PREDICTIONS OF REINFORCED CONCRETE DURABILITY IN THE MARINE ENVIRONMENT

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Cape Town, 1995.

DECLARATION

I declare that this thesis is essentially my own work and is being submitted for the degree of Doctor of Philosophy at the University of Cape Town. It has not been submitted before for any degree or examination in any other university.

Signed Signed by candidate

On 20th day of December 1995

ABSTRACT

This thesis presents an investigation where an empirical method is proposed for predicting the durability of reinforced concrete structures in the marine environment. The objective was to identify reliable means of characterizing early-age properties of concrete which affect durability and relate these to the durability performance of the material under marine conditions. Establishing a relationship between early-age testing and long-term performance of concrete is a necessary precursor to implementing a system of performance-based durability specifications.

A range of concretes was cast and cured in the laboratory before being characterized at 28 days using durability index tests which measure transport properties of ions and fluids through concrete. After 28 days the concrete specimens were exposed to the marine environment and the durability performance monitored over a period of two years. The durability performance was inferred primarily from the rate of chloride ingress since the risk of reinforcement corrosion may be related to the chloride level at the steel surface. Marine concrete structures of varying ages were also investigated to assess the long-term durability performance of concrete in service. Observations from structures indicated that chloride levels at the reinforcement were a good indicator of corrosion risk of steel. Further accelerated testing of concrete in the laboratory was undertaken to confirm trends observed on site with regard to corrosion activation and rates of chloride ingress.

A prediction model was proposed which allowed for different concretes types, construction practice, and the severity of exposure. The model was formulated using the relationship between concrete characterization results and medium-term durability performance determined from field exposure samples. Long-term chloride levels were predicted using a modified solution of Fick's Law of diffusion incorporating a reducing material diffusivity with time. The prediction model was validated using results from case studies of marine concrete structures and was found to be reliable and an improvement over existing methods.

Research indicated that existing durability specifications are inadequate due to inherent deficiencies such as the poor chloride resistance of OPC concrete, and because many specifications cannot be tested for compliance on site. A more rational approach using performance-based specifications was proposed which should improve concrete durability by quantifying the durability properties of concretes and assessing the quality of site concrete.

DEDICATION

I dedicate this thesis to my father, Professor W.R. Mackechnie who inspired me to pursue a career in Civil Engineering with his commitment and devotion for the profession. Some of his love for his calling is evident in an extract from his inaugural lecture delivered before the University of Rhodesia in 1975, reproduced below.

> "Nature presents the upper layer of the earth's crust for us to explore, to learn, to love it for the beauty of its consistency in some places, but more often for the capriciousness and variability of its engineering imperfections or shortcomings. The greater the bond of love between engineer and the environment, the more stable and secure the structure he sets out to create."

ACKNOWLEDGEMENTS

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CHAPTER ONE

INTRODUCTION

1.1 Background

The durability performance of building materials has long been a source of concern for builders and engineers. Building materials have evolved from natural media such as timber and stone to man-made materials such as steel and concrete which are not always as durable as their natural counterparts. The interaction of material and the environment has been understood to be the key regarding the deterioration process and Shakespeare noted that even the most solid of structures may not be immortal.

> "O how shall summer's honey breath hold out
> Against the wrackful siege of battering days
> When rocks impregnable are not so stout
> Nor gates of steel so strong but time decays"

Concrete is generally regarded as being chemically and dimensionally stable in most environments thereby possessing inherently durable characteristics. This perception is also associated with reinforced concrete structures which are expected to be relatively maintenance free during their service life. These assumptions are being increasingly questioned given the weight of evidence of premature deterioration of concrete structures, particularly in severe environments. While concrete structures exposed to moderate conditions may provide adequate serviceability without onerous specifications, special precautions may be necessary to ensure concrete durability in severe conditions such as the marine environment. It has been postulated however that premature failure of reinforced concrete structures in the marine environment is more often the results of poor quality concrete or bad design than an intrinsic lack of durability of the material [1].

The increasing number of concrete structures exhibiting unacceptable levels of deterioration has attracted widespread research attention in recent years [2,3]. Information flowing out of this research has increased the fundamental understanding of concrete as a material and the complex interactions between material, environment and structure which cause deterioration.

Durability specifications have become more stringent in response to these developments particularly with regard to grade of concrete, cover to reinforcement and crack width limits. Unfortunately the durability performance of modern concrete structures has not always shown a corresponding improvement. The problem would appear to be translating theory into practice by ensuring the requisite concrete quality has been achieved on site.

There is an increasing awareness that concrete is a complex composite material with variations, at both macro and microscopic level, which affect material properties such as strength and permeability [4]. The exterior layer of concrete displays greater variations due to drying and surface effects which cause a microstructural gradient of porosity. The interior concrete is more homogeneous but variations exist at aggregate and steel surfaces and from localized areas of unhydrated cement or the presence of calcium hydroxide crystals. Macro effects such as segregation, bleeding and cracking further complicate the complex structure of concrete. Quantitative methods for determining the pore structure of concrete are essential prerequisites for assessing transport properties of concrete which influenc the deterioration process.

Deterioration of concrete may begin almost immediately after casting as fresh concrete may be affected by plastic cracking, bleed voids and channels, segregation of mix constituents and thermal effects related to cement hydration. In the hardened state, concrete may be affected by a variety of internal and external factors which cause damage in the form of alkali aggregate reaction, steel corrosion, physical degradation and chemical attack. The interaction between deterioration mechanisms is complex and dynamic which complicates studies of concrete durability and requires multivariate analysis techniques.

Reinforced concrete structures in the marine environment are most susceptible to chloride-induced reinforcement corrosion due to high chloride concentrations and humid or saturated conditions [5]. Steel is passive in concrete due to the high alkalinity but depassivation may occur when the pore water pH is reduced by carbonation or when sufficient chloride ions are available to break down the passive layer on the steel surface. Corrosion may occur once the steel has been depassivated and favourable conditions of low resistivity and availability of moisture and oxygen in the vicinity of the reinforcement occur. Two primary forms of corrosion damage are observed: cracking and spalling of the cover concrete due to the expansive nature of corrosion products generated at the reinforcement, and localized pitting at the anode which reduces the cross-sectional area of the steel.

Given the nature of the corrosion process, a concrete must be able to either restrict the migration of chlorides to the reinforcement or retard the corrosion rate once depassivation of the steel has occurred. The corrosion rate may be reduced by restricting the access of oxygen to the cathode or by reducing the moisture content of concrete thereby maintaining a high resistivity. Reliable methods of retricting oxygen and moisture from concrete have not been developed and a durable concrete is assumed to require a high chloride resistance to provide adequate protection from the marine environment. A durable concrete may also be able to tolerate higher chloride levels at the steel without depassivation due to the presence of corrosion inhibitors.

1.2 Scope of Problem

Steel corrosion is the biggest threat to the durability of reinforced concrete worldwide and repairs are consuming increasing proportions of the total costs of concrete structures. Allen et al estimate that 60 % of worldwide construction expenditure is spent on assessment, repair and maintenance of structures with concrete structures making up an increasing component of the total amount [6]. In the United States the Federal Highway Administration estimates that the current repair bill for bridges and roads of deficient quality is now over $ 167 billion [7]. Poor design and construction of structures in the Middle East during the last twenty years has resulted in a legacy of damaged structures with limited serviceability [8]. The economic aspects of this remedial work are staggering and the flood of repairs shows little sign of abating.

The insidious nature of steel corrosion makes effective repairs expensive and sometimes impracticable if remedial work is only contemplated once damage occurs, since extensive chloride contamination may have already occurred. Repair strategies such as electrochemical desalination and cathodic protection may be effective in cases of severe deterioration while reliable diagnostic techniques have been developed which can can provide adequate fore-warning of impending deterioration. There is however a pressing need for techniques which are able to characterize the potential durability of concrete before and during construction. This will ensure adequate design and specifications and will enable any remedial work to be done before exposure thereby ensuring improved durability performance and reducing future repairs and maintenance.

The durability performance of marine concrete structures in South Africa has largely mirrored trends observed elsewhere with inadequate durability performance being fairly common. Modern structures appear to be condemned to the fate of earlier construction with many examples of rapid deterioration of structures built in the last fifteen years. Inadequate durability specifications and poor construction are thought to be largely to blame for the continued durability problems being experienced. A recent survey of marine concrete structures in the Western Cape revealed that many structures will require major repairs to achieve their original design lives (Figure 1.1) [9].

Several trends related to durability have been observed in response to the continued lack of durability of reinforced concrete structures in South Africa. The specified grade of concrete has increased steadily from 25 to above 50 MPa in some cases, galvanised reinforcement has occasionally been used, cement extenders have been recommended and cover depths of 60 mm or greater have been specified. Selection of these options has been rather haphazard and the technical motivation was often based on limited and often misleading laboratory findings. Economic imperatives often controlled the selection of concrete materials, resulting in regular use of cement extenders in the Western Cape during the 1970's and early 1980's and limited use thereafter when prices became restrictive due to transport tariffs. Cover depths to reinforcement have also reduced in some cases due to structural considerations or where crack widths were deemed to be the limiting criterion.

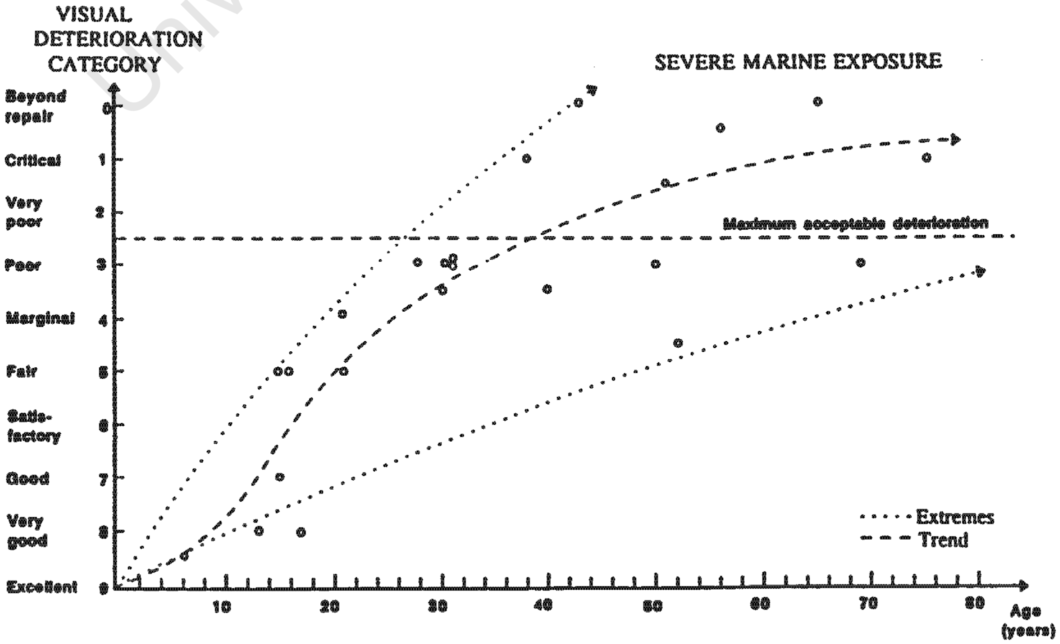

Figure 1.1: Visual Evidence of Deterioration versus Time [9]

Future trends in South Africa would indicate that considerably more infrastructural development will occur along the coastal regions. The harbour facilities in particular are poised for major expansions which will require a large investment in new concrete structures. Given the shortage of funds and economic constraints prevailing in South Africa these developments need to be carefully planned and well executed to ensure maximum use is made of all resources. Durable marine structures will be essential to this end and systems must be in place to ensure that a legacy of poor quality structures is not inherited by future generations.

1.3 Solutions to Lack of Concrete Durability

Many questions about concrete durability remain unanswered despite continued research interest in the subject [10]. Theoretical studies are still required to improve the understanding about deterioration mechanisms particularly when the results of short-term studies are used to predict long-term performance [11]. Current advances in fundamental understanding of concrete durability need to be disseminated to other researchers and converted into practical applications which may benefit the quality of site concrete.

Construction often favours empirical information based on practical experience rather than relying on theoretical information based on an understanding of the materials and mechanisms involved. Empirical information about durability is gained from current and previous experience, from which extrapolations are made about future performance. Extrapolations which do not have a sound theoretical base may be rigid and react poorly to construction changes associated with new materials and technologies. Care must be taken when using empirical methods that the limitations and assumptions inherent in the approach are well defined and critically examined for compliance. Empirical approaches to concrete durability may be regarded as interim solutions to the problem since real progress can only be made once there is a better fundamental understanding of the factors involved. Beeby argued that 'only the fundamental understanding of a problem offered by a properly constructed theoretical approach can offer a sound basis for moving forward into changing circumstances with any real confidence' [12]. Given the complexity of modern life and the rate of technological change this advice is becoming increasingly pertinent.

Intensive research has been done improving concrete repair strategies but the real issue to be addressed is how to improve the potential durability of new concrete structures and thereby reduce our reliance on repairs. Ensuring adequate durability at construction can only be done by guaranteeing that durability specifications are sufficient and complied with on site, contractual obligations are fully defined and binding, and economic imperatives do not compromise potential durability. Clearly it is impossible to ensure durability simply by having the correct technology in place but this remains the primary issue which must be addressed before other constraints are tackled.

Existing specifications are prescriptive in nature, often have little relation to durability and may be difficult to check on site. Cather stated that 'there is a philosophy in writing specifications which holds that that which cannot be checked should not be specified' [13]. This is certainly true of curing specifications which are difficult to check and routinely ignored on site. Some durability specifications such as concrete grade and cement content have been shown to be poor indicators of durability performance. Performance-based specifications where durability-related properties of concrete are specified appears to be a more rational approach to ensuring concrete durability [14].

The properties of the concrete protecting the steel have been identified as playing a crucial role in the durability of a marine structure. Dhir et al stated that 'the solution to the problem of the durability of concrete in structures is most likely to come about from an understanding of its outer layer or skin characteristics' [15]. Intrinsic properties determining transport of fluids and ions through concrete have been measured using laboratory techniques which may accurately model site conditions. These tests have limited practical application however due to extended test periods required and more rapid durability techniques have been proposed in response to this need.

Durability index tests which quantify the quality in the concrete near-surface layers at early age have been developed. Material properties affecting the transport of fluids and ions through concrete are measured which are related to concrete deterioration mechanisms. These material properties of concrete need to be related to long-term performance under defined environmental conditions so that predictions of durability can be made before significant deterioration occurs. Durability index tests can only be contemplated for use as performance specifications for durability when these methods have been shown to be reliable indicators of potential durability.

Predictions of durability are made by defining the material, assessing the environmental conditions and monitoring the durability performance of the material in that environment. Predictions are complicated by changing materials, service conditions and even the severity of environmental exposure. Given the complexity of the task, a broad framework needs to be formulated and the viability of the approach examined before detailed application is possible. The current trend towards developing early age durability tests will be of only limited academic interest until reliable predictive methods have been formulated and the implications of these tests fully realized.

1.4 Thesis Objectives

The objectives of this thesis are to formulate techniques whereby early-age material properties may be used to predict the potential durability of concrete in the marine environment. The predictive process needs to be practical in order to be used as part of a set of specifications for improving concrete durability. To achieve these objectives a relationship between early-age properties and durability perfomance needs to established. Suitable tests which are able to characterize relevant early age material properties must be defined together with methods of monitoring the durability performance of marine concrete in service.

The complexity of the interaction of material, environment and structure in the deterioration of marine structures militates against a comprehensive theoretical model. Such an approach may be possible in the future when substantive improvements in the fundamental understanding of concrete durability have evolved. Given the present limitations in knowledge an empirical approach is proposed in this work, based on sound theoretical principles. The predictions are based on the premise that deterioration is linked to the rate of ingress of chloride to the reinforcement and result in the subsequent activation of corrosion. Predictions are based on experimental data where characterized concrete was exposed to the marine environment and the medium-term durability performance monitored during exposure. The prediction model is then validated using long-term results from case studies of marine structures in service. A flow-chart showing the interlinked nature of the various components of the thesis which are used to formulate the prediction model is shown in Figure 1.2.

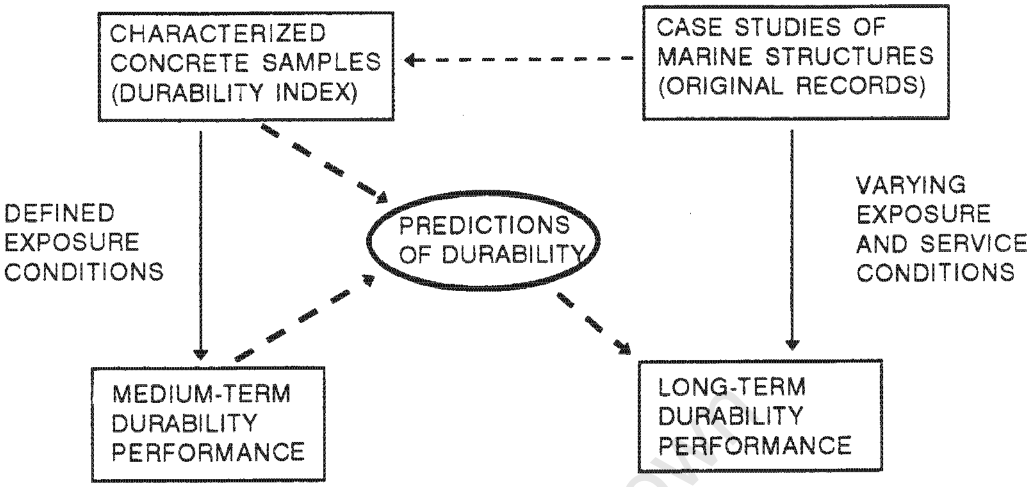

Figure 1.2: Flowchart Showing Components of Durability Investigation

1.5 Outline of Thesis Document

Before selecting techniques for characterizing concrete, factors affecting durability performance in the marine environment are reviewed (Chapter Two). The interaction of environment and material is investigated and important deterioration mechanisms discussed. The potential durability is defined in terms of transport properties such as absorption, permeation and diffusion which are relevant for marine concrete. Reliable durability index tests which characterize these transport parameters were selected and a range of concrete types and grades tested at 28 days and early age material properties characterized (Chapter Three).

The characterized concrete samples were exposed to a variety of marine environments after 28 days and the durability performance monitored over a period of two years (Chapter Four). Concrete panels were also exposed to different marine spray zone conditions to determine the effect of severity of exposure on the relative rate of chloride penetration into concrete. Relationships established between characterization results and the observed durability performance are expected to form the basis of predictions of durability. The relatively short length of exposure would produce only limited data of a short-term nature, so complimentary work was required to assess long-term trends.

Case studies of a range of marine concrete structures aged between three and seventy five years are presented (Chapter Five). The durability performance of the structures was assessed using standard diagnostic techniques and with durability index tests. A limited amount of accelerated laboratory testing was also done to determine specific aspects of durability such as corrosion initiation limits, coating systems and surface deposition of insoluble salts from sea water (Chapter Six). Data gained from investigation of laboratory and field samples were combined to formulate a prediction model which was validated using results from the case studies of marine structures (Chapter Seven). The empirical model was expected to be more reliable and broadly applicable than previous predictive techniques because allowance was made for different materials, construction practices and exposure conditions. The model is a preliminary proposal which can be refined and modified with further testing and validation.

Predictions of concrete durability should provide a rational basis for ensuring concrete durability which will have major economic savings for owners of marine concrete structures. The predictive model will allow materials to be optimized, provide a more flexible design approach and rationalize concrete design of marine concrete structures. Existing specifications may be improved and new technologies may be incorporated into construction rapidly for the benefit of all parties. An increased awareness of the complexity of concrete is required and suitable research and development done to ensure that the concrete becomes 'the advanced industrial material of the 21st Century' [16]

1.6 Definitions

Durability is defined as the ability of a material or structure to withstand the conditions of service for which it was designed over a prolonged period without significant deterioration.

Serviceability is defined as the capability of the material or structure to perform the functions for which it was designed or constructed.

Service Life is defined as the period of time in which a structure is deemed to have adequate serviceability.

Cover concrete refers to the outer surface layer of concrete which protects the reinforcement from corrosion.

Durability index testing of concrete refers to measurement of the resistance of concrete at early-ages to transport of fluids and ions through the cover concrete which affect deterioration mechanisms.

Durability performance of concrete refers to the extent to which concrete resists deterioration from environmental attack.

Durability potential is defined as the expected durability performance of concrete determined from early-age tests which quantify the concrete quality in the cover concrete.

