damage to the structure must also be evaluated in terms of relevance and impact on future usage.

7.4.3 Design

Numerous factors must be considered fully in the 'design' phase, not least of which is addressing all the available repair options. In the RAMP repair contracts it seems the one option in particular, patch repair, was favoured. Each option should be considered in terms of practicality and likelihood of success. Invariably the budget of the programme will affect the level of repair that is implemented, but the effects of adopting less expensive methods should be assessed and presented to the client. Consideration must also be made for the future, addressing ways in which the repair methods can be tested for success and giving a prediction of future repair needs where possible.

7.4.4 Implementation

Once the work is put to hand on-site, it is important to be able to monitor the quality of the work. While two of the four contracts in this work have not opted for full time supervision, it is still something that should be mandatory on repair contracts, depending on size and complexity. The testing of repair works is also something that should be included as mandatory, wherever possible.

7.4.5 Monitoring

The repair works should be evaluated some time after implementation to assess the level of success. Regular inspection and maintenance should be performed on the structure.

Both figures show a number of important factors that are to be considered in all phases, particularly upon the 'decision' phase. One of the most important of these is the input that can be provided by the detailed assessment and understanding of the damage processes at work. Also prevalent in both figures is the need for the consideration of future requirements for repair and maintenance. The realistic performance of the repairs should be evaluated and accordingly factored into the recommendations made by the engineer.

Unfortunately the limited budgets that are currently available for infrastructure repair in this country are a limiting factor in the decision process. The adoption of a working strategy for repair will help avoid unnecessary spending. Thus an overview of the process such as is given in these two figures is suggested for consideration in planning and designing repair works to reinforced concrete structures.

One manner in which repair systems could be improved is by standardising the factors that are being used in decision making process. A predetermined set of checks that should be performed on the structure in order to fully assess the damage is already commonplace in engineering practice, for example with condition survey diagnostic sheets. A checklist approach to the whole process, however, would help to prevent the possible overlooking of critical aspects of damage, but also help to consider other aspects such as alternative repair options and future maintenance, ignored due to simple forgetfulness or even inadequate training and education.

At present it is common practice to use standardised inspection sheets for assessing damage, but there is no clear way forward from this point. Engineers are then left to use their own judgement to decide upon which tools, if any, are to be used to further investigate the structure to help gain a better understanding of the damage and necessary action.

It would be dangerous, however, to adopt a blanket approach and give recommendations for remedial measures from a single viewpoint. The nature of reinforcement corrosion in concrete, as has been discussed in detail throughout this thesis, shows that each location should be treated individually and uniquely. What is suggested, instead, is the use of a 'checklist' approach, in order to ensure that no possible areas have been neglected or overlooked. This would not be a governing document in a contract, as it is possible for this to be limited and outdated as new advances are made in the concrete field, but it is to be used as a suggested guideline for inspection, assessment and repair decision making.

The use of such a systematic approach also creates a transparency for the client to be able to see how the adopted repair philosophy has impacted the repair contract. For example if the philosophy is to apply minimal repair works to a structure at lowest expense in order to prolong it's life by five years, the level of assessment, forensic tests, choice of materials and repair methods, implementation on site and future maintenance and monitoring requirements will all be attuned to that philosophy.

Alternatively, if the repair philosophy incorporated an uncapped budget to repair/rehabilitate a structure for an indefinite lifespan (for example a historical structure), the implications of this would also effect all decisions throughout the repair project and not just in the adoption of more expensive methods of repair.

By adopting a repair philosophy at an early stage and understanding a repair contract from this 'strategy' overview, repair contracts can be managed more efficiently and decision making processes will benefit directly by revealing clearer repair objectives.

7.5 Conclusions

The objectives that were set in first chapter of this thesis have been fulfilled. The literature review presented in Chapter 2 gave detail regarding the fundamentals of the repair of reinforced concrete in the marine environment, and served as a base to which reference could be made when discussing the various repair strategies that had been performed at the four sites. Detail was included of investigative methods and repair strategies that were not adopted, such as Corrosion Rate Measurement and Cathodic Protection, but this was necessary to understand what other options were available but overlooked.

A review of the Contract Documentation was performed in chapter 4 and it soon became evident that while similar (if not identical) project specifications were used for the contracts, the difference in detail and description of the construction drawings existed between all four, even between two contracts involving the same company.

Chapter 5 presented a survey of the works performed at each site, and began to show the differences in approach: at some locations 'conventional' patch repairs were adopted, while at others innovative surface treatment combinations were tested, and at many locations demolition and reconstruction was favoured. Each contract summary commenced with a table showing the locations of the repair works, the type of repair, a description of the damage found, a description of the repair works performed and a summary of the relevant specifications.