Short-term refers to early ages of less than six months.

Medium-term refers to ages of between six months and five years.

Long-term refers to ages beyond five years.

Slag refers to ground granulated blastfurnace slag derived from steel processing plants

Fly ash refers to pulverized fuel ash derived from coal fired thermal power plants

OPC refers to ordinary or normal portland cement

Silica fume refers to the powder condensate resulting from the production of silicon or silicon-based metals alloys.

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CHAPTER TWO

CONCRETE DURABILITY IN THE MARINE ENVIRONMENT

2.1. Introduction

The sea is characterized by saline water, tidal action, waves and the presence of marine organisms. While these characteristics are common to all seas and oceans their relative contributions and intensities vary significantly with location, season and time. The marine environment needs to be classified carefully taking into account local variations of salinity, temperature, tidal action, currents, winds and wave action in order to determine the severity of exposure at a particular location. Detailed classification of marine exposure is essential since conditions vary widely between different marine sites.

Whilst salinity values may vary considerably the chemical composition of seawater is fairly consistent in the open oceans; the average chemical composition for different seas is detailed in Table 2.1 [1]. Sodium and chloride ions form the major component of seawater with magnesium and sulphate ions also occurring in significant concentrations. The pH of seawater is normally between 8.2 and 8.4 but may be lowered to below 7.0 by high concentrations of dissolved carbon dioxide and hydrogen sulphide from marine organisms in sheltered bays [2].

Table 2.1 : Average Chemical Composition of Seawaters [1]

Ion	Concentration (g/l)			
	North Sea	Atlantic Ocean	Persian Gulf	Baltic Sea
Sodium	12·20	11·10	13·10	2·19
Magnesium	1·11	1·41	1·48	0·26
Potassium	0·55	0·40	0·67	0·07
Calcium	0·43	0·48	0·50	0·05
Chloride	16·85	20·00	23·00	3·96
Sulphate	2·22	2·81	4·00	0·58
Total	33·36	36·2	42·75	7·11

The salinity of seawater in the open oceans varies between 34 and 37 g/litre with lower values in polar regions and higher values in tropical areas. The surface salinities of the oceans during a Northern Summer are shown in Figure 2.1 [3]. Closed seas such as the Mediterranean and Baltic seas are more affected by local conditions such as water runoff from the land and evaporation rates giving rise to significantly different salinity values. The salinity of seawater along the coastline of South Africa is consistently between 35 and 36 g/litre but may be considerably lower in tidal estuaries and river mouths.

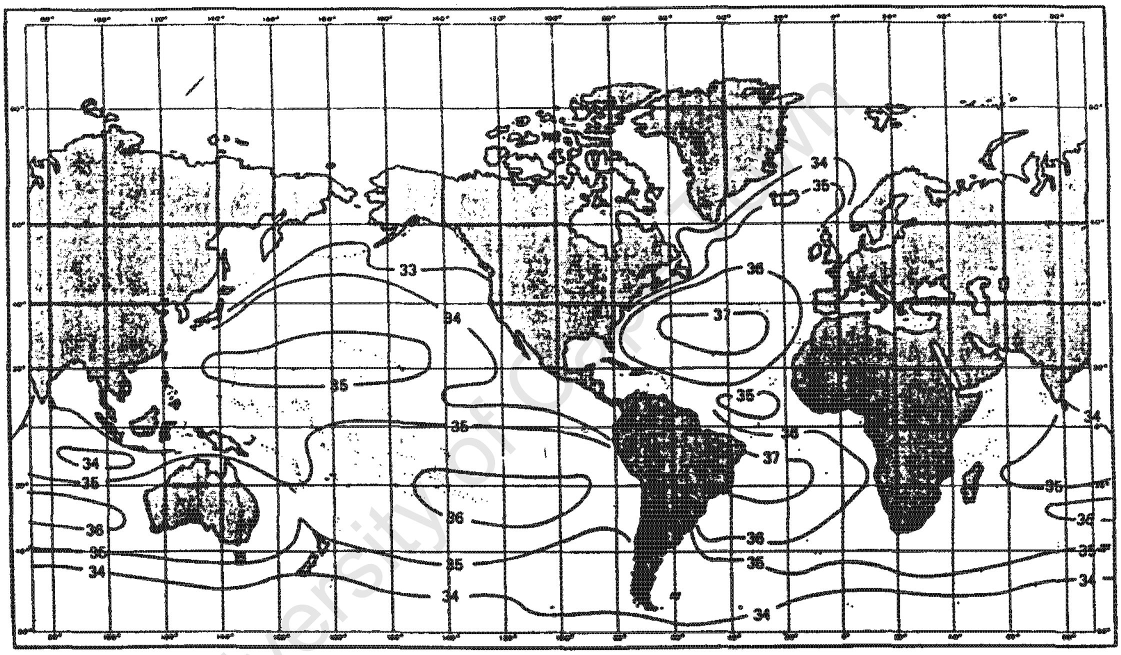


Figure 2.1 : Surface Salinity in the Oceans in the Northern Summer (g/litre) [3]

The surface temperature of seawater varies from a low of -2 °C in cold regions to 30°C in tropical climates. Seawater temperature varies with depth, season, winds and currents so that many areas may experience large temperature fluctuations. In colder climates freeze-thaw action is prevalent while hot climates increase reaction rates of chemical processes in seawater. In South Africa the presence of the warm Agulhas current down the Eastern seaboard maintains warm water temperatures of above 20 °C while the colder Benguela current moving up the West coast keeps sea temperatures below 15 °C. Cape Point is generally regarded as being the boundary between the Benguela and Agulhas currents. This results in sea temperatures being on average 6 °C colder on the Atlantic side of the Cape Peninsula than in False Bay [4]. Figure 2.2 shows a satellite image of surface sea temperatures off the South African coast, showing clearly the effect of the cold and warm currents [5].

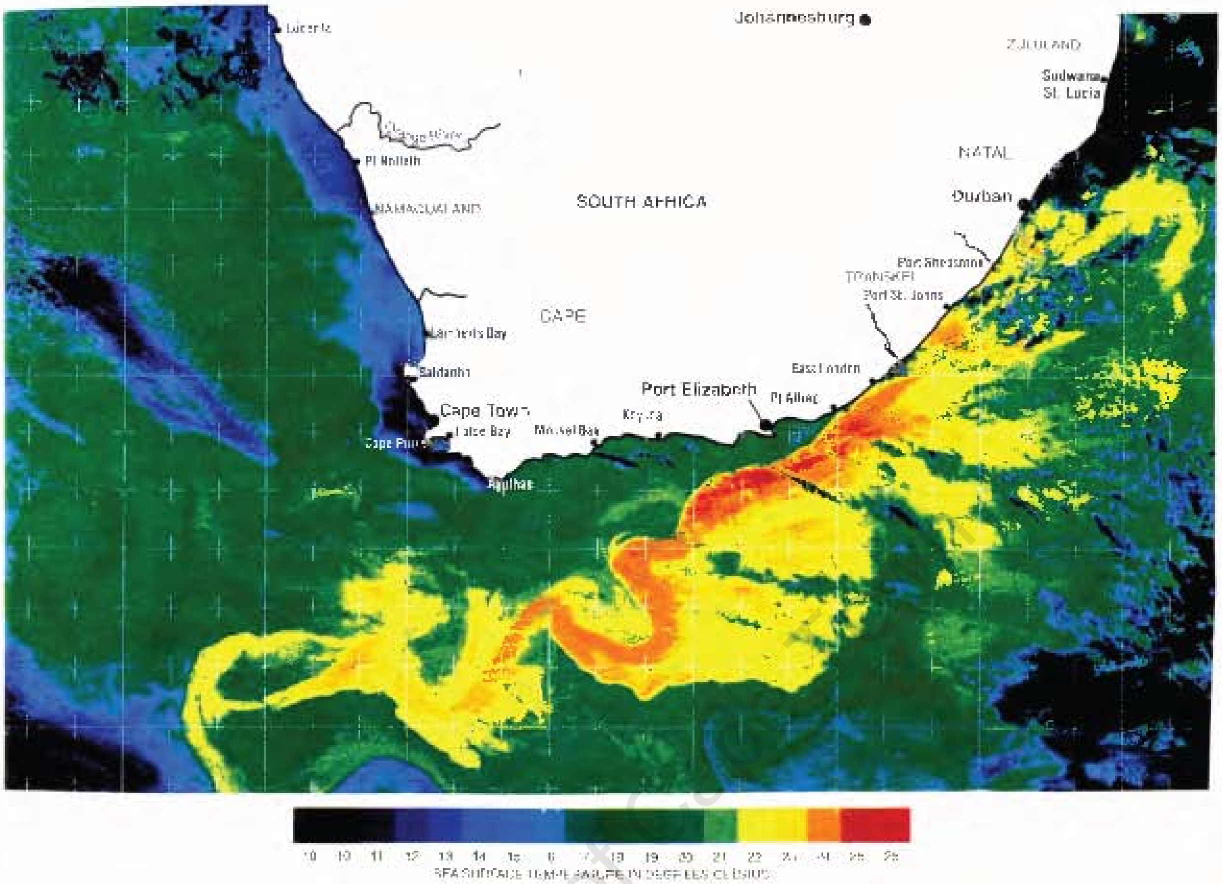
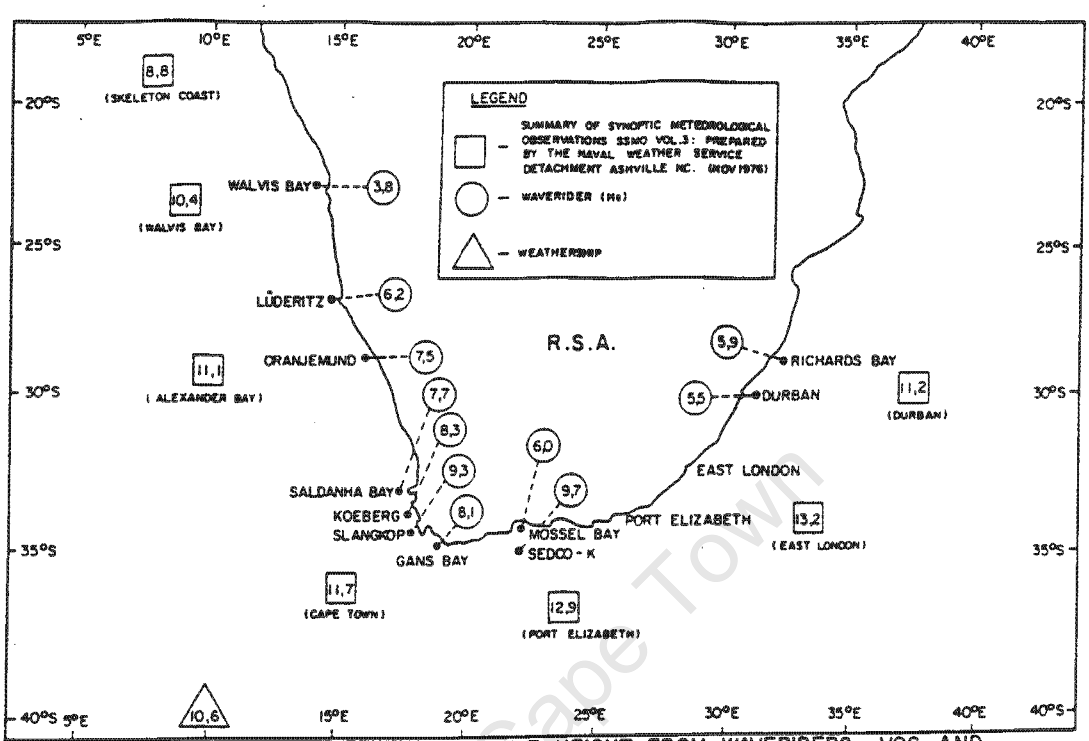


Figure 2.2 : Satellite Image Showing Sea Temperatures Off South Africa [5]

Tidal action occurs in all oceans and seas but the amount of tidal range is determined by the size, depth and openness of the sea. Some seas have tidal ranges of less than 0.5 m while estuaries such as the Bay of Fundy has a tidal range of 15 m. In South Africa the tidal range averages 1.2 m due to open oceans surrounding the subcontinent.

Wave action and the effect of storm waves depend on local conditions such as coastline shape, weather systems, currents and sea bed topography. Some coastal areas experience mild conditions with occasional severe storms while other areas are exposed to regular rough weather with heavy seas. The South African coastline is generally exposed to severe wave action with the region between Cape Point and Cape Agulhas being particularly prone to heavy seas. Numerous shipwrecks along this stretch of coast testify to the ferocious nature of the sea with the predominant swell being from the South West across the South Atlantic Ocean. Figure 2.3 shows the wave climate off the South African coast with peak wave heights being recorded for various coastal locations and at sea [6]. Interactions between the South Westerly swells and offshore currents can produce extremely large storm waves.



DISTRIBUTION OF 0,01 PERCENT WAVE HEIGHT FROM WAVERIDERS, VOS AND WEATHERSHIP

Figure 2.3 : Wave Heights Along the South African Coast (m) [6]

Wind is a major factor in the marine environment, being influenced by the interaction of sea and land weather systems along the coast. Onshore wind causes splashing and drives salt-laden air inland several kilometres under severe conditions. Wind also causes rapid drying which in combination with tidal and splash action may cause severe wet-dry cycling. The Cape is characterised by strong winds in summer from the South East and strong and gusty winds from the North West in winter.

Seawater also contains varying amounts of marine organisms such as barnacles and mollusks which can damage porous surfaces by secreting acids and boring holes. Organic growth on marine structures may increase the hydaulic loading by increasing the drag forces and may also affect the serviceability of some structures. The sea is characterised by physical aggression from ice impact, particle abrasion and damage from shipping on harbour structures. All these factors make the marine environment a very aggressive place for materials and Civil Engineers have learnt after a history of failures that special precautions are required for durability in the marine environment.

2.2. Marine Deterioration of Concrete

Classifying the causes of concrete deterioration into separate categories is done in an attempt to simplify the complex nature of the interaction of the marine environment with concrete. Research has shown that deterioration of concrete is almost always the result of several interdependent chemical and physical processes which do not occur in isolation. Figure 4 shows the common forms of deriorations which can affect marine concrete structures and some of the important processes are discussed in more detail below [7].

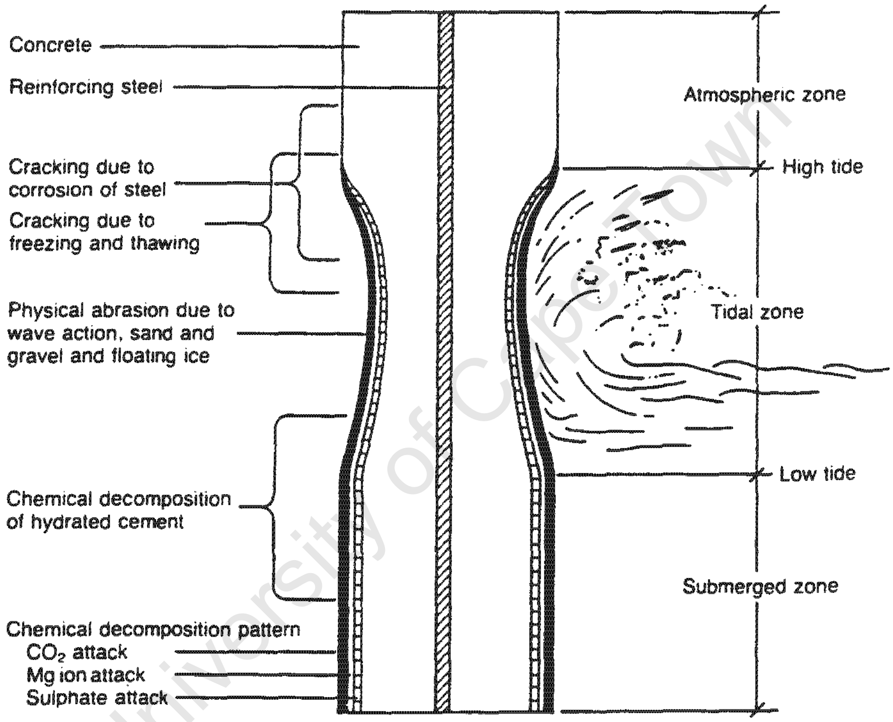


Figure 2.4 : Common Forms of Marine Concrete Deterioration [2]

2.2.1 Wave action

The surface of the sea is constantly moving and changing in response to the forces of nature. Waves are driven by the wind but may also be affected by tidal surges, ocean currents and abnormal weather. Storm waves can cause great destruction and breaking waves can have an impact pressure of up to 300 kPa [2,8]. The direct impingement of waves on concrete generates high local pressures at the surface causing physical damage. Marine structures such as breakwaters, sea walls and other coastal defences bear the brunt of wave action and localised damage and stability failure is fairly common. Coastal structures need to be carefully designed using sound hydraulic and coastal engineering principles to be durable and provide adequate serviceability.

High energy waves may cause physical damage to concrete in the tidal and splash zones from the action of suspended particles which abrade the concrete surface. Abrasion damage is usually confined to the upper tidal and splash zones and may lead to failure of slender elements if not repaired in good time. Good quality concrete is generally able to resist physical attrition from wave action but once other deterioration processes such as chemical attack have reduced the integrity of the material significant damage may take place. To resist the effects of physical damage from the sea requires a dense surface layer produced using a high strength concrete which has been well cured. Under exceptional conditions a sacrificial layer may be used on marine concrete structures to allow a limited amount of surface damage to occur. Some codes of practice recommend sacrificial layers of up to 300 mm to be used for extreme cases of abrasion [9].

2.2.2 Chemical deterioration

Chemical attack of concrete is caused mainly by the magnesium salts in seawater (magnesium sulphate and magnesium chloride). The deleterious effect of magnesium sulphate on concrete was first noted by Vicat in 1818 [10]. Magnesium sulphate reacts with calcium hydroxide of the cement paste to produce magnesium hydroxide and calcium sulphate. The calcium aluminates in the cement paste also react with magnesium sulphate to produce ettringite which is expansive in most sulphate environments. Sulphate attack in seawater is not characterised by the formation of expansive reaction products but rather by soluble products which are leached out of the concrete. The higher solubility of calcium sulphate and calcium sulphoaluminate in chloride solutions together with the action of wave and tidal movements is thought to reduce the risk of expansion [11]. It has been suggested that the ettringite expansion is suppressed in the marine environment where Cl^- ions have essentially replaced OH^- ions [12]. Magnesium sulphate decomposes the hydrated calcium silicates in addition to the calcium aluminates and calcium hydroxide by an ion exchange reaction so that eventually the entire cement matrix may be broken down.

Chemical attack of concrete may be seen as the combined effect of several reactions which proceed simultaneously. Different reactions occur with depth into the concrete; at the surface carbonation and magnesium ion exchange with calcium are prevalent while deeper in the concrete sulphate attack is more common. Leaching of cementitious products is possible due to their sparingly soluble nature which continues the chemical attack and may eventually compromise the integrity of the material. Concrete in the submerged zone suffers only minor chemical attack as leaching can only occur by diffusion which is a relatively slow process. Chemical attack is accelerated in the tidal zone where leaching is assisted by tidal and wave action which wash out soluble reaction products.

Some of the reaction products of seawater attack on concrete may be beneficial to concrete durability due to their low solubility which helps block pores from further seawater ingress. Research has shown that a protective layer may be formed on the concrete surface due to the deposition of magnesium hydroxide (brucite) and calcium carbonate (aragonite) [13]. The protective layer of brucite and aragonite on the concrete surface is usually less than 100 micron thick and may be disrupted by wave action and abrasion.

2.2.3 Steel corrosion

Steel corrosion is the dominant mechanism causing deterioration of reinforced concrete in the marine environment [14]. Corrosion of steel is complex and is affected by ionic concentrations, pH of solution, temperature, steel composition, internal stresses, stray currents and electrolytic potentials. Steel is thermodynamically unstable under normal environmental conditions and also under alkaline conditions found in concrete (pH = 12.5 to 13.5). Steel embedded in concrete is initially passivated by the formation of a dense oxide layer. Under favourable conditions (pH values below 11.0) the parent metal will oxidise to its most stable state as ferric oxide. The transformation of metallic iron to rust is associated with an increase in volume causing cracking and spalling of concrete. Pitting corrosion reduces the cross-sectional area of the rebar and thereby lowers the load-carrying capacity of the structure.

Corrosion of steel in concrete is an electrochemical process which is driven by concentration differences of ions and gases in the vicinity of the metal. At the anode metal molecules lose electrons to form metal ions which pass into solution while at the cathode electrons generated from the anode pass from the metal to some chemical species in solution adjacent to the cathode. The type and severity of corrosion is determined by local conditions at the reinforcement and by external influences. Four states of corrosion of steel in concrete have been defined by Arup: the passive state, pitting corrosion, general corrosion and active, low potential corrosion [15].

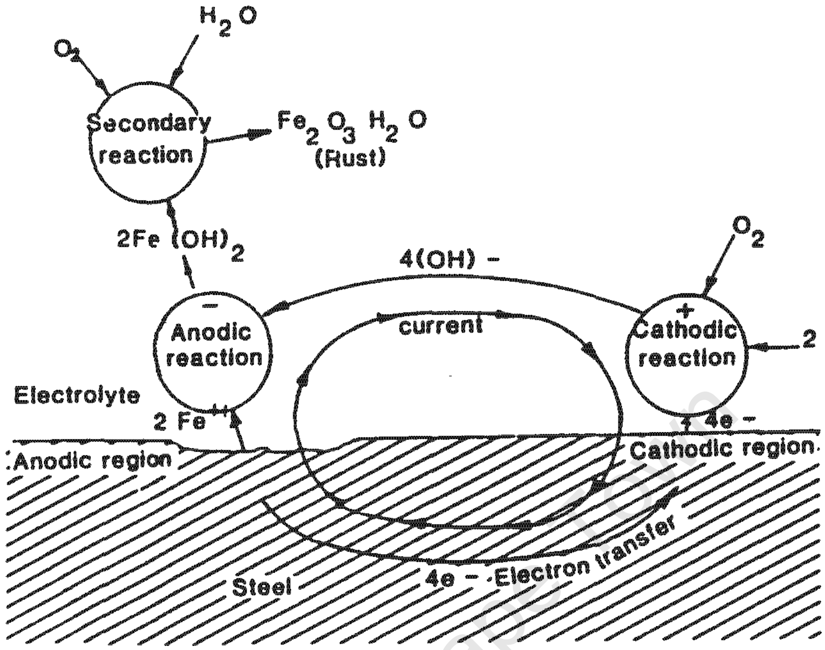


Figure 2.5 : Idealised Corrosion Diagram [17]

Passivation occurs in uncarbonated concrete where the anodic steel surface becomes coated with a thin protective layer of ferric oxide. This oxide layer, often referred to as gamma ferric oxide, is stable and acts as a protective coating to the steel so that corrosion virtually stops. Chloride ions appear to be able to break down this protective layer when their concentration reaches a certain threshold level which depends on the type of cement and the pore water solution. A reduction of the alkalinity of concrete surrounding the reinforcement due to carbonation may also destabilize the ferric oxide layer allowing corrosion to occur.

Pitting corrosion results from a local breakdown of the passive film at the steel surface usually due to the presence of chlorides. Adjacent passive steel acts as the cathode, with the cathode area being considerably larger than the anode, and the anode dissolves to produce a pit. The rate of corrosion is limited by polarization of the cathode, the concrete resistivity and the availability of oxygen. Figure 2.5 shows the idealized corrosion process.

General corrosion results from a general loss of passivity due to carbonation or the presence of excessive amounts of chlorides. This produces a large number of pits close to each other which generate solid rust and cause cracking and spalling of concrete. Where carbonation is the primary cause the rate of corrosion is governed by the presence of moisture whereas excess chloride is more likely in wet concrete and the rate of corrosion is therefore governed by the availability of oxygen.

Active, low potential corrosion is very slow and occurs when passivity is lost due to lack of oxygen in a strongly alkaline concrete. Pitting cannot occur due to the low potentials and the corrosion rate is very low resulting in no distress to the concrete. These conditions generally occur in concrete which is under continuous saturated conditions with negligible amounts of oxygen diffusion.

The interaction of different materials and electrolytes in reinforced concrete gives rise to electromotive forces or potentials between anode and cathode. The corrosion process causes oxidation to take place at the anode and reduction at the cathode with a resultant electron transfer from anode to cathode. The primary corrosion reactions at the anode are [16]:-

$$Fe \text{ ----> } Fe^{2+} + 2e^- \qquad (2.1)$$

$$Fe + 3H_20 \text{ ----> } Fe(OH)_3 + 3H^+ + 3e^- \qquad (2.2)$$

$$3Fe + 4H_20 \text{ ----> } Fe_3O_4 + 8H^+ + 8e^- \qquad (2.3)$$

$$Fe + 2H_2O \text{ ----> } FeO(OH)^- + 3H^+ + 2e^- \qquad (2.4)$$

The type of reaction depends on the potential and the pH of the electrolyte and may be estimated from the following simplified Pourbaix diagram [17].

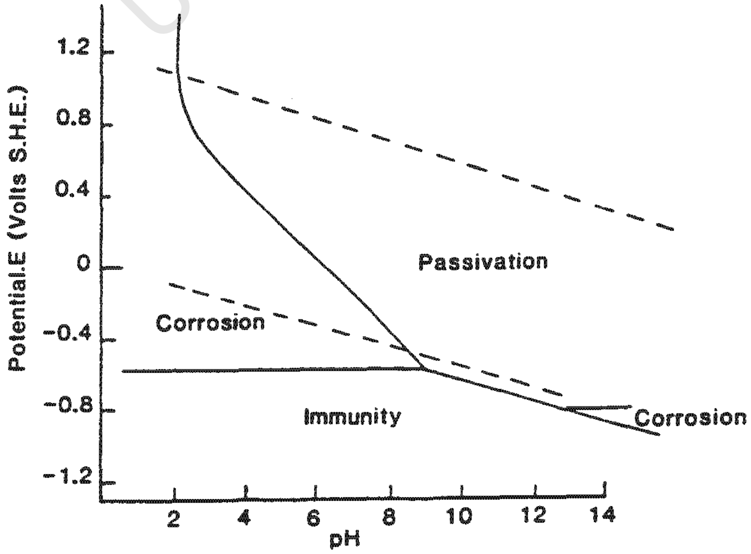


Figure 2.6 : Simplified Pourbaix Diagram

Carbonation refers to the reaction of atmospheric carbon dioxide with calcium hydroxide and other hardened cement phases to form various carbonate minerals. Cement paste constituents such as calcium silicates are carbonated simultaneously with the more soluble calcium hydoxide but the process is relatively slow even under optimum conditions at 65 % R.H. Carbonation is not detrimental to concrete directly but the lowering of the pH from around 13 to 8 by the process is sufficient to destabilise the passive layer surrounding embedded steel thereby allowing steel corrosion to occur [18].

The passive ferric oxide layer on the steel surface may be disrupted when the chloride concentration reaches a level referred to as the corrosion threshold level. The concentration of chlorides required to exceed the corrosion threshold depends on the cement type, steel type, pore water pH and other local conditions. Empirical chloride corrosion threshold levels have been determined by researchers but these are only appropriate for a limited set of conditions and only provide a rough guide for assessing the risk of corrosion [19]. Chloride ions appear to act as catalysts in the disruption of the passive layer and are recycled so that they can be used elsewhere along the steel (Figure 2.7). Increases in the chloride concentration above the corrosion threshold only have the effect of increasing the rate of corrosion by reducing the resistivity of the concrete.

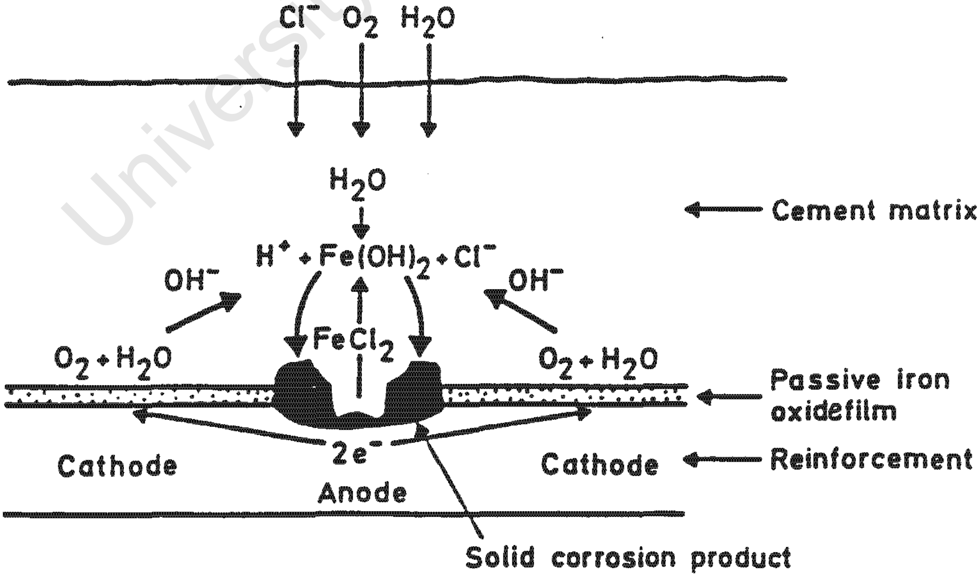


Figure 2.7 Chloride-Induced Reinforcement Corrosion

The exact mechanism of chloride disruption of the passive oxide layer on the steel is not fully understood but three theories have been proposed [20]. The complex formation theory suggests that chloride ions form a soluble complex with the ferrous ions which then diffuse from the anode. Decomposition of the complex molecule precipitates the iron and allows the chloride ions to be recycled [21]. The oxide film theory postulates that chloride ions penetrate the oxide layer which surrounds the steel through defects in the film, thereby accelerating the transport of ferrous ions away from anodic sites. The adsorption theory proposes that chloride ions are adsorbed preferentially on to the metal surface in competition with dissolved oxygen and hydroxyl ions. The high reaction rate of metals with chloride helps produce soluble species of iron and chloride which facilitate anodic dissolution.

Damage from reinforcement corrosion is generally associated with disruptions to the cover concrete due to the expansive products of corrosion although localised pitting may cause severe loss of cross-sectional area of rebar. Clear proposed a two phase corrosion model with an initiation phase where little damage occurs followed by a propagation phase in which damage occurs once the corrosion threshold level has been exceeded. This simplistic model has been developed by researchers such as Tuutti and Miyagawa in an attempt to predict the service life of concrete structures [22,23]. Miyagawa developed a three phase model which assumes that the corrosion process is accelerated once major cracking and spalling occurs since chlorides moisture and oxygen have easy access to the reinforcement. Details of the Miyagawa model, showing a typical example, are given in Figure 2.8.

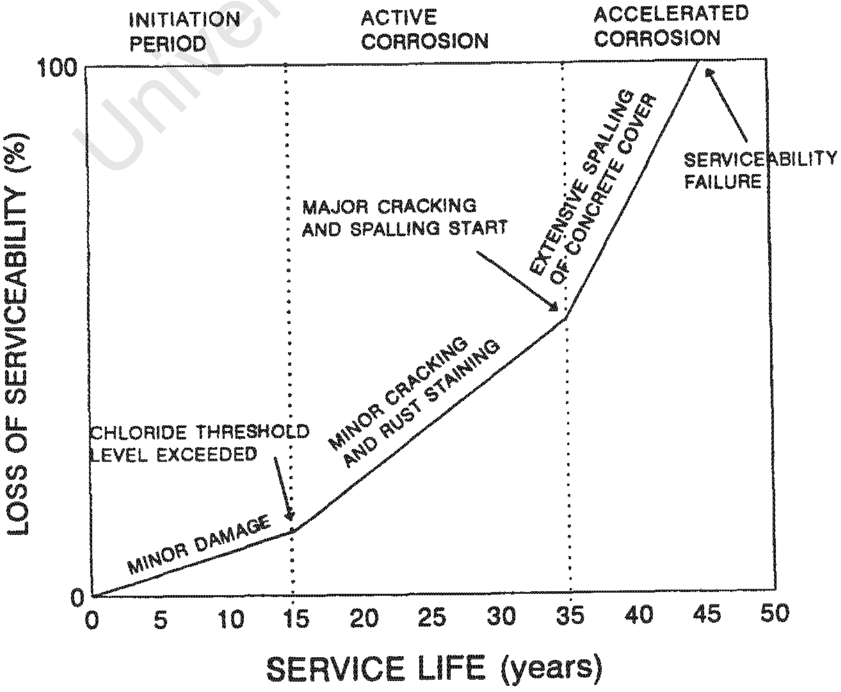


Figure 2.8 : Example of Corrosion Damage Model [23]

Experience from marine concrete structures indicates that corrosion of reinforcement is most prevalent in the splash and spray zones. Corrosion activity decreases rapidly with distance below the high tide level due to the saturated nature of the concrete which limits the availability of oxygen. Optimum conditions for corrosion are generally located in the splash zone where both oxygen and moisture are available and where chloride penetration is accelerated by regular wetting and drying action at the concrete surface. The risk of corrosion decreases in the spray zone where conditions are generally drier and impedes the electrolytic proceess by increasing the concrete resistivity. Submerged concrete has a low risk of corrosion except for very porous concrete where macrocell corrosion has been observed.

2.2.4 Temperature effects

Temperature affects marine concrete by either frost action, cracking due to excessive heat generated by cement hydration and thermally induced cracking during construction and during the service life of the structure. Frost action from repeated freeze-thaw cycles can disrupt the concrete microstructure particularly with marine structures where tidal action may cause several hundred freeze-thaw cycles each year. Water freezing in concrete capillaries increases in volume by 9 %, forcing excess water into the surrounding finer capillaries and gel pores. The resulting build-up of pressure is sufficient to disrupt the gel system. Frost action is aggravated by the presence of salt in the concrete due to phase changes, higher hydraulic pressure and lower freezing temperature of salt solutions [24]. Damage from thermal shock may also occur on some marine concrete structures where temperature differentials of up to 30 °C are possible.

2.2.5 Salt crystallization

Salt crystallization in the capillary pores of concrete is generally caused by high evaporation rates in hot dry climates. Rapid evaporation from the concrete surface increases the pore water concentration resulting in a super-saturated solution which on drying produces crystallization pressures on the surrounding concrete. Salt crystallization pressures are also affected by temperature variations and liquid phase changes where pore water dries at a temperature above the liquid phase transition followed by wetting at a lower temperature resulting in the formation of crystalline hydrates with a larger volume than the anhydrous salt.

Damage caused by salt crystallization is usually restricted to the concrete surface where drying is taking place. Partially immersed structures such as jetty piers can suffer internal damage from salt crystallization where there is continuous water movement from the saturated zone to the drying face.

2.2.6 Alkali aggregate reaction

The expansive reaction of alkali reactive aggregate with cement paste is complex but basically involves the formation of a gel at the aggregate-paste interface. The gel attracts water molecules causing high pressures to develop [25]. These pressures may result in disruptions to the concrete pore structure and may ultimately affect the overall integrity of the material. The marine environment accelerates alkali aggregate reaction since sodium and chloride ions are known to be catalysts in the reaction and the high humidity close to the sea provides ideal conditions for the reaction [2]. Extensive research of the problem has enabled designers to eliminate the risk of alkali aggregate damage of marine concrete by careful selection of materials and correct concrete mix design.

2.3. Factors Affecting Concrete Durability in the Marine Environment

The durability of concrete may be defined as the ability of the material to withstand the environmental conditions to which the concrete is exposed without unacceptable deterioration. Investigations into durability are complicated by the complex and dynamic interactions between material and environmental effects. The durability performance of concrete is influenced by fluid transport properties of the hardened concrete since deterioration is dependent on the ingress of aggressive agents and the chemical interactions that ensue. Some of the main influences which have an effect on durability are discussed in more detail below.

2.3.1 Microstructure and properties of concrete

Concrete is a composite material consisting of coarse and fine aggregate in a matrix of cement paste which acts as the binder. Cement is alkaline by nature consisting of various phases of calcium silicates and aluminates which react with water to form a hardened cement paste. Macroscopically, concrete may be considered to be a two phase material consisting of aggregate particles dispersed in a matrix of cement paste. At the microscopic level however the paste is not homogenous, with variations caused by transition zones at aggregate surfaces and changes at concrete surfaces.

The concrete skin is composed of several layers, an outer cement skin of about 0.3 mm, a mortar skin of about 5 mm and the "curing affected zone" which extends up to 50 mm from the concrete surface [26]. A microstructural gradient of porosity develops which is affected by the permeation of water and evaporation from the concrete surface. Drying may lower the relative humidity in the pores sufficiently to stop cement hydration resulting in a coarse pore structure developing in the covercrete zone. The interior concrete is generally more homogenous but variations exist at aggregate and steel surfaces and from localized areas of unhydrated cement or the presence of calcium hydroxide crystals. Macro effects such as segregation during compaction, bleeding of concrete and cracking further complicate the complex structure of concrete.

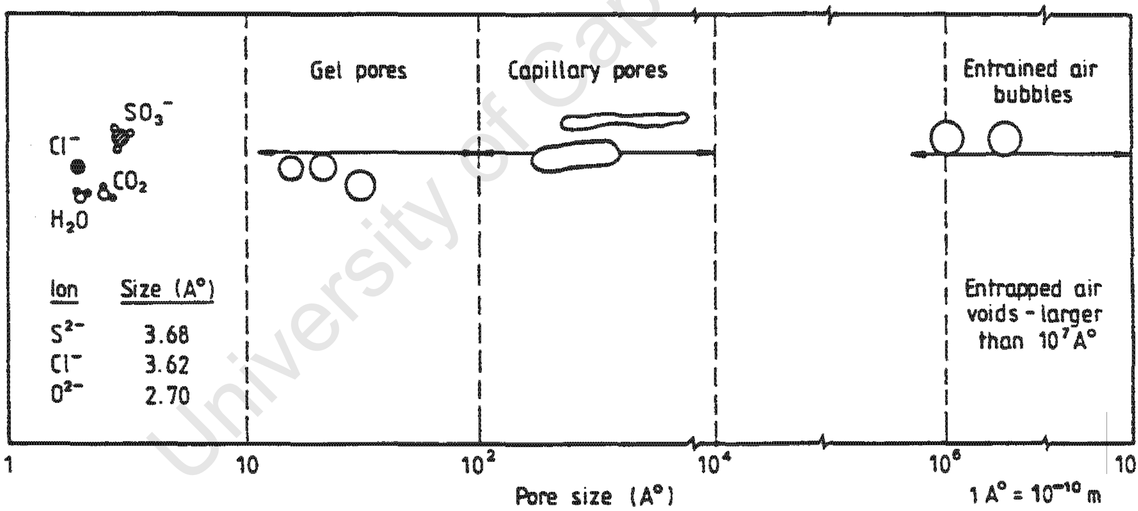


Figure 2.9 : Pore Size Distribution of Concrete [27]

The hardened cement paste is considerably more porous than the aggregate and controls the permeation of fluids through the material. The voids vary from interparticle spaces in the gel of less than one nanometer to entrapped voids of over one millimetre. Capillary voids are formed by water channels in the freshly cast concrete that dry out leaving air-filled voids. The size and interconnected nature of the capillary voids determines the resistance of the concrete to the permeation of gases and liquids. The pore structure is therfore fundamental to the durability of concrete. Figure 2.9 shows the size and range of voids found in the concrete pore structure [27].

The hardened cement paste is chemically stable in most environments and is virtually insoluble in water. The alkaline nature of concrete does however make it vulnerable to leaching from soft and acidic waters which dissolve the solid constituents. Leaching of calcium hydroxide is detrimental to the concrete as the hardened cement paste becomes unstable when the pore water pH drops and decomposes to release more calcium hydroxide into solution. Under extreme conditions of leaching such as in concrete pipes carrying soft or acidic water, complete failure of the cement matrix can occur from decomposition of most of the hardened cement paste.

2.3.2 Cement types

Concrete is usually designed for a desired consistency and workability to allow effective placing and compaction and also for a specified compressive strength to withstand the service loads. Compressive strength was a convenient measure of concrete quality as it gave an indication of likely structural performance and, it was thought, durability performance as well. Early work by Powers showed that decreasing the water/cement ratio had a dramatic effect in reducing the permeability of concrete [28]. Similar work by other researchers led to the acceptance of strength as the main criterion for specifying concrete for both structural and durability performance.

The emphasis on strength for specifying concrete quality has been reassessed after widespread lack of durability performance of concrete structures. The introduction of new technologies leading to finer, quicker hydrating cements, the use of admixtures and poorer site control all contributed to decreasing the potential durability of modern concrete for the same specified strength [29]. The production of a dense, impermeable concrete is enhanced by using a low water/cement ratio but the choice of cement type is critical particularly in the marine environment.