Forensic tests performed at Saldanha Bay and St Helena Bay were presented in Chapter 6, and not only did these tests provide more information about the state of deterioration of the reinforced concrete elements, but they also gave guidance as to what repair methods would be most suitable. At Saldanha Bay the repair trial of a corrosion inhibitor in conjunction with other surface treatments was performed.

The results from these forensic tests were discussed and compared in the first section of chapter 7, after which the four contracts were critiqued and evaluated in terms of repair methodologies. This compared the differences in the four contracts at various stages, ranging from initial investigations to detailed design and ultimately execution of works. Marked differences were discussed with specific reference to improving current practice. The chapter closed with a brief summary of the four contracts, and presented an outline of a typical repair contract giving important considerations that must be addressed at various stages.

In general, the evaluation showed that there is room for improvement in current practice. There are inexpensive and relatively 'quick' forensic tests that can directly enhance knowledge and guide repair decisions that are not being utilised fully. The contract documentation does not fully address the individual contracts, but rather uses outdated specifications repetitively without incorporating the 'state of the art'. The repair methods chosen often seem to be a copy of existing repairs which have failed, and some repairs which have doubtful track records (such as localised patch repairs) are still being used regularly. Despite specific instructions in the contract regarding trial repair testing, this is not commonly found in practice. Thus it is felt that the recommendations made in the following section will improve current practice and certainly make repairs more durable and likely to succeed.

7.6 Recommendations

7.6.1 For Repair and Rehabilitation Contracts

- **Corrosion Inhibitor use – combinations**
  The testing in this thesis showed that the use of a corrosion inhibitor is improved in conjunction with a hydrophobic surface treatment. This allows the inhibitor to penetrate to a greater depth, in this work 50 -70 mm.
- **Cover depth**
  While specifying cover without accounting for binder type and other material properties is dangerous, the chloride analysis results in this work showed that in existing concretes (typically plain OPC), chlorides were able to penetrate in quantity (above 0.4% by mass of binder) to a depth of at least 50 mm within the serviceable lifespan of the structure. More research is needed in this regard, but the cover used for new reinforced concrete elements in the St Helena Bay harbour repair (75 mm) is encouraged. This value could be modified if blended binders were incorporated into the concrete mix design.
- **Failure of existing repairs**

Instances of failed repair works have emerged in this research, but similar methods are still being specified, in particular some patch repairs. Caution must be taken when prescribing such methods – the reasons for failure should be investigated and addressed in new repair methods. Simply replacing 'like' with 'like' without proper investigation and analysis is not sound engineering.

- **Contract Documentation**

  This research showed that the copying of standards across contracts was common, but there is a marked difference in the level of detail shown of the various contract drawings. It is recommended that all relevant information pertaining to repair works, products, material information etc. be included in the contract drawings.

- **The use of forensic testing in remedial measure specifications**

  The direct benefit of using forensic testing such as chloride profiling for informing repair decisions has been shown on numerous occasions. Not only are the tests capable of indicting the level of damage in a structure, in some instances guidance has also been given about which repair method to adopt.

- **Anatomy of a repair strategy/proposed checklist for repair projects**

  A checklist approach as well as the holistic adoption of a repair philosophy has been presented and is recommended for future contracts.

- **Combining existing and new elements**

  The effect of combining old and new concrete elements has not yet been fully assessed and care should be taking when assessing such 'composite' structures.

- **Trial repair tests**

  The tests performed at Saldanha Bay showed that it is possible even in locations with limited access to perform trial repair tests that inform repair processes and decisions. While mandatory testing is included in the contract documentation, the reality found on site is that it is seldom put into practice.

- **Performance based testing**

  The need for systems that allow the performance and success of repair methods to be quantified and assessed has been noted. The inclusion of such methods as payment items in contracts would ensure a higher quality end product. It could even be included as an incentive to the contractors, encouraging better workmanship.

- **Site supervision**

  The tendency of sites not to have a permanent resident engineer is evident in two of the four contracts. While the effect of such on-site control is hard to measure, the lack of such measures can only result in poor quality repair works.

7.6.2 For Further Work

- **Surface leaching**

  The possibility of surface leaching and the effect of chlorides and corrosion inhibitors to migrate out of concrete were suspected in tests performed in the work. Further investigation could prove useful in this field, in order to confirm the original assumptions.

- **Assessment of current repair works**

  An assessment of the performance of the current repair works, made at a later date (>10years) could prove extremely valuable in guiding current practice.

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9 Appendices (in CD format)

9.1 Saldanha Bay Harbour Forensics

9.2 St Helena Bay Harbour Forensics