Cements and cement extenders vary considerably in chemical and minerological composition. These variations have a major effect on the properties of concrete and the potential durability. Figure 2.10 shows the ternary diagram for the $CaO\text{-}SiO_2\text{-}Al_2O_3$ system and the relative location of various cementitious binders in that system.

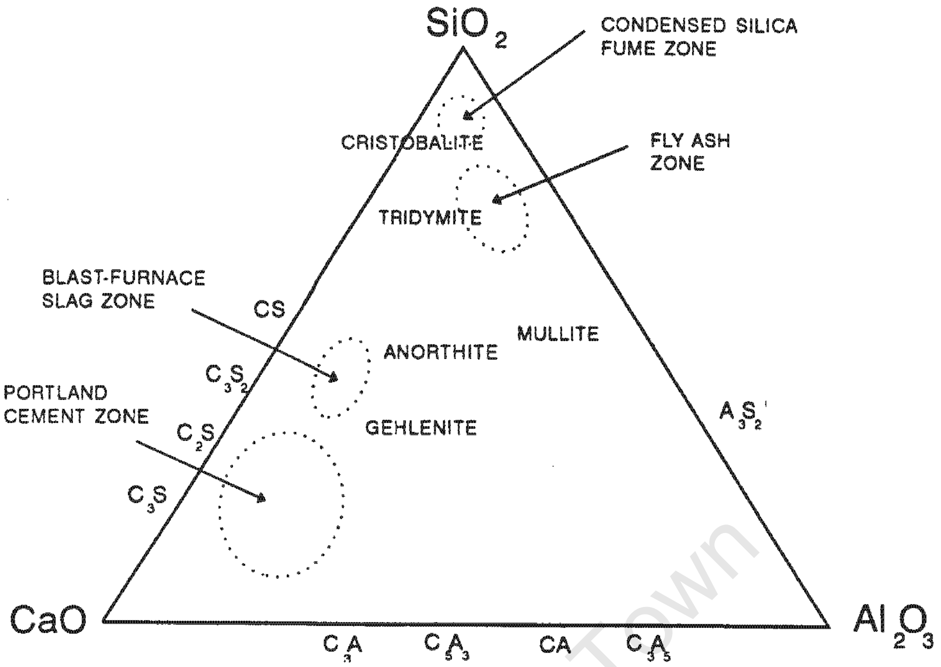


Figure 2.10 : Ternary Diagram for CaO-SiO_2-Al_2O_3 System

The performance of OPC concrete is dependent on the chemical composition and fineness of the cement. Variations in the chemical composition of cement occur due to different raw material used and the manufacturing process. Hydration of cement, well described in the literature, helps seal the initially water-filled interconnected pores within the cement paste. Cement hydation is rapid at first and tapers off after between three and seven days particularly if significant drying occurs at the surface. Dense and impermeable concrete can be produced using OPC but a low water/cement ratio is essential together with good compaction and curing. Whilst new cement types are being developed OPC concrete still remains the benchmark against which new cements are measured.

The pozzolanic reaction of fly ash with calcium hydroxide in concrete produces a dense concrete with a refined pore structure [30]. Fly ash concrete is generally less permeable than OPC concrete particularly in moist conditions when long-term hydration is possible. The reported durability performance of fly ash concrete is conflicting. Carbonation appears to be unaffected by fly ash in concrete except when poorly cured while chloride ingress through fly ash concrete is reduced significantly [31]. Fly ash is useful for reducing the risk of alkali aggregate rection and for reducing the heat of hydration in large concrete pours. The pozzolanic reaction of fly ash depletes hydroxyl ions from solution. The reduction of hydroxyl ions may be seen as a potential problem to maintaining steel passivation but no evidence has been found to support this premise. Dhir et al found that fly ash concrete has similar chloride corrosion threshold levels to that of OPC concrete under laboratory conditions [32]. The slower and extended hydration of fly ash concrete does make the material sensitive to poor curing on site [33].

Ground granulated blastfurnace slag is a latent hydraulic binder where the hydration of slag is accelerated in the presence of cement. The pore structure of slag concrete is similar to that of OPC concrete but some researchers indicate that there are fewer capillary pores than OPC concrete producing a slightly less permeable concrete [34]. Slag concrete effectively resists chloride diffusion due to the superior chloride binding capacity of the material [35]. There is concern however about possible lower activation levels required for chloride-induced corrosion of slag concrete [36] The use of slag in concrete increases the sulphate resistance and helps reduce the expansion associated with alkali aggregate reaction. Carbonation rates in slag concrete have been found to be higher than in OPC concrete particularly when poorly cured [37]. Slag concrete is particularly vulnerable to poor curing which has implications for the quality of the covercrete protecting the reinforcement [38].

Condensed silica fume (csf) is produced in electric arc furnaces during the production of silica and ferrosilicon alloys. The material is almost pure silica and its fine amorphous nature results in it being a highly active pozzolanic binder. The use of csf has been shown to have many beneficial effects on concrete and its use in construction has increased considerably in recent years.

2.3.3 Construction practice

The potential durability of concrete may be compromised by poor construction practice particularly with regard to curing, compaction and cover to reinforcement. The influence of these factors on durability performance is difficult to measure as existing site monitoring and compliance criteria are inadequate.

Curing of concrete involves maintaining an adequate moisture condition and temperature after casting to promote cement hydration. Effective curing of concrete helps produce a high quality surface layer which protects the interior from ingress of aggressive agents [39]. The lack of durability of many concrete structures, particularly in hot dry climates, is thought to be largely due to inadequate curing but there are few examples in the literature to substantiate this opinion [40]. The lack of evidence is due to the complex nature of deterioration which makes diagnosis of the underlying causes of damage difficult. In severe hot and dry conditions, curing is essential to prevent early age damage such as plastic shrinkage.

Good concrete mix design and compaction is fundamental for achieving a dense and durable concrete. Poorly compacted concrete is often found on site due to inadequate vibration, congested reinforcement and lack of suitable specifications. The implications of poor compaction are difficult to quantify but when honeycombing and segregation occur the inherent protection of the covercrete may be completely destroyed.

The selection of the correct cover to reinforcement and ensuring that this is achieved on site is as important as any other factor affecting the durability of reinforced concrete. Poor cover control is the most quoted fault responsible for poor durability performance of reinforced concrete structures in the marine environment [41]. Often a compromise must be made between greater covers and limiting crack widths in the tensile zone of concrete. A minimum cover depth of 50 mm is specified by BS6349 regardless of other requirements to ensure adequate protection to the reinforcement under severe marine exposure [9]. The occurrence of construction faults such as voids, blowholes, surface cracks and poor compaction can easily justify this minimum cover depth.

2.3.4 Exposure and service conditions

Exposure conditions vary considerably in the marine environment depending on a wide range of factors such as micro and macro-climate, position relative to the sea, orientation and exposure to prevailing winds. The durability performance of a marine concrete structure must be assessed from the original concrete properties and the severity of exposure. Broad classification of marine exposure zones has been proposed by many design codes with categories usually being either extreme, severe or moderate [42]. The category limits are fairly arbitrary and usually relate to the distance of the structure from the sea regardless of local climate. Table 2.3 shows the severity ratings of marine environments determined by different codes of practice which shows the arbitrary nature of classification. Fookes has proposed a more rational system of exposure categories determined on the basis of exposure points for hot, dry marine environments [43].

Table 2.2 : Severity of Marine Exposure From Codes [44,45,46,47]

CODE NAME	EXPOSURE CATEGORY NAME	DESCRIPTION OF CONDITIONS	MINIMUM GRADE (MPa)	MAXIMUM w/c RATIO	MINIMUM CEMENT (kg/m^3)	MINIMUM COVER (mm)
BS5400/ TMH 7	Extreme	Abrasive action of seawater	40	-	-	75
	Very severe	Seawater spray	40	-	-	50
BS 8110	Extreme	Abrasive action of seawater	45	0.50	350	60
	Very severe	Seawater spray	40	0.55	325	50
CEB	4a	Seawater environment	30	0.55	300	40
	4b	Seawater with frost	30	0.50	300	40
ACI/ CSA	C-1	Chloride environments	35	0.40	320	50
	C-2	Moist with freeze thaw	30	0.45	290	50

Even the most durable concrete will suffer some deterioration during its service life and the amount that is acceptable will depend on the function, aesthetics and consequence of failure of the structure. Large marine structures such as breakwaters and harbour structures may suffer extensive damage but remain fully serviceable while bridges and buildings may need major repairs after limited deterioration. This difference in the amount of acceptable deterioration has a major bearing on the service life and consequent durability performance of the structure.

2.4. Measuring Deterioration of Marine Concrete

A considerable number of in situ and laboratory tests have been developed which attempt to measure the residual durability of concrete. Some of the more appropriate methods of measuring deterioration of marine concrete are discussed in more detail below.

2.4.1 Visual assessment

A visual assessment of exposed concrete which identifies and defines areas of distress forms the basis of a condition survey for a structure. Confirmation of observations can be made with in situ and laboratory testing of the concrete. Most deterioration processes such as abrasion damage, reinforcement corrosion and alkali aggregate reaction show visible signs on the surface and site diagnosis is often possible. Quantifying the full extent and implications of deterioration needs more extensive testing using in situ and laboratory methods.

Damage classification models which objectively define the extent and type of deterioration have been formulated [48]. Evidence of damage is classified into standard categories which are carefully defined in terms of their appearance, location and causes. Defects such as cracks may therefore be divided into corrosion, temperature, shrinkage, plastic, map, hairline, craze, diagonal, longitudinal, corner and transverse cracks. These defects may then be rated in terms of severity ranging from very slight damage to very severe. The damage rating is selected by measuring the defect on site and selecting the appropriate range in the damage classification model. Cracks for instance are rated in terms of their surface crack widths with crack widths less than 1 mm being rated very slight while crack widths greater than 25 mm would be rated very severe.

Condition surveys of marine concrete structures should form part of the overall maintenance strategy to ensure maximum service life is achieved. Visual evidence of deterioration can be recorded relatively cheaply at regular intervals and further testing can be done should the results of the condition survey warrant further action. Visual survey should form the first phase of any durability audit which correctly done can ensure timely and cost effective repairs.

2.4.2 In situ techniques

In situ test methods for concrete can be used on site and being generally non-destructive allow greater freedom in testing. In situ techniques coupled with an extensive condition survey can provide valuable information on which a full assessment of the extent of deterioration is often possible. A common feature of in situ tests is their sensitivity to external influences which must be allowed for to obtain reliable results.

Determination of cover depths of reinforcement is simply done using modern covermeters and, with chloride profiles and carbonation depths, can be used to assess the risk of corrosion. Ultrasonic pulse velocity equipment and impact echo instruments provide sophisticated assessment of discontiuities and presence of cracks, flaws and voids in concrete. Surface hardness of concrete can be assessed using the Schmidt hammer which is also able to estimate the compressive strength of concrete when suitably calibrated.

Corrosion monitoring of steel can be done using in situ methods such as resistivity and electropotential mapping. Concrete resistivity is believed to control the rate of corrosion of reinforcement in concrete once favourable conditions for corrosion have been established [49]. Resistivity is dependent on the moisture condition, permeability and pore water ionic concentration of concrete. A four point Wenner probe assembly connected to a digital display makes resistivity measurements simple to perform on site. Variations caused by moisture gradients, carbonated layers, wetting fronts and high salt contents at the surface may cause erroneous resistivity readings [50]. Potential mapping of reinforcement determines the thermodynamics of steel corrosion but cannot evaluate the kinetics of the reaction [51]. Copper/copper sulphate electrodes are generally used to determine the potential of the reinforcement, from which the risk of corrosion may be assessed. Interpretation of results requires experience as often the half-cell potential represents the chemistry of the solution in contact with the steel and is not related to corrosion at all [52]. Influences such as carbonation, concrete delamination and impressed currents may completely change the thermodynamics of the system.

Other useful in situ techniques for assessing deterioration of marine concrete include test kits for assessing carbonation depth and presence of chlorides on broken concrete samples. These methods are less accurate than laboratory methods but can provide useful site data. Less reliable in situ techniques include pore humidity measurement and in situ permeability testing using methods such as the covercrete absorption and ISAT tests [53].

2.4.3 Chloride testing

Chloride testing of concrete is done to assess the risk of corrosion of reinforcement given that depassivation of steel may occur when the chloride concentration reaches a certain threshold value. Concrete is analysed for chlorides by extracting the chlorides from powdered concrete using either acid digestion or water dissolution and the chlorides liberated are measured using various chemical analytical techniques.

Chlorides in concrete consist of free chlorides in the pore water solution, chlorides chemically bound to the calcium aluminate phases and chlorides physically bound to the cement by adsorption. Only the free chlorides are thought to be responsible for depassivating the reinforcement with the chemically and physically bound chlorides being unreactive with regard to the steel [22]. Water soluble chloride testing techniques were developed in an attempt to determine these free chloride levels [54]. A rough estimate of the free chlorides is made but several factors affect the reliability of this approach: grinding of concrete releases physically bound chlorides; a dynamic equilibrium exists between free and bound chlorides resulting in release of bound chlorides into solution during the test; and different cement types will have different rates of dissolution which may result in incomplete release of free chlorides into solution at the end of water immersion. Water soluble chloride tests provide an approximation of the free chloride content but this only applies for a limited set of conditions and materials.

Many researchers prefer the acid soluble chloride test as bound chlorides may decompose to release chlorides when free chloride concentrations reduce. It is therefore possible for all chlorides to be potentially aggressive to the steel. Hope stated that there is general consensus that the acid soluble chloride test is more reliable than the water soluble chloride test [55]. Acid soluble chloride tests are said to measure the total chloride content of concrete although organic chlorides and some chemically bound chlorides from chloride contaminated fresh concrete have been shown to be resistant to the strong acid digestion process [52,56,57]. From the chloride content determined, the free chloride content can be estimated if the equilibrium characteristics of free to total chloride are known. Assessment of corrosion risk is made using empirical chloride corrosion thresholds determined from laboratory and site investigations. Determination of acid soluble chlorides from marine concrete can provide useful information about the rate of chloride ingress and chloride binding characteristics of concrete.

A number of rapid chloride tests have also been developed for site testing of concrete. Methods such as the Hach test kit and the Quantab test strip system provide a rough diagnosis of chloride content but laboratory methods should be used for accurate assessment [58]. In situ chloride tests have the advantage of providing instant results and can be used effectively when the depth of chloride contamination is required. Chloride profiling equipment is also available for site testing using a calibrated electrode. The method is however fairly tedious to use on site with a profile of depth 50 mm taking over two hours which is prohibitive for multiple tests. Table 2.3 shows a comparison between the different test methods used for analysing chloride content.

Table 2.3 : Comparison of Chloride Tests

CHARACTERISTIC	METHOD OF CHLORIDE ANALYSIS			
	VOLUMETRIC TITRATION	POTENTIOMETRIC TITRATION	ION SELECTIVE ELECTRODE	INSITU METHODS
EASE OF USE	Simple test but time consuming	Simple test, fairly quick	Simple test, quick	Simple, quick site tests
ACCURACY	Fair	Good	Good but problems with interference	Poor
COST	Inexpensive	Fair initial capital cost	Low initial capital cost	Low initial capital cost
MEASUREMENT PARAMETER	Indicator colour change	Redox potential equivalence point	Calibrated potential reading	Indicator colour change
COMMENTS	Suitable for site laboratories	Suitable for research laboratories	Not suitable for marine structures	Rough measure of chloride content

Several methods have been developed for determining the free chloride content of concrete such as the pore water expression device, solvent leaching and X-ray diffraction [59,60]. These methods provide useful research information but are not suitable for repetitive site testing of concrete. Determination of the free chloride content is best done using empirical relationships between free and total chlorides derived from the pore expression device and applying a conversion factor to the acid soluble chloride content results measured from standard sampling techniques [61]. Accurate measurement of the free chloride content does not guarantee a reliable assessment of corrosion risk however as Schiessl has shown that the chloride corrosion threshold value is affected by the local environment and the quality of concrete with respect to concrete grade, cover to reinforcement and concrete permeability. The relationship between corrosion threshold and these internal and external factors is illustrated graphically in Figure 2.11 [62].

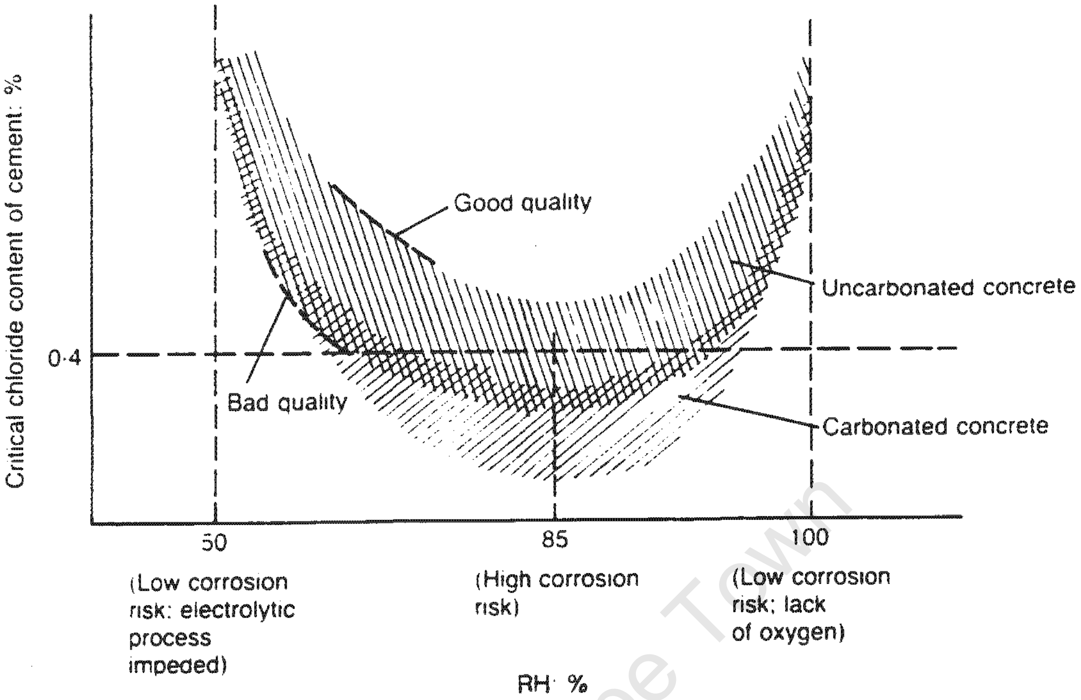


Figure 2.11 : Chloride Corrosion Threshold Level [65]

Analytical methods for measuring chloride contents of concrete include volumetric analysis using Volhard's standard silver nitrate titration, potentiometric titrations using redox probes and ion selective electrode analysis [63]. Volumetric titrations are relatively easy to perform except that filtering of the solution is time consuming with cement extenders. The titration requires significant operator judgement in determining the titration end-point particularly with slag concrete where the indicator colour change is indistinct. The method is also fairly inaccurate and is not able to accurately determine minimum specified concentrations in fresh concrete. Ion selective electrodes are able to analyse chlorides quickly and accurately but care should be used on marine concrete samples because interference of sea water ions such as halides and sulphides affects the reliability of the test [64]. Potentiometric titration is an extremely reliable and accurate analytical method for chlorides but the method is time consuming when done manually. Automatic titrators make the potentiometric titration the most effective method of analysing chlorides in concrete as the titrant dosage, redox potential measurement and analysis can be preprogrammed allowing automation of the titration and analysis, which decreases the time and increases the accuracy of the process.

2.4.4 Corrosion monitoring

Several electrochemical methods have been developed to assess the state of corrosion of reinforcement. Corrosion monitoring can be done using non-destructive methods such as potential mapping, galvanostatic pulse techniques and in situ polarization resistance or by using embedded sensors. These electrochemical methods all require considerable experience to interpret the results before a realistic assessment of the state of corrosion can be made. Details of various corrosion monitoring techniques are shown in Table 2.4.

Potential mapping is currently the only electrochemical method which is standardized and used extensively on site. Potential mapping is best used on concrete where chloride induced corrosion is prevalent with fairly steep potential gradients [65]. The method can measure the thermodynamics of the system but cannot measure corrosion rates. The galvanostatic pulse method provides similar information to that of potential mapping but produces better differentiation between areas of active and passive reinforcement [66]. Polarization resistance measurements can be used for either carbonation or chloride induced corrosion and provides information about the mean corrosion rate within the area tested. Polarization resistance is therefore more suitable for concrete exhibiting general corrosion and cannot measure local corrosion rates from pitting [67].

Several other methods of corrosion monitoring have been developed including electrochemical noise, impedance spectroscopy and harmonic analysis [68]. The electrochemical noise method does not perturb the system during measurement but is unreliable on structures due to interference from non-corrosive sources. Electrochemical impedance spectroscopy and harmonic analysis can provide extremely sophisticated analysis of corrosion but require expensive monitoring equipment and are not used often on site.

Embedded sensors can be used for severe exposure conditions where the onset of corrosion needs to be monitored. Reference electrodes can measure the likelihood of corrosion but the long-term stability of the electrodes is poor. Macro cell sensors have good long-term stability and are able to give an idea of the corrosion rate and measure the infiltration of depassivating agents. Other embedded sensors which are being developed include resistivity, humidity and linear polarization sensors.

Table 2.4 : Corrosion Monitoring Techniques [68]

CHARACTERISTIC	CORROSION MONITORING TECHNIQUE				
	VISUAL OBSERVATION	GRAVIMETRIC	POTENTIAL MAPPING	LINEAR POLARIZATION	GALVANOSTATIC PULSE
SPEED OF MEASUREMENTS	Quick	Slow	Quick	Quick	Quick
RESPONSE TO CHANGES	Slow	Slow	Quick	Quick	Quick
QUANTITATIVE INFORMATION	Fair to Poor	Good	Fair to Poor	Good	Good
DAMAGE TO STRUCTURE	No damage	Major damage	Minor damage	Minor damage	Minor damage
MEASUREMENT PARAMETER	Appearence changes	Avg. corrosion current density	Probability of corrosion	Corrosion current density	Corrosion current density
COMMENTS	Useful low cost initial survey	Impractical on most sites	Simple site test but inexact	Increasingly used on site	Complex analysis

2.4.5 Laboratory testing of samples

Concrete core samples extracted from structures can be tested under controlled conditions in the laboratory. Core strength testing is commonly done to estimate the residual strength of the material where structural performance is essential. Visual examination of cores can provide information about carbonation depth, aggregate quality, crack depths and corrosion of reinforcement at predetermined anode areas. Extraction of cores may affect the aesthetics of a structure but suitable repairs will lessen the impact of coring. Some deterioration processes may only be evident internally and valuable information is often gained by examination of cores and petrographic analysis of internal concrete.

Core samples may also be tested for fluid transport properties and abrasion resistance in the laboratory to estimate the quality of the covercrete. Interpretation of these results may be complicated by the effects of deterioration processes which alter the concrete pore structure. Carbonation has been found to reduce the water absorption rate of concrete thereby appearing to improve the absorption characteristics of concrete [69]. These effects make comparisons between different concretes with variable amounts of deterioration extremely difficult.

2.5. Repair of Marine Concrete Structures

Opinions differ with regard to repair strategies for concrete structures with an increasing number of repair materials and systems competing in an expanding market. The technical benefits of any repair method must always be assessed in terms of site logistics, long-term performance and cost effectiveness. A brief review of repair strategies for marine concrete structures is given below.

2.5.1 Basic repair strategy

Before repairs are contemplated for a marine concrete structure several factors need to be addressed. Initially the types, causes and consequences of deterioration need to be thoroughly assessed. Condition surveys are often inadequate due to budget restraints from the client who would rather spend money on repairs than reports. This short-sighted approach can result in ineffective repairs which may actually exacerbate the deterioration process.

Once the extent and causes of the deterioration have been suitably assessed appropriate repair strategies can be examined. The practicability of repairs must be assessed in terms of site conditions, access, availability of specialist contractors and acceptability of repair processes such as sand-blasting and water jetting. Trial work might be needed when insufficient information is available from comparable situations elsewhere. The execution of repair work should be done by trained personnel and site quality control is essential to ensure compliance with project specifications. Repairs should also be monitored at regular intervals to assess the long-term performance, and improvements made on poor repair designs.

The type of repair strategy selected will depend on the type of deterioration together with local conditions. Five different approaches have been identified for repairs to marine concrete structures.

a) No intervention

Deterioration is not deemed to be serious enough to warrant repairs or the structure has insufficient serviceable life remaining. This option should be used on older structures which are still serviceable and where the implications of further deterioration are not severe.

b) Localised repairs

Areas of deterioration are repaired locally with no attempt being made to reduce the causes of damage or prevent further deterioration. This method of repair is cheap in the short-term but continued maintenance may prove costly in the long-term. Generally only cosmetic protection is provided and the potential service life is not significantly prolonged.

c) Reducing the deterioration process

The rate of deterioration may be reduced by changing the environmental and service conditions which are responsible for the damage. Barrier coatings are often applied on the concrete surface to reduce the ingress of aggressive agents from the environment. Sacrificial concrete layers are also used to lessen the impact of surface effects such as abrasion and chemical attack. Conditions remote from the structure may also be altered by construction of new works to reduce the amount of damage such as changing drainage and providing additional coastal protection in exposed conditions. Electrochemical methods such as cathodic protection and desalination may be used to alter environmental conditions experienced by the reinforcement.

d) Major reconstruction

Major reconstruction of damaged elements of a structure would involve extensive replacement of concrete to ensure a durable repair. Sites of future damage would also be repaired and strengthened to ensure that the service life of the structure is significantly improved. Areas of substandard construction would also be improved to guarantee durability performance.

e) Demolition or use of an alternative structural system

Structures with major deterioration may require such extensive repair that it may be more cost effective to demolish and rebuild. Should this option be taken it is vital that the new structure is designed to resist the deterioration processes which were responsible for the failure of the previous structure. Some structures must remain in service but where deterioration has compromised the structural integrity, construction of an alternative structural system is the most feasible method of restoring the structural integrity. The deteriorated surface can then be repaired aesthetically and regular maintenance done at a later date. A decision flow chart showing how a rational method of instituting repairs is shown in Figure 2.12.

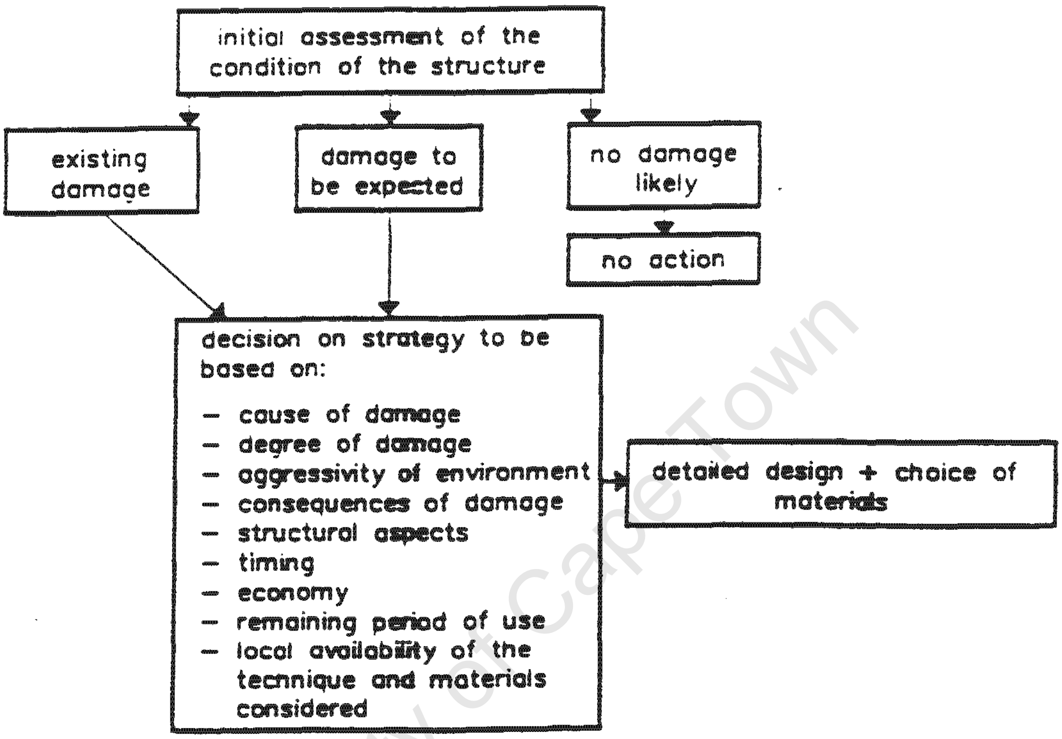


Figure 2.12 : Decision Flow Chart for Concrete Repairs [65]

2.5.2 Surface damage

The concrete surface may be damaged due to the effects of abrasion, impact loads, weathering, salt crystallization, and frost action. Surface repairs to restore durability must be carefully designed and executed to withstand the service loadings and environmental stresses. Effective surface repairs require compatibility between the existing substrate and the repair material [70]. Compatibility involves a balance between the physical, chemical and dimensional properties of old and new materials.

Dimensional stability of the repair material requires that volume changes of the materials due to drying shrinkage, thermal expansion and modulus of elasticity are compatible with the existing substrate. What is important is that the repair material must accomodate the strains imposed upon it without cracking and debonding. Significant differences in these material properties between existing concrete and repair material may result in failure when exposed to drying, loading and temperature fluctuations. Chemical compatibility involves chemical composition and durability considerations. Physical compatibility entails ensuring that load carrying capacity, adequate substrate preparation and bond between substrate and repair material are satisfactory. Repairs may be subject to a variety of loadings due to working loads, environmental effects and sporadic impact loads.

The insistence by designers on low permeability as the main criterion for repair materials has often resulted in unsuitable choices of materials that are incompatible with the underlying substrate. These problems may result in cracks or debonding of the repair material thereby defeating the advantage of using a low permeability material. The primary consideration when designing repair materials should be compatibility, with low permeability being a secondary consideration.

2.5.3 Reinforcement corrosion damage

A corrosion cell consists of an anode, a cathode and electronic and ionic conduction paths. Stopping any of these processes will break the corrosion cell and therefore the damage caused by the corrosion process. To achieve these different objectives a variety of methods can be used to repair reinforcement corrosion. Repair strategies for reinforcement corrosion are shown in Figure 2.13.

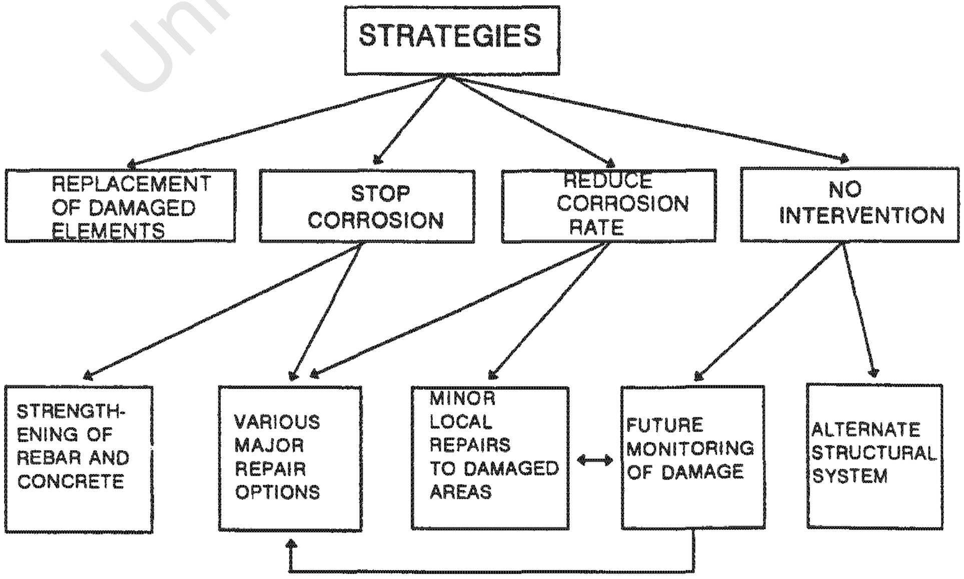


Figure 2.13 : Basic Repair Strategies For Reinforcement Corrosion [61]

a) Local patch repairs

Patch repairs are usually short-term solutions to corrosion damage but may suit the maintenance strategies of some owners who have limited annual repair budgets. If the old concrete adjacent to the repairs does not ensure passivity for the reinforcement further corrosion damage is likely. Patch repairs are generallly seen to provide only short-term relief and do not significantly improve the service life of marine concrete structures [65].

b) Repassivation

General corrosion of reinforcement caused by carbonation of concrete may be stopped by repassivation either by replacing the carbonated concrete with alkaline material or by electrochemical realkalization. The diffusion of hydroxyl ions to the reinforcement by electrochemical means is achieved by applying an alkaline coating to the surface and driving the hydroxyl ions to the reinforcement by electro-osmosis. Care must be taken during the electrochemical process to ensure that current densities are within the safe range of application.

c) Limiting moisture content

Surface treatments can be used to reduce the moisture content of concrete. A reduction of moisture is generally effective in reducing the corrosion rate of carbonated concrete. For chloride-induced corrosion the effectiveness of this method is not clear as there is insufficient knowledge about the critical moisture content required for corrosion although corrosion does appear to be dependent on the moisture condition of concrete [71]. Hydrophobic impregnation of the concrete surface is effective in reducing the absorption of water from the environment but allows drying to lower the internal humidity of concrete. Laboratory research has shown promising results using this method but the long-term performance under field conditions still needs to be evaluated particularly under severe marine conditions.

d) Coating of the reinforcement

When the repair material cannot ensure repassivation of reinforcement and other methods are impracticable a protective coating can be applied to the steel surface. The coating must act as a physical barrier and epoxy based organic coatings are most effective. All steel surfaces which are likely to be exposed to critical chloride levels must be coated after being cleaned to remove all rust using high pressure water jetting followed by moderate sand-blasting. Negative effects of the method include loss of anchorage capacity of bars, difficulty of cleaning corrosion pits and chloride penetration through coatings.

e) Electrochemical methods

Electrochemical protection methods such as cathodic protection, electrochemical chloride extraction and electrochemical realkalization are increasing in popularity with the refinement of these methods. Cathodic protection requires a permanent power source to drive the low voltage direct current which lowers the potential of the steel and slows down the corrosion process. Elecrochemical chloride extraction and realkalization use higher current densities and are run for short periods until the passivity of the reinforcement has been restored. These rapid electrochemical methods have several disadvantages which are caused by the relatively high current densities and associated effects. Problems include a build up of alkalis at the reinforcement causing alkali aggregate reaction, hydrogen evolution at the reinforcement resulting in hydrogen embrittlement, acid and chlorine gas evolution at the anode causing local damage and chloride ions beyond the reinforcement are not removed by electrochemical desalination and may migrate back to the reinforcement [65,72].

2.5.4 Chemical deterioration

Chemical deterioration of marine concrete is generally a surface phenomenon with chemical reactions and leaching of soluble products occurring within a relatively thin surface layer. Repair of this surface deterioration is best achieved by removal of deteriorated material to sound substrate, making good with suitable repair material and then applying a barrier coating to prevent further damage. Internal distress of marine concrete may result from alkali aggregate reaction which requires a different repair strategy. Reducing the moisture content of concrete has been shown to be effective in reducing the expansive effects of alkali aggregate reaction [73]. Hydrophobic impregnation is effective in reducing the moisture content of concrete but is generally not effective for concrete exposed to direct wetting from tidal action.

2.6. Conclusions

The marine environment provides a severe test of durability of reinforced concrete structures due to the nature of the physical and chemical attack. Deterioration of concrete may occur rapidly unless the structure is well designed and built with sound materials using good construction practice. The outer layer of concrete provides most of the protection to the structure from aggressive action and it is essential that the quality of this layer is guaranteed. Current design approaches do not guarantee durability because durability related properties are not adequated specified by the design codes. Furthermore, what is specified is often not achieved on site. This uncertainty at both the design and construction phases needs to be reduced by specifing suitable durability performance criteria for concrete. Performance specifications must have a rational basis and allow measurement of concrete quality on site. The concept of performance specifications and methods that can be used to measure the relative quality of concrete are discussed in more detail in Chapter Three.

The severity of marine exposure varies considerably depending on conditions such as micro and macroclimate, position relative to the sea, orientation and exposure to prevailing winds. Existing codes of practice do not adequately define marine exposure in terms of these factors and a rational system of environmental rating needs to be developed. The lack of clarity results in many structures being over-designed in terms of durability protection while others are not adequately protected because the severity of exposure was under-estimated. A rational system of rating the marine environment will help optimise construction materials and will be useful for determining the durability performance of marine concrete structures.

More research is required on the effectiveness of repair systems under field conditions so that designers can make best use of the wide range of options available for concrete repairs. A rational approach is also required toward maintenance strategies and life cycle costs of marine concrete structures. The advantage of early repairs and preventative maintenance can only be shown if thorough documentation of the performance of all types of repairs is done. Armed with this information, engineers can start making progress in the area of concrete durability and repairs which for many years has been researched intensively with little positive impact on concrete quality on site.

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CHAPTER THREE

DEFINING THE POTENTIAL DURABILITY OF CONCRETE

3.1 Introduction

Deterioration of marine concrete is generally associated with external agencies such as chlorides which penetrate into the concrete interior causing damage. The movement of aggressive agents from the environment into the concrete is affected by chemical interactions with concrete. These chemical interactions may affect the transport properties of the material. Using the premise that durability may be defined by the resistance of the surface layer of concrete to penetration of aggressive agents, intrinsic material properties which determine transport mechanisms and physical and chemical interactions with concrete need to be quantified before the potential durability can be assessed [1]. This assumption may not be valid for surface effects such as chemical dissolution and abrasion or when the material has inherently low resistance to the medium of environmental attack. Internal disruptions from alkali aggregate reaction may also be less dependent on the infiltration of fluids and ions from external sources as suitable conditions for deterioration may already exist in the concrete when sufficient alkalis and reactive aggregate are present in moist conditions.

Movement of fluids and ions through concrete can occur by three main types of transport mechanisms: absorption, diffusion and permeation [2]. These mechanisms may occur simultaneously or independently and their contribution to fluid transport is dependent on local environmental conditions. Transport mechanisms may be significantly affected by adsorption of ions and fluids on to the pore surfaces of concrete due to either physical or chemical bonds. These physical and chemical interactions may alter the concrete pore structure thereby changing the resistance of the concrete to fluid transport processes [3].

If concrete is to be accurately defined for potential durability a rational system of characterizing the resistance of the material to fluid transport processes and chemical interactions needs to be formulated. Marine concrete is affected by all three transport mechanisms while chemical interactions between sea water and concrete constituents are also important. Practical durability tests which are based on transport mechanisms and chemical and physical interactions between concrete and the environment need to be identified. These durability tests will help characterize the resistance of the concrete cover. The tests must have a sound theoretical and technical basis to ensure the transport mechanisms are accurately modelled without significant interference.

Comparisons between the potential durability of different concretes can be done using the results of early age durability tests but the limitations of these comparisons must be appreciated. Accurate comparisons of durability of concrete can only be done once the long-term characteristics of the material are understood and allowed for when comparing early age durability tests. Silica fume concrete for instance accelerates cement hydration and may result in the potential durability of the material being over-estimated when compared to concretes containing fly ash and slag which have considerably slower early age development.

3.2 Transport Mechanisms

Transport through concrete involves the movement of ions, liquids and gases from external sources into the concrete where chemical interactions may result in deterioration of the material. These agents are driven by gradients in physical and chemical properties including ionic concentration, hydraulic pressure and capillary potential. The three main transport processes in concrete are absorption, diffusion and permeation which are shown for marine concrete structures in Figure 3.1.

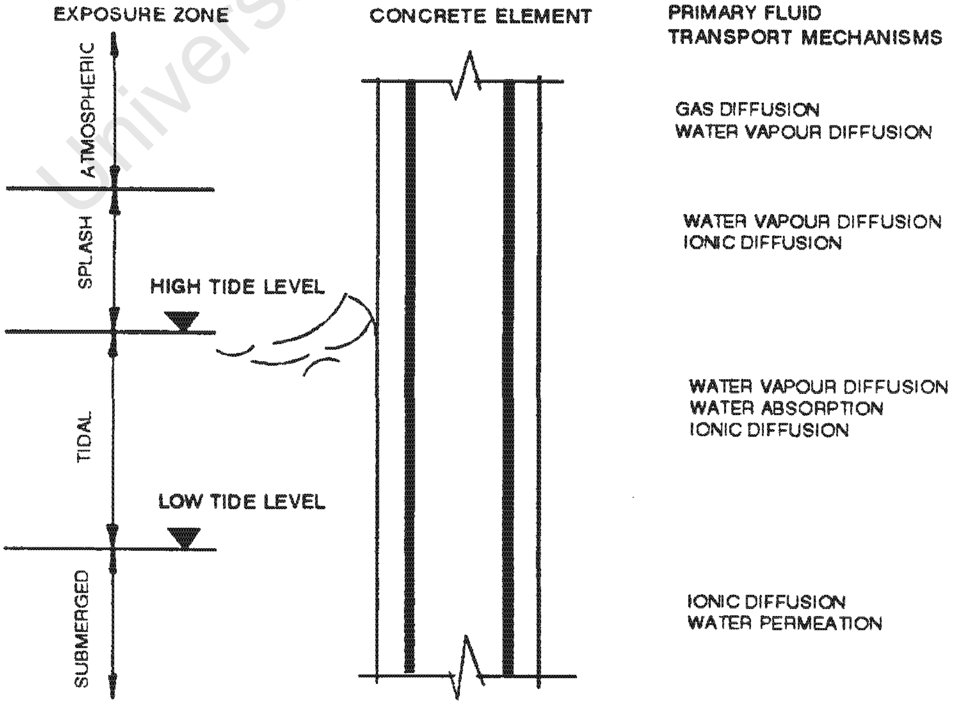


Figure 3.1 : Fluid Transport Mechanisms in Marine Structures [2]

3.2.1 Absorption

Absorption is the process whereby fluid is drawn into a porous material to fill the unsaturated pores under the action of capillary forces. The capillary potential or suction is dependent on the geometry of the pores and the saturation level of the concrete. Water absorption caused by wetting and drying action at the concrete surface is the dominant fluid transport mechanism in the outer skin of concrete for most marine structures in and above the tidal zone [4]. The absorption process results in high surface concentrations and steep concentration gradients, thus driving salts inwards.

The theory of unsaturated flow can be used to describe the absorption process since steady state conditions have not been achieved during absorption [5]. The unsaturated flow theory was initially developed for water transport through soils but applies equally to concrete with the flow being expressed in terms of the Darcy-type equation, i.e.

$$\frac{dq}{dt}=\frac{kAh}{L} \quad \text{........................} \quad (3.1)$$

where dq/dt is the rate of flow, k is the coefficient of permeability, A is the cross-sectional area and h/L is the hydraulic gradient. The unsaturated flow equation can be expressed in vector form as follows :-

$$q=K(\theta)F_c(\theta) \quad \text{......................} \quad (3.2)$$

in which q is the vector flow velocity, K the hydraulic conductivity and F_c the capillary force. Both the hydraulic conductivity and capillary force are dependent on the water content Θ. In addition the capillary force is dependent on the gradient (∇) of the capillary potential (Y).

$$F_c=-\nabla Y(\theta) \quad \text{......................} \quad (3.3)$$

The one dimensional case assuming isotropic conductivity produces the extended Darcy-type equation.

$$q=-K(\theta)\frac{dY}{dx} \quad \text{........................} \quad (3.4)$$

Expressing the equation in terms of hydraulic diffusivity produces the following equation.

$$q=-D(\theta)\frac{d\theta}{dx}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.5)$$

where the hydraulic diffusivity, D(Θ) represents the material property which allows capillarity to occur and is given by

$$D(\theta)=K(\theta)\frac{dY}{d\theta}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.6)$$

Simply expressed the rate of flow is dependent on the product of the hydaulic conductivity and the gradient of the capillary potential of the concrete. The dynamics of the water absorption process are shown in Figure 3.2.

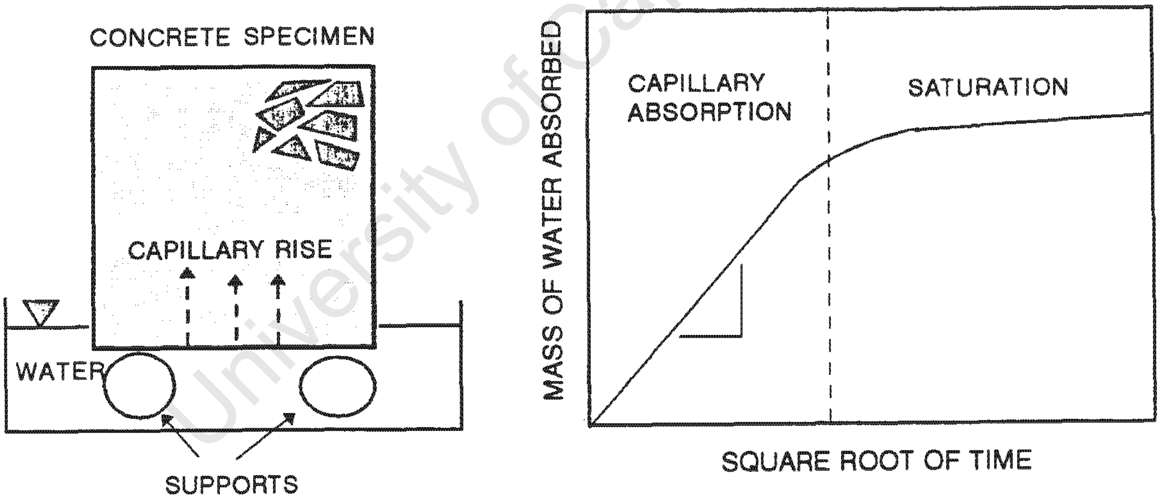


Figure 3.2 : Water Absorption into Concrete

Sorptivity is defined as the rate of movement of a wetting front through a porous material under the action of capillary forces. Using the one dimensional case of water absorption with defined boundary conditions it is possible to define sorptivity in terms of the extended Darcy equation. The cumulative water absorption per unit area i, is dependent on the square root of time and may be expressed as follows.

$$i=St^{1/2} \qquad (3.7)$$

in which S is the sorptivity of the material. Sorptivity is related to the hydraulic gradient and capillary potential gradient and may be used to define the absorption ability of concrete. While the hydraulic diffusivity may be considered the fundamental water absorption property of porous materials its determination is complicated. Sorptivity on the other hand is relatively simple to determine and is directly related to the hydraulic diffusivity.

Extensive experimental work has confirmed the theoretical basis for sorptivity, and sorptivity tests have gained popularity due to their simplicity and accuracy [6,7,8]. Sorptivity is affected by internal influences such as initial water content and microstructural configuration and external factors such as temperature, tranmission fluid and sample geometry. Water absorption is also affected by the orientation of the material as the flow is influenced by capillary and gravity driven forces. Dry concrete may have capillary suction heads far in excess of gravity driven forces, with capillary heads of up to 15 m being recorded [7].

3.2.2 Diffusion

Diffusion is the process by which a liquid, gas or ion moves through a porous material under the action of a concentration gradient. Diffusion occurs in partially and fully saturated concrete and is the dominant transport mechanism in concrete internally for most marine structures [9]. High surface salt concentrations developed due to the effects of absorption and diffusion cause the ions to migrate towards the low concentrations in the internal material.

The diffusion process is described by Fick's first law of diffusion which is as follows for the free liquid phase :

$$J_x=-D_f\frac{dC}{dx} \qquad (3.8)$$

in which J_x is the flux per unit cross-sectional area of the liquid, D_f the diffusion coefficient and C the concentration of the liquid. When the liquid is constrained in a pore structure with longer diffusion paths to that of the free liquid phase a diffusion coefficient D_p may be defined with respect to the pore liquid of the material and is given by :

$$J_x=-D_p\frac{dC}{dx}\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.9)$$

The configuration of the pore structure with longer diffusion paths and localised constrictions produces lower diffusion coefficients D_p compared to D_f for the free liquid phase [10]. This is due to the constrictivity δ and tortuosity τ of the pore system which effectively restrict the diffusion process such that

$$D_p=D_f\frac{\delta}{\tau^2}\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.10)$$

The flux may be expressed in terms of the medium rather than the liquid to produce the following equations where $< >$ refers to the medium generally and not only the pore liquid.

$$\langle J_x\rangle=-D_i\frac{dC}{dx}\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.11)$$

$$D_i=\frac{D_f\epsilon\delta}{\tau^2}\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.12)$$

where ϵ is the volume fraction of porosity and D_i is the intrinsic diffusion coefficient. The quantity $\epsilon\delta/\tau^2$ is a material property which characterizes the pore structure and is referred to as the diffusibility of the material. Diffusibility is an intrinsic material property defined by the concrete pore structure while the term diffusivity defines the material resistance to diffusion being dependent on material properties and the concentration and mobility of the diffusing ions. Concentrations of species in concrete are generally measured with respect to the material which produces an apparent diffusion coefficient D_a using Fick's equation.

$$\langle J_x\rangle=-D_a\frac{d\langle C\rangle}{dx}\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.13)$$

where $\langle C\rangle$ is the average concentration over the solid and liquid phases and may be expressed in terms of the volume fraction of porosity.

$$\langle C\rangle=\epsilon C_1+(1-\epsilon)C_s\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.14)$$

where C_l and C_s are the concentrations in the liquid and solid phases respectively. Apparent diffusion coefficients are relatively easy to determine but are affected by chemical interactions and adsorption effects. Assuming a linear relationship between the solid and liquid phases of the diffusing species one can define the binding ratio or adsorption isotherm as follows:-

$$\pi = \frac{C_s}{C_l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.15)$$

Using this ratio the apparent diffusion coefficient can be related to the original diffusion coefficient D_f as follows.

$$D_a = \frac{D_f \delta \epsilon}{\tau^2 \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.16)$$

where the parameter α is referred to as the binding capacity of the material with the diffusant and represents the maximum capacity of the material to hold the diffusant. The binding capacity therefore is determined by the volume fraction of the porosity and the binding ratio of the solid constituents such that the following relationship may be defined.

$$\alpha = \epsilon + (1 - \epsilon)\pi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.17)$$

The solution of Fick's Law is used to predict the concentration of diffusing species at any particular depth and time. The diffusibility and related properties may also be used to define the resistance of concrete to diffusion and forms the basis of many accelerated tests. Figure 3.3 shows the theoretical chloride profiles resulting from diffusion for concrete under saturated conditions assuming a constant diffusion coefficient of 1.0E-8 cm²/s and surface concentration of 4.0 % by weight of cement.

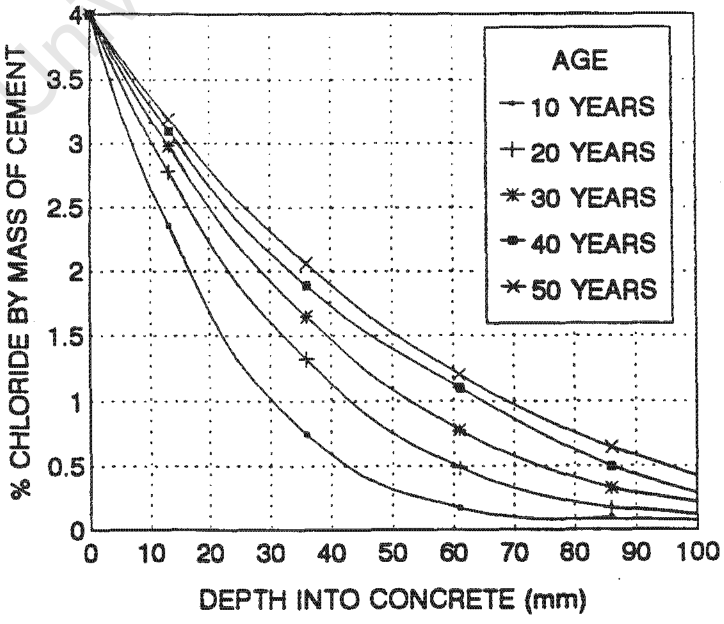


Figure 3.3 : Chloride Diffusion Profiles in Concrete

The rate of ionic diffusion is largely dependent on the mobility of ions and the rate of counter diffusion of oppositely charged ions [10]. Diffusion rates are dependent on temperature, saturation level of concrete, type of diffusant and inherent diffusibility of the material. Diffusion of ions and fluids into concrete is complicated by chemical interactions, partially saturated conditions, defects such as cracks and voids and electrochemical effects due to steel corrosion and stray currents. In the marine environment the diffusion of chloride ions into concrete is the main diffusion process and has been the subject of intensive research in recent years [11,12].

3.2.3 Permeation

Permeation describes the movement of fluids through a saturated porous material under the action of an externally applied pressure head. Permeability is that material property of a porous medium which characterizes the ease with which a fluid will pass through it due to permeation. The term "permeability" has been used fairly loosely to refer to all fluid transport mechanisms but should only refer to saturated flows caused by an applied pressure differential.

Under saturated conditions flow will only occur if an applied pressure differential exists and the flow velocity is given by Darcy's law.

$$v=-K\frac{dh}{dL} \dots\dots\dots\dots\dots\dots\dots\dots (3.18)$$

where K is the coefficient of permeability and dh/dL is the hydraulic gradient. Expressing the flow velocity in terms of the pressure differential gives the following equation.

$$v=-\frac{k}{\mu}\frac{dp}{dL} \dots\dots\dots\dots\dots\dots\dots\dots (3.19)$$

where k is the intrinsic permeability coefficient of the material, μ the fluid viscosity and dp/dL the pressure differential. The intrinsic permeability coefficient is dependent on material charateristics and not fluid properties, making it a useful method of defining concrete for permeation. The intrinsic permeability coefficient may be related to the Darcy coefficient of permeability by the following equation:-

$$k=\frac{K\mu}{\rho g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.20)$$

The permeability of concrete is dependent on the concrete pore structure, the characteristics of the permeating fluid and the applied presssure differential. A multitude of permeability tests has been developed. These permeability tests measure the permeability of concrete to different gases and liquids and employ either throughput, input or falling head permeability techniques. Permeation does not constitute a major transport mechanism in marine concrete but permeability tests do provide useful information with regard to durability and are particularly sensitive to changes in the large pore fraction caused by cracking and other defects [13]. Gas permeability may also be used as an accelerated test for determining the ease of gaseous diffusion through concrete and can be used to predict carbonation resistance of concrete [14]. Figure 3.4 shows the permeation process through concrete exposed to an external pressure differential.

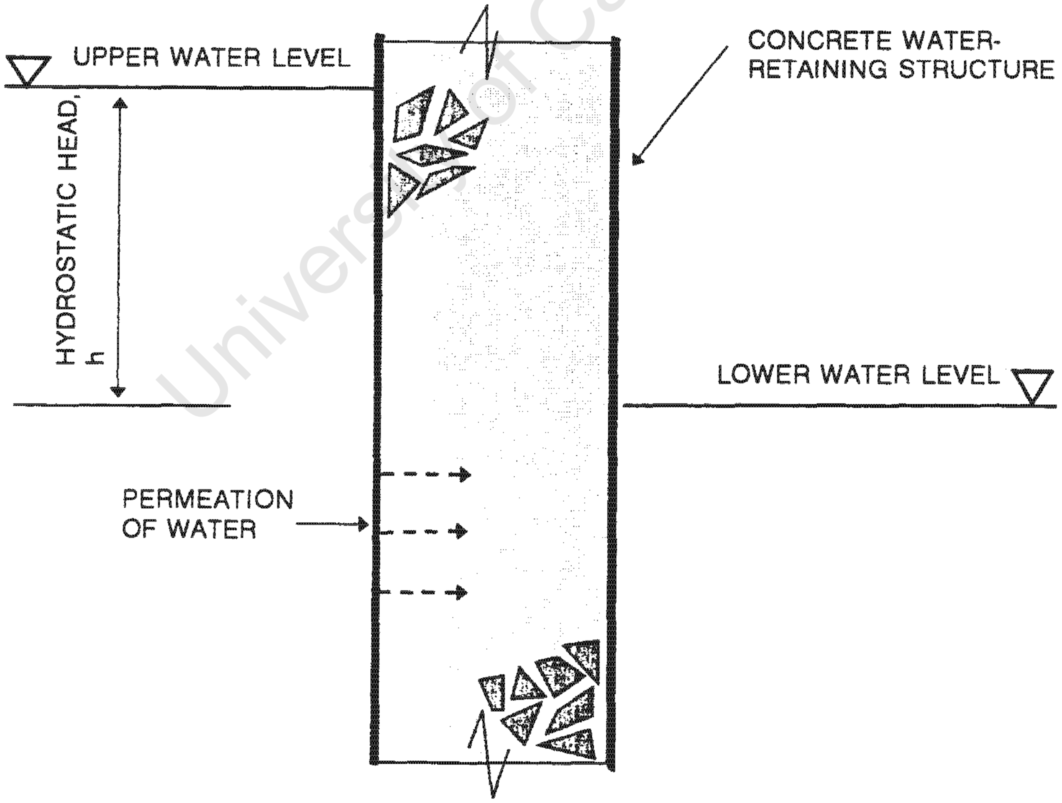


Figure 3.4 : Permeation of Water Through Concrete

3.3 Durability Index Tests for Marine Concrete Structures

A number of intrinsic durability tests has been developed to measure transport mechanisms such as absorption, diffusion and permeation through concrete. Sophisticated equipment and lengthy test periods are generally required to model these mechanisms accurately. The tests have provided useful research information but have limited practical application for site concrete given the rigorous constraints of the test methods. In response to the need for more practical durability tests a number of in situ and accelerated laboratory tests have been developed. The suitability and soundness of these new durability tests required validation and a rational system of characterizing the early age properties of concrete was undertaken as part of this work.

A system of durability index tests which appeared to solve many of the problems associated with intrinsic and accelerated durability tests has been developed by Ballim [14]. The approach was subject to several practical constraints and decisions had to be made about factors such as the age of testing, whether in situ or laboratory testing will be done, size of samples to be tested, what preconditioning will be done to the concrete and how the results will be related to long-term durability performance. The basic philosophy of characterising concrete using durability index tests was largely developed by the work of Ballim in his PhD thesis 'Curing and the Durability of Concrete' [14]. The strategy employed in characterising concrete may be summarised as follows :-

a) Laboratory tests on cored samples from structures or laboratory concrete are selected rather than in situ tests. This was done because durability index tests performed under controlled laboratory conditions produce more reliable results than in situ tests. The material can be characterized using laboratory samples while constructional factors can be investigated from site cored samples.

b) Concrete samples are cored from the outer skin of concrete so that the resistance of the "covercrete" to transport mechanisms can be measured. Testing the outer surface layer enables the effect of curing and surface defects such as cracks to be assessed in terms of the protection this layer provides to the internal concrete. Samples are standardized at a diameter of 68 mm and a thickness of 25 mm to fit into purpose built test rigs used for durability index testing. The relatively small sample size also represents minimal disruption to site elements, and yet is large enough to cater for normal aggregate sizes (19 mm) used in concrete.

c) Concrete is preconditioned at 50°C and 10 - 15 % R.H. to constant mass to ensure standard moisture conditions at the start of testing. The method of drying does cause some microstructural damage to the concrete but less severe drying regimes were found to be too slow for practical purposes. It is also believed that the outer skin of concrete structures could be exposed to similar drying under extreme environmental conditions which would cause microstructural damage to concrete in service.

d) Durability index test samples are extracted after 28 days to conform with standards for concrete cube strength testing.

e) Durability index tests used are based on transport mechanisms and have a sound theoretical basis. No indirect tests for durability, such as pullout tests, are used due to the unreliability of these methods.

f) Durability index tests are designed to be simple to operate, inexpensive and relatively quick to run. This is done so that the methods can be incorporated as performance specifications for construction and used in site laboratories.

The overall approach to measuring durability related properties is to extract concrete cores 28 days after casting, precondition the material and then run durability index tests under controlled conditions in the laboratory. Results can then be used for comparative purposes and to assess the potential durability of the material.

3.3.1 Sorptivity Testing

A number of general absorption tests have been developed for concrete where samples are immersed in water and the mass of water absorbed is used as a measure of the absorption ability of the material [15]. These tests do not have any theoretical basis and measure the bulk and surface properties as opposed to only the surface layer of concrete affected by wetting and drying action. Empirical tests for measuring absorption have gradually been replaced with one dimensional absorption tests where the sorptivity value is determined.

Ho and Lewis introduced the concept of sorptivity to concrete materials using a series of concrete prisms which were exposed to continuous water spray and samples split at intervals to determine the depth of water penetration [16]. Capillary rise tests have also been developed where the movement of the water front is measured either visually or by weighing the sample at intervals [17,18]. Kelham developed a sorptivity test where the absorption was measured automatically by suspending the sample in water and measuring the weight gain using a data logger [19]. The method proved to be reliable as external effects such as evaporation and two dimension water flow were virtually eliminated [20]. Measuring sorptivity using Kelham's method does have certain drawbacks however such as the lengthy preparation of the sample, the need for sophisticated equipment to monitor multiple samples and the duration of testing can exceed 24 hours. Ballim proposed a simplification of Kelham's sorptivity test which requires little equipment and allows rapid testing of concrete samples [21].

Details of the sorptivity test arrangement developed by Ballim are shown in Figure 3.5. Concrete samples are initially preconditioned at 50°C and 15 % R.H. to ensure suitably low moisture contents at the start of the test. A reservoir is created above the external surface of the core and a water layer of approximately 3 mm is placed on the concrete surface and at pre-determined time intervals the water is poured off and the sample weighed to determine the weight of water absorbed. Mass measurements are taken at intervals up to 64 minutes after which the concrete sample is vacuum saturated under water to determine the water saturated weight. The sorptivity of the material may be determined from the plot of mass of water absorbed versus the square root of time. The slight reservoir on top of the concrete does introduce some minor hydraulic pressures and recommendations have been made to use an upflow water absorption method with only the bottom surface in contact with water which simplifies the sample preparation and test procedure.

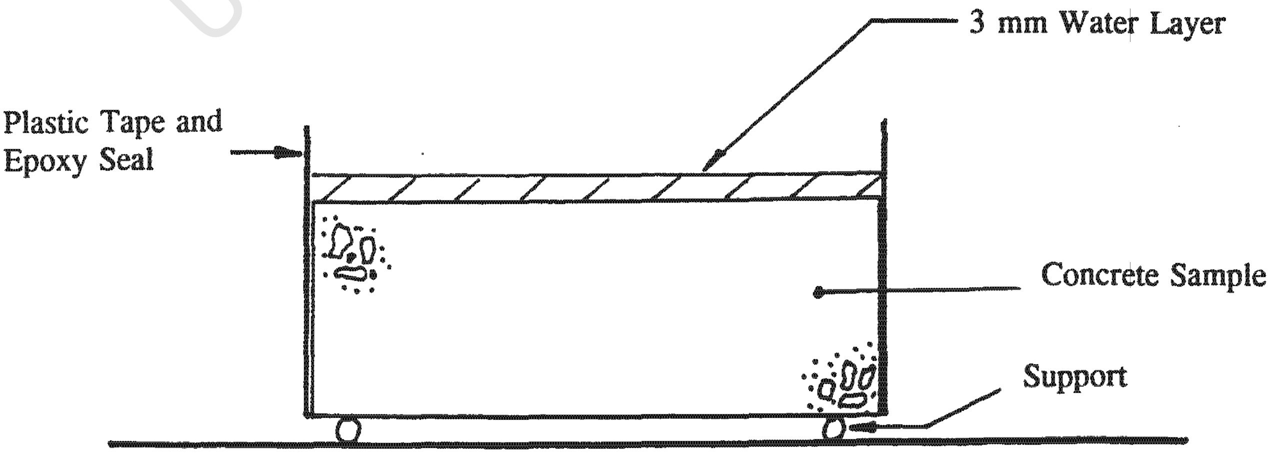


Figure 3.5 : Water Sorptivity Apparatus [21]

The theoretical basis of the method can be shown using the basic water absorption equation.

$$i=St^{1/2} \tag{3.21}$$

where i is the depth of infiltration, S the sorptivity and t the elapsed time. Expressing the equation in terms of the mass change during absorption results in the following equation.

$$M_t=An\rho_w St^{1/2} \tag{3.22}$$

where M_t is the change of mass of the sample after time t, A is the cross-sectional area of flow, n is the effective porosity and p_w is the density of water. The effective porosity is determined as follows.

$$n=\frac{(M_{sat}-M_0)}{(Ad\rho_w)} \tag{3.23}$$

where M_{sat} and M_0 are the saturated and dry masses of the sample respectively and d is the sample thickness. The sorptivity of the material may therefore be expressed as follows.

$$S=\frac{M_t}{t^{1/2}}\cdot\frac{1}{An\rho_w} \tag{3.24}$$

The quantity $M_t/t^{1/2}$ may be determined from the slope of the line produced when the mass of water absorbed is plotted against the square root of time. The sorptivity may therefore be calculated from the equation.

$$S=\frac{\text{Slope}}{An\rho_w} \tag{3.25}$$

The slope of the line produced from the plot of mass of water absorbed versus square root of time is determined by linear regression with the degree of fit being good as shown by the linear regression correlation coefficient usually being greater than 0.99. Improved accuracy is produced by eliminating the first one minute absorption from the regression analysis as initially there was a period of rapid absorption due to surface tension and roughness effects. Parrott argued that assumptions such as constant concrete porosity with depth and a sharp liquid front are not strictly true and will affect sorptivity results [18]. Experimental work has shown that these effects are not significant for fair and good quality concrete. Future modifications will result in the outer 5 mm surface layer being removed to eliminate surface variations.

Experimental results correlate well with theoretical predictions for different types of concrete. Typical results produced during this research work from the sorptivity test for OPC concrete are shown in Figure 3.6. The test is sensitive to surface effects and care must be taken to remove deposits from curing compounds and shutter release agents that can adversely affect the results. These problems may be obviated by cutting or grinding off the outer 5 mm layer of the concrete. Temperature and humidity should also be controlled during the test to minimise external effects.

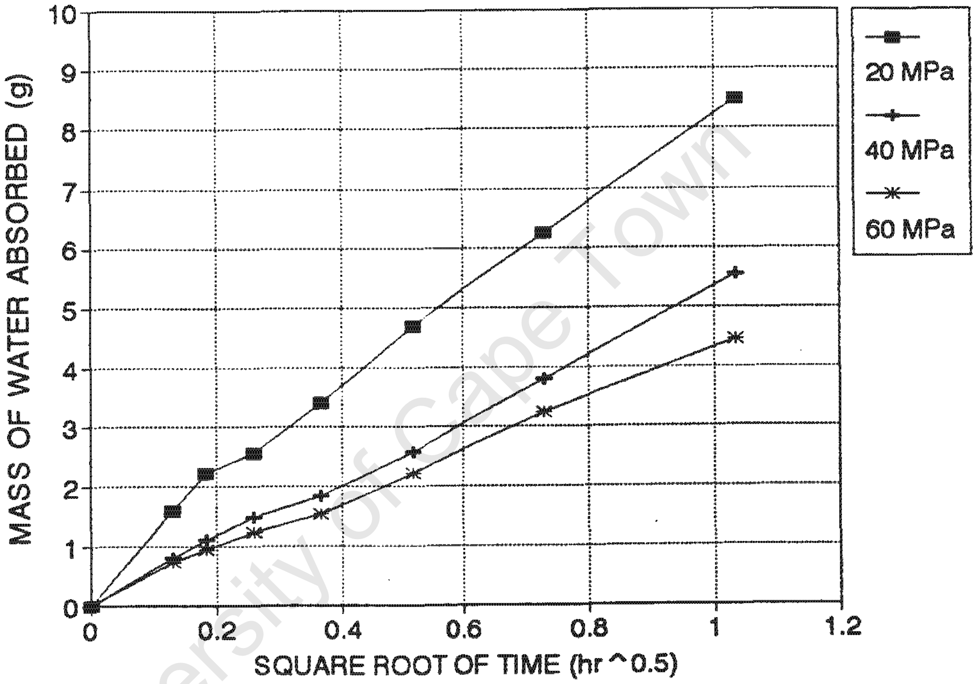


Figure 3.6 : Typical Water Absorption Curves For OPC Concrete

3.3.2 Diffusion Testing

Chloride diffusion through concrete is the main diffusion process of interest in marine concrete and reference in this section is only made to this form of diffusion. Classical diffusion tests where concrete is exposed to a high and low concentration of the diffusing species on opposite faces have been used successfully to measure the coefficient of diffusion. The diffusion process is too slow for laboratory testing even when very high concentrations are used at the upstream face. Diffusion tests involving chlorides through concrete may take over six months before significant penetration has occurred. Two basic approaches have been developed to obtain rapid estimations of the diffusivity of concrete: accelerated diffusion tests using an applied electric field, and resistivity measurements.

Early research showed that ionic migration is accelerated by the application of electric fields and this forms the basis of several corrosion protection techniques such as electrochemical chloride extraction of concrete. Whiting developed a rapid chloride diffusion test where a 60 V potential difference was applied across a concrete sample for six hours and the total charge passed was measured [22]. The test has been criticised because steady state conditions may not be achieved during the test, the total current passed represents the total ionic movement and not the chloride flow, heat may be generated due to the high voltage and gases are evolved at the electrodes. Tang and Nilsson proposed a similar test but with a potential difference of 30 V and where the chloride penetration was chemically analysed on completion of the test [23]. The development was a technical improvement on Whiting's test but the test became more time consuming due to the extra chemical analysis required. Dhir et al developed an accelerated test which ran at 10 V and allowed steady state conditions to be achieved [24]. The test took up to two weeks to run however and involved repetitive chemical analysis of the downstream cell for chlorides.

A fundamental weakness of these rapid chloride diffusion tests is the assumption that the applied potential difference accelerates the diffusion process. This assumption is not valid and Streicher and Alexander have shown that the potential difference increases the flux of ions due to the mechanism of conduction while the diffusion process is unaffected [25]. Techniques such as the AASHTO rapid chloride test measure a combination of diffusion and conduction resulting in poor correlations between the inherent diffusivity of the material and the test results obtained. The tests may also produce erroneous results due to non-steady state conditions being measured and prolonged immersion causing continued cement hydration during the test. Concern has also been expressed about the side effects such as heat generation and gas evolution, produced when using high potential differences of 30 V or more [26].

Feldman et al proposed a resistivity test for determining the diffusivity of concrete where a potential difference is applied across a concrete sample in the absence of a diffusing medium and the instantaneous current is measured [27]. Resistivity measurements are dependent on the diffusivity of the material, moisture content, carbonation and the pore water conductivity. Concretes with inherently high pore water conductivities due to mobile ions such as sodium and potassium would therefore be unfairly rated using this type of approach.

Streicher has developed a rapid chloride conductivity test at the University of Cape Town in which virtually all ionic flux occurs by the process of conductivity due to a 10 V potential difference [28]. The apparatus consists of a two cell conduction rig shown in Figure 3.7. Each cell contains a 5M NaCl solution so that there is no concentration gradient across the sample and all chloride migration is the result of conduction from the applied potential difference. The concrete core sample in the test is 68 mm diameter and 25 mm thick. The sample is initially preconditioned by drying at 50°C and 15 % R.H. to constant mass before being vacuum saturated in a 5M NaCl solution for two hours. This preconditioning is done to ensure that all concrete has practically the same pore solution conductivity when tested. The conductivity of the concrete is determined by applying a 10 V potential difference across the sample and immediately measuring the current passing through the concrete.

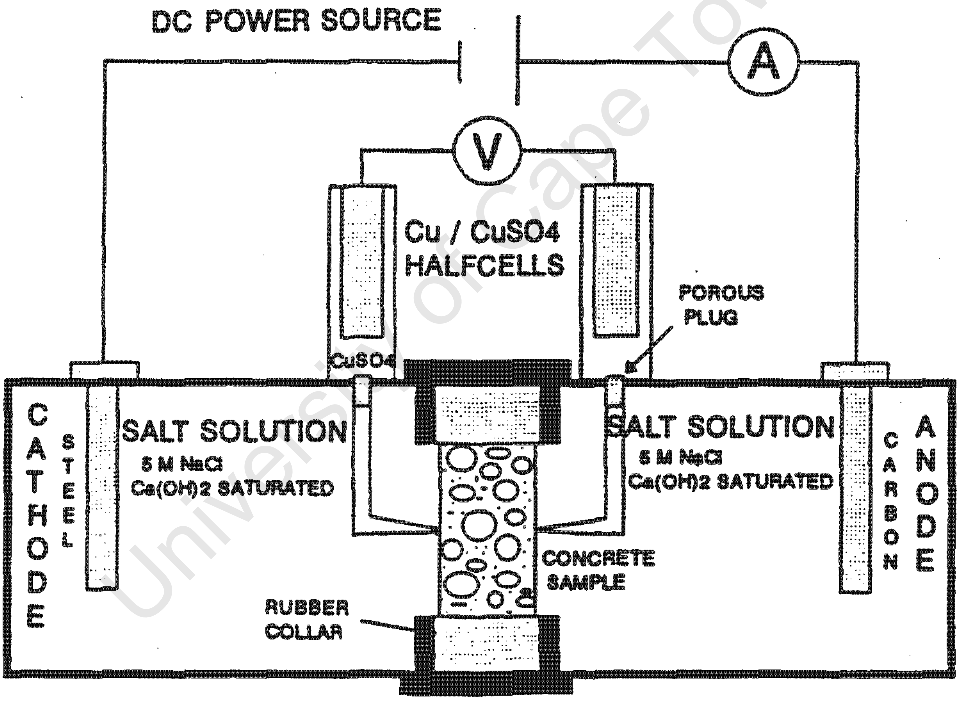


Figure 3.7 : Chloride Conductivity Apparatus [28]

The conductivity of the sample may be defined as follows:-

$$\sigma = \frac{i}{V} \cdot \frac{t}{A} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.26)$$

where σ is the conductivity of the sample (mS/cm), i is the current (Amps), V is the potential difference (Volts), t is the thickness of the sample and A is the cross-sectional area. The relationship between diffusivity and conductivity may be defined as follows.

$$Q=\frac{D}{D_0}=\frac{\sigma}{\sigma_0}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.27)$$

where Q is the diffusibility of the material, D is the diffusivity of the ion through the material, D_0 is the diffusivity of the ion through the pore solution, σ is the conductivity of the material and σ_0 is the conductivity of the pore solution.

3.3.3 Permeability Testing

A variety of permeability tests have been developed for concrete including input, throughput and falling head types while permeating fluids include gases and liquids. Most permeability tests have been developed for laboratory use only due to the extensive equipment required for many of the methods. Comparisons between test methods are difficult because the tests measure different intrinsic and empirical properties related to permeability.

Input permeability tests, also called water penetration tests use a high pressure to force water into concrete for a fixed period of time and the depth of water penetration is measured visually by splitting the sample at the end of the test [29]. These tests have been criticized because the length of contact time between concrete and water allows further cement hydration and the visual determination of water penetration is not sufficiently accurate.

Throughflow permeability tests attempt to measure the Darcy coefficient of permeability by forcing a fluid through concrete under a constant pressure head while monitoring the pressure gradient or amount of flow through the sample. Maintaining a constant pressure head and ensuring steady state conditions where inflow equals outflow requires sophisticated instrumentation and monitoring equipment [30]. Throughflow permeability tests using liquids are extremely slow and flow rates are very low. Tests with gases are more rapid but measurement of the amount of permeating gas is difficult. These constraints made the throughflow permeability test impracticable for repetitive testing.

Falling head permeaters apply a pressure to one side of a concrete sample and the pressure decay is monitored as the fluid permeates through the material. The pressure head is not held constant but allowed to fall as permeation occurs. Regular pressure measurements are taken during the permeation process to record the pressure decay curve with time. No sophisticated equipment is required and the testing is fairly rapid when using gas as the permeating fluid. Blight developed a simple and inexpensive falling head permeameter for measuring the ~~permeability of road asphalt mixes [31]. Schonlin and Hilsdorf have~~ developed a rising pressure head apparatus with an initial vacuum being applied that also seals the sample to the test apparatus and can be used for in situ testing [32].

Ballim made several modifications to the Blight apparatus in developing a falling head permeameter for measuring the oxygen permeability of concrete [33]. Details of the apparatus are shown in Figure 3.8. In the test oven-dried concrete samples are subjected initially to a 100 kPa pressure using oxygen and the pressure decay is monitored until a pressure of 50 kPa is reached. The coefficient of permeability is determined from the slope of the line produced when the log of the ratio of initial pressure to actual pressure is plotted against time. The test was automated at the University of Cape Town by using pressure transducers linked to a computer based data logger which was especially useful with high performance concretes which could run for over 24 hours.

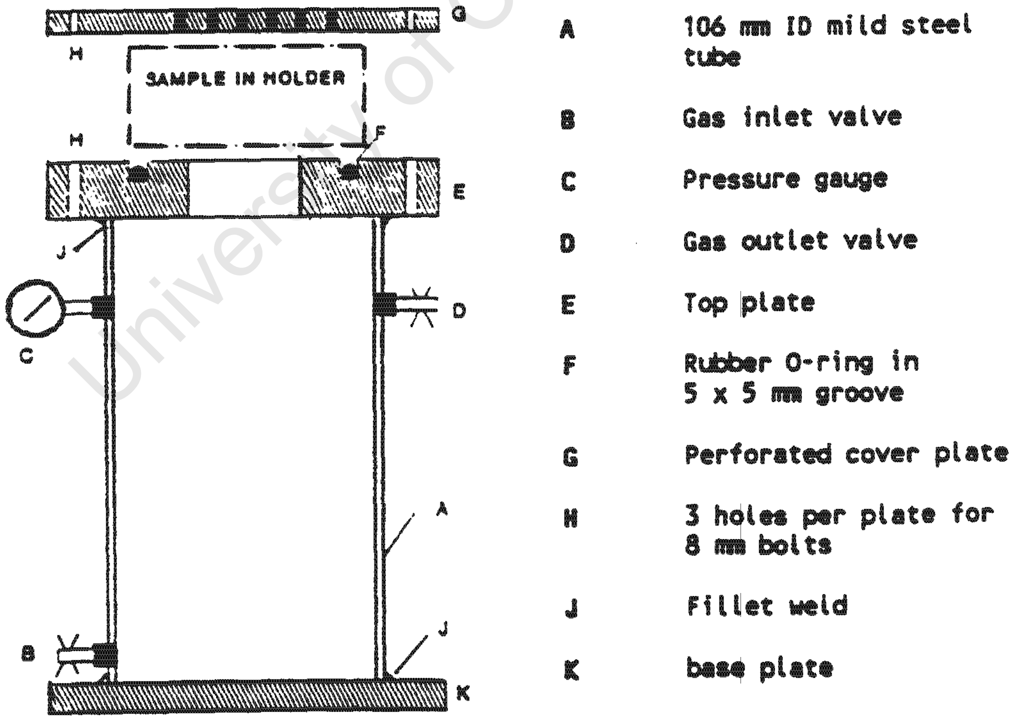


Figure 3.8 : Oxygen Permeability Apparatus (Ballim) [33]

Ballim has shown that the rate of pressure decay of a falling head permeameter is governed by the Darcy equation such that the coefficient of permeability k is given as follows.

$$k=\frac{\omega Vgd}{RA\theta t}.\ln(P_0/P)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.28)$$

where w is the molecular mass of the permeating gas, V is the volume of the permeameter, g is the acceleration due to gravity, R is the universal gas constant, A is the cross-sectional area of the sample, d is the sample thickness, Θ is the absolute temperature, t is the time for the pressure to decrease from P_0 to P, P_0 is the pressure at the beginning of the test and P is the pressure at time t. The coefficient of permeability is an unwieldy exponential number and it was therefore simplified by defining a oxygen permeability index as follows:-

$$\text{Oxygen Permeability Index} = -\operatorname{Log}(k)\ \ldots\ldots\ldots\ldots(3.29)$$

Extensive experimental work by Ballim verified the theoretical relationship used in determining the cofficient of permeability for the falling head permeameter. Linear regression correlation coefficients of greater than 0.99 are commonly found for oxygen permeability results. The test was selected as a durability index test for measuring permeation due to its accuracy, simplicity and relative speed as compared with other permeability tests. Gas permeability tests have the advantage of being able to be used as an accelerated durability test for gas diffusion processes such as carbonation.

3.3.4 Other Methods of Characterizing Concrete

Three commonly used tests of concrete quality were also used in the concrete characterization investigation so that comparisons could be made with the selected durability index tests. Core strength testing was done to assess the grade of the material although it was accepted that strength is a poor indicator of potential durability. Abrasion resistance was measured to determine the physical resistance of the concrete surface to abrasive action. Resistivity measurements were recorded to compare with chloride conductivity results and were also used because the method allows rapid non-destructive testing to be done.

Cores used for strength testing were 68 mm diameter to conform with the standard diameter used for the durability index tests. The outer 25 mm of the cores was used for durability index testing and the internal section of the core from 30 to 90 mm was tested for core strength in accordance with BS 1881 [34]. The curing-affected outer 30 mm was therefore not tested for strength and the results represent the internal concrete strength.

Abrasion resistance testing of concrete was done in accordance with ASTM C418-89 where high pressure sand blasting was used to assess the physical resistance of the concrete surface [35]. It was necessary to have a test which measures physical resistance to compliment the other tests which are related to the chemical resistance and pore structure of the concrete. Marine concrete may be exposed to severe abrasion due to heavy wave action and high abrasion resistance is essential to ensure adequate performance. Testing for abrasion resistance was done on concrete core samples used for durability index testing. The cores were of sufficient size (68 mm diameter) to allow two tests per sample.

Resistivity is dependent on the concrete moisture content, the diffusivity of the material and the pore solution conductivity. Rapid in situ readings were obtained using a Wenner probe arrangement with four equally spaced probes connected to a digital resistivity meter [36]. An alternating current passes through the outer pair of contacts and the resulting voltage drop between the inner contacts used to determine the resistivity. Resistivity measurements are affected by carbonation, wetting fronts and highly conductive surface layers which can cause erroneous results. By measuring concrete at early age under controlled laboratory conditions it was possible to largely eliminate these influences.

3.4 Characterizing Concrete For Potential Durability

Characterizing concrete for durability may be defined as the process whereby early age properties of concrete related to durability of the material are assessed using durability index tests. The effectiveness of the concrete characterisation process using durability index tests can only be determined by measuring the durability performance of concrete over time. Durability index tests must be sensitive to the effects of factors such as curing, compaction and chemical composition which are known to have a marked effect on durability.

3.4.1 Concrete Materials, Mixes and Curing

In order to assess the sensitivity of concrete characterisation tests to concrete quality a wide range of concrete types was investigated. Variables investigated included the following :-

Concrete grade - 20 MPa
40 MPa
60 MPa
Concrete type - 100% OPC
70% OPC and 30% fly ash
50% OPC and 50% slag
Curing regime - Continuously wet cured at 23 °C for 28 days
7 days moist curing (90% R.H. and 23 °C), 21 days outdoors
7 days dry curing (50% R.H. and 23 °C) 21 days outdoors

Fly ash and slag were used in the concrete because these materials are gaining acceptance in marine concrete applications and their performance could be compared with OPC concrete. The chosen replacement levels of cement with fly ash and slag were those currently being used in South Africa. The chemical composition and physical properties of the OPC, fly ash and slag used are given in Table 3.1. OPC used in this work was from Riebeek West cement factory and may be described as a low alkali cement with a C_3A content of 4.5 %.

Table 3.1 : Analysis of OPC, Fly Ash and Slagment

COMPOUND /PROPERTY	Ordinary Portland Cement	Lethabo Classified Fly Ash	Vanderbijl Park Slagment
SiO_2	21.5%	54.1%	34.2%
Al_2O_3	3.8%	32.9%	16.5%
Fe_2O_3	3.3%	3.3%	0.4%
TiO_2	0.2%	1.7%	1.9%
SO_3	2.3%	0.4%	1.4%
MnO	0.1%	0.0%	0.8%
MgO	0.8%	1.3%	10.4%
CaO	65.1%	4.7%	32.8%
Na_2O	0.3%	0.6%	-
K_2O	0.5%	0.6%	0.9%
P_2O_5	0.1%	-	-
Loss on Ignition	2.3	0.5	-
Blaine Fineness	2835	4300	3770

Fine and coarse aggregate selected were Cape Flats dune sand and 19 mm greywacke respectively. These aggregates represent those generally used for concrete in the Cape Town area. The sand is a poorly graded material with a low fines content and a high shell content from its origins as a dune deposit. The stone is commonly referred to as Malmesbury shale and is known to be highly reactive in the presence of excessive alkalis. Concrete containing the reactive aggregate is known to exhibit alkali aggregate reaction expansion for alkali contents above 2.1 kg/m^3. The stone is also very angular and may result in harsh and unworkable mixes. Details of the properties of the fine and coarse aggregates are shown in Table 3.2.

Table 3.2 : Properties of Fine and Coarse Aggregates

Material Property	Cape Flats Dune Sand	19 mm Greywacke Stone
Loose Bulk Density	1492 kg/m^3	1341 kg/m^3
Compacted Bulk Density	1689 kg/m^3	1515 kg/m^3
Relative Density	2.63	2.68
Fineness Modulus	1.92	7.15

Concrete mixes were designed using the standard PCI method with the details of the mixes being shown in Table 3.3 [37]. Both fly ash and slag were found to have significant water reducing properties resulting in lower water requirements of up to 23 l/m^3 for fly ash and 15 l/m^3 for slag. These major reductions in water requirement were partly due to the harsh shaped aggregate being used and partly due to the improved workability of fly ash and slag concrete mixes. All mixes were designed for a standard slump of 50 mm and concrete mixes were accepted with slumps between 25 and 75 mm.

Table 3.3 : Concrete Mixes For OPC, Fly Ash and Slag Concrete

CONCRETE TYPE	CONCRETE MATERIALS	Grade 20 MPa (kg/m³)	Grade 40 MPa (kg/m³)	Grade 60 MPa (kg/m³)
OPC ONLY CONCRETE	Cement	240	346	515
	Sand	862	794	637
	Stone	1050	1050	1050
	Water	200	192	198
	Water : Cement Ratio	0.83	0.56	0.38
30% FLY ASH CONCRETE	Cement	173	253	370
	Fly Ash	74	108	158
	Sand	843	762	580
	Stone	1102	1102	1102
	Water	177	168	179
	Water : Cement Ratio	0.71	0.46	0.34
50% SLAG CONCRETE	Cement	116	176	269
	GGBS	116	176	269
	Sand	872	782	596
	Stone	1080	1080	1080
	Water	185	180	189
	Water : Cement Ratio	0.80	0.51	0.35

Concrete blocks of size 300x300x450 mm were cast in steel shutters and compacted in two layers using a poker vibrator. A steel mat of 8 mm diameter mild steel bars was included in the bottom of each block with cover to reinforcement of 50 mm for long-term corrosion studies. After 24 hours the blocks were demoulded and exposed to either submerged wet curing at 23°C, six days of moist curing or six days of dry curing. Seven days after casting the moist and dry cured concrete blocks were stored outdoors under mild autumn weather (9-18°C and 70-95% R.H.) until being tested at 28 days. The blocks were characterized by extracting two cores from each block and tested using durability index tests. Only two cores were extracted because it was originally intended that cores would be extracted every six months for two years which, given the size of the block, allowed testing of only two cores at each test date. This had a detrimental effect on the reliability of the results since statistical outliers could not be discarded and the test variability could not be fully quantified. The variability and reliability of the characterization techniques had been investigated by other researchers who were involved in their initial development.

3.4.2 Experimental Results (After 28 days)

Analysis of concrete characterisation results revealed that the test methods had varying degrees of sensitivity to the grade of concrete, type of cement and extent of active curing. Details of the 28 day experimental results are given in Appendix 3.1.

a. Compressive Strength

Cube strengths were close to design strengths except for fly ash and slag concrete of grade 60 which were slightly in excess of the design strength. Figures 3.9a and 3.9b show the cube strengths for the nine concretes plotted against water/cement ratio and total binder content respectively. OPC concrete had higher compressive strength than either fly ash and slag concrete when comparing water cement ratios. When comparing the concretes on the basis of total binder content similar cube strengths were achieved due to the water reducing affect of fly ash and slag on the relevant concrete mixes. All compressive strength results were recorded after 28 days.

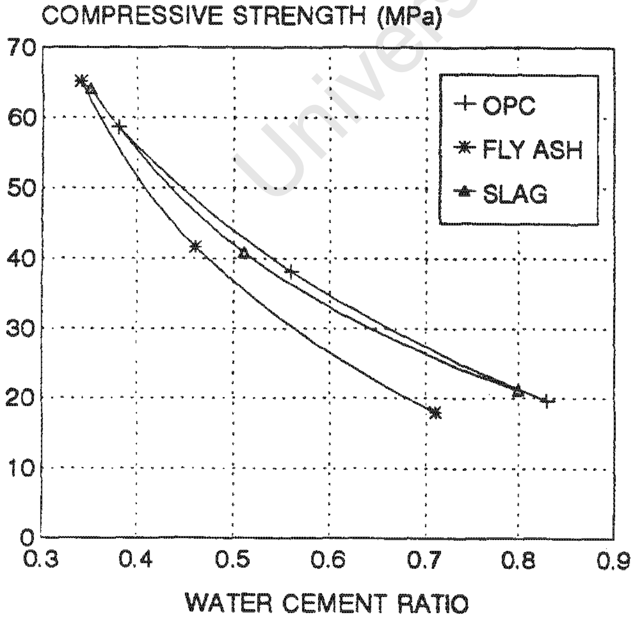


Figure 3.9a : Compressive Strength vs. Water / Cement Ratio

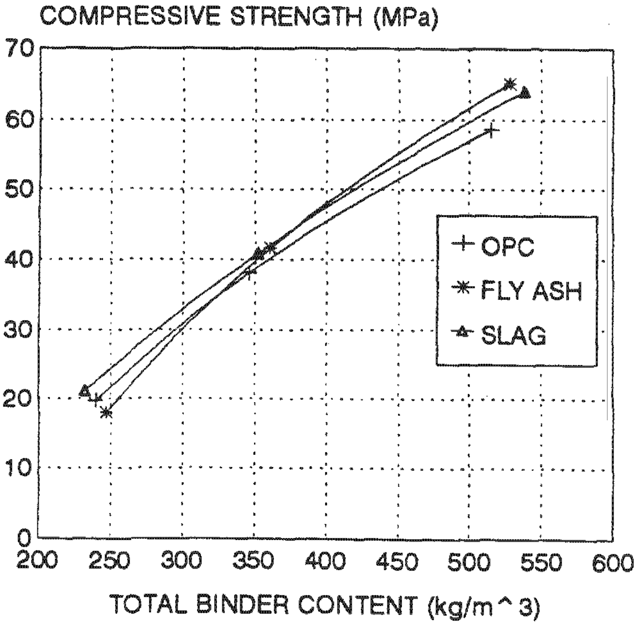


Figure 3.9b: Compressive Strength vs. Total Binder Content

Compressive strength is generally regarded as not having a low sensitivity to changes in pore structure caused by poor curing or compaction. Concrete cores were tested from internal concrete (30 - 90 mm) and it was therefore expected that the effects of different curing regimes would be minimal. OPC concrete was found to confirm this assumption with wet, moist and dry cured concrete all having similar core strengths. Fly ash and slag concrete were however significantly affected by the type of curing particularly for grade 40 and 60 concrete. Dry curing resulted in reduced strengths of as much as 18% for fly ash concrete and 15% for slag concrete when compared to similar wet cured concrete. Details of the cube and core strengths for OPC, fly ash and slag concrete are shown in Figures 3.9c-3.9e.

These results are significant because large concrete samples were used so that excessive drying could not have occurred due to a high surface area to volume ratio. Clearly the different amounts of curing significantly affected the extent of cement hydration resulting in microstructural changes in the concrete. Durability tests such as permeability and sorptivity which are sensitive to these microstructural changes were able to define these effects since surface samples were tested instead of internal samples.

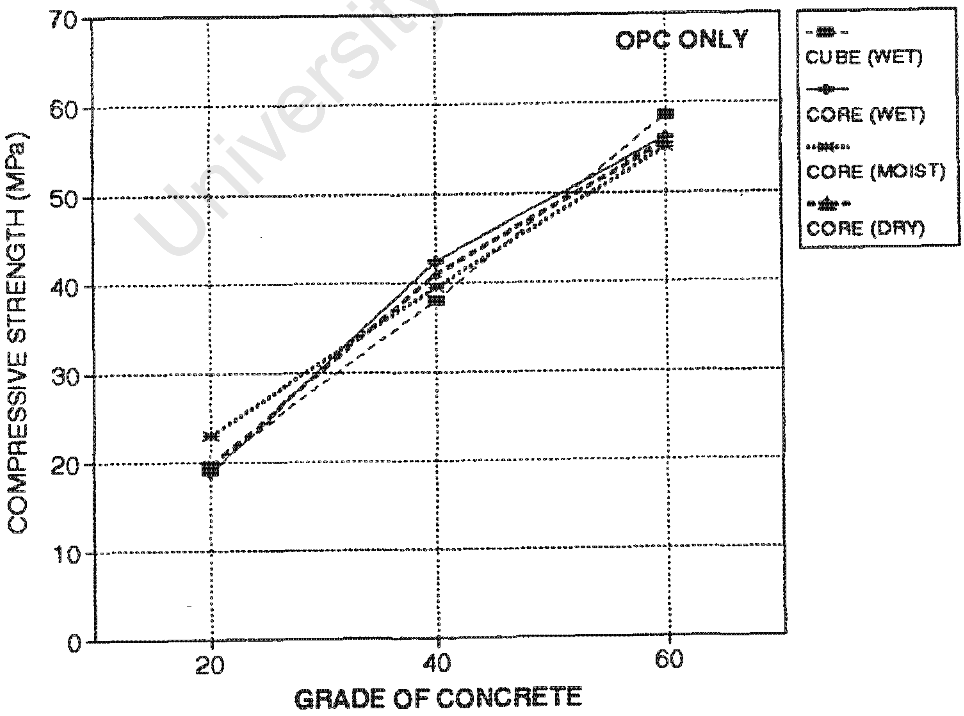


Figure 3.9c : Core Strength vs. Grade of OPC Concrete

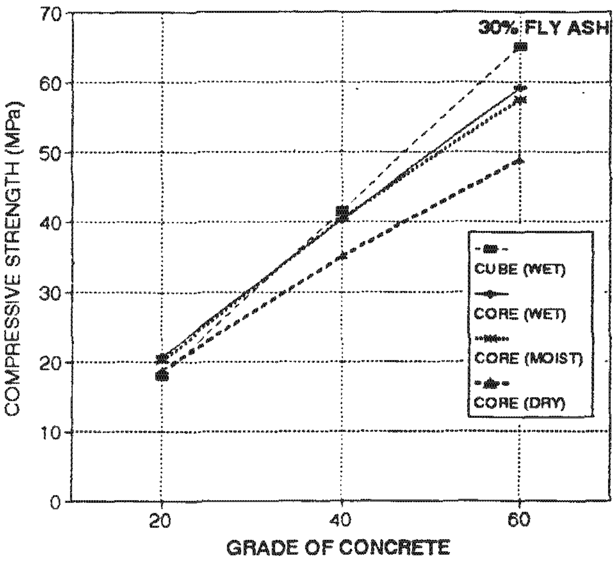


Figure 3.9d : Core Strength vs.
Grade of Fly Ash Concrete

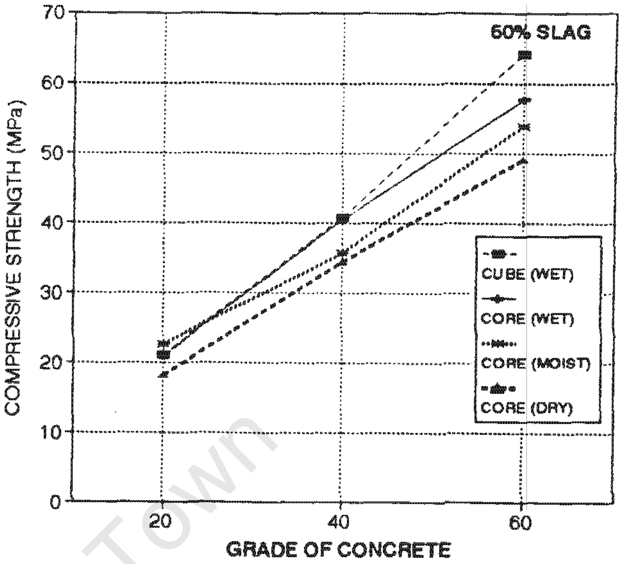


Figure 3.9e : Core Strength vs.
Grade of Slag Concrete

b. Oxygen Permeability

The permeability of concrete, as measured with the oxygen permeability test, reduced with increasing grade of concrete and extent of active moist curing. Fly ash and slag concrete were less permeable than OPC concrete when well cured but were more permeable than OPC concrete when dry cured. Figure 3.10a - 3.10c show the oxygen permeability index results for OPC, fly ash and slag concrete. The oxygen permeability test by its nature depends on the amount and continuity of larger pores in the concrete where most of the flow will occur and is less sensitive to the finer capillaries. The inherently finer pore structure of fly ash and slag concrete was therefore not fully reflected in the oxygen permeability results recorded at 28 days.

All three concrete types were affected by the quality of the initial curing with fly ash and slag concrete being particularly sensitive to poor curing. The effects of dry curing were generally equivalent to reducing the grade of the material as measured by the oxygen permeability index by 20 MPa. This means that a dry cured grade 60 material was equivalent to a wet cured grade 40 and similarly with grades 40 and 20 concrete. Comparisons between OPC, fly ash and slag concrete oxygen permeability results for moist cured concrete recorded at 28 days are shown in Figure 3.10d.

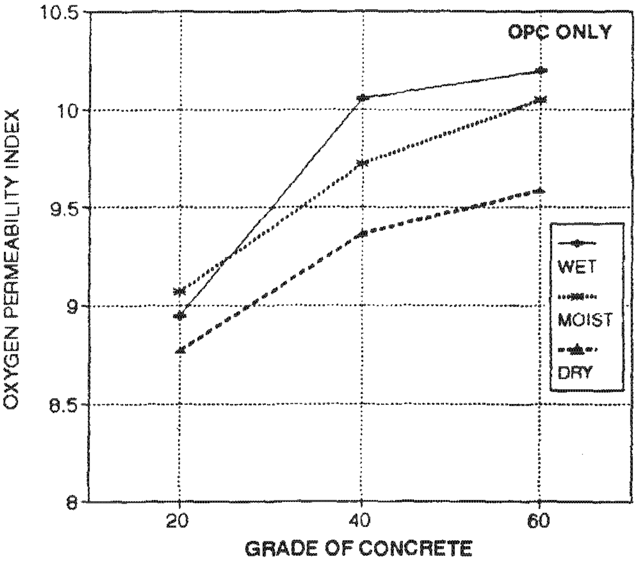


Figure 3.10a : Oxygen Permeability Index vs. Grade of OPC Concrete

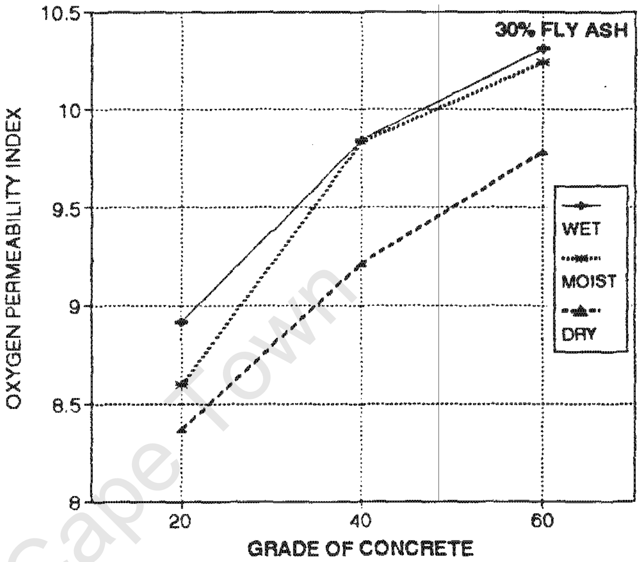


Figure 3.10b : Oxygen Permeability Index vs. Grade of Fly Ash Concrete

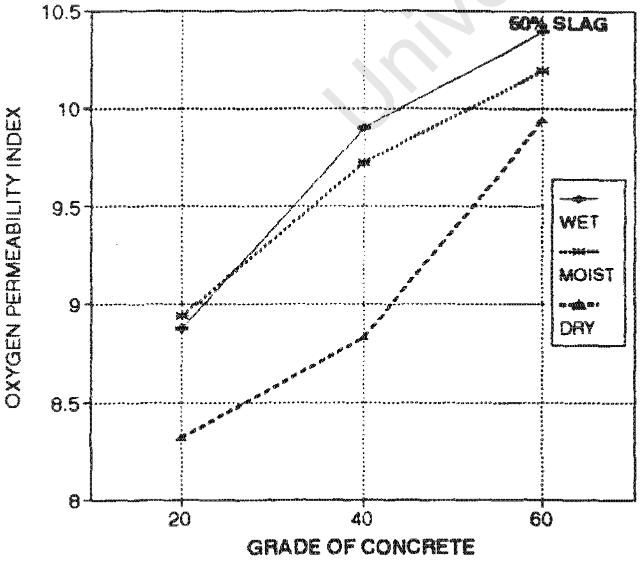


Figure 3.10c : Oxygen Permeability Index vs. Grade of Slag Concrete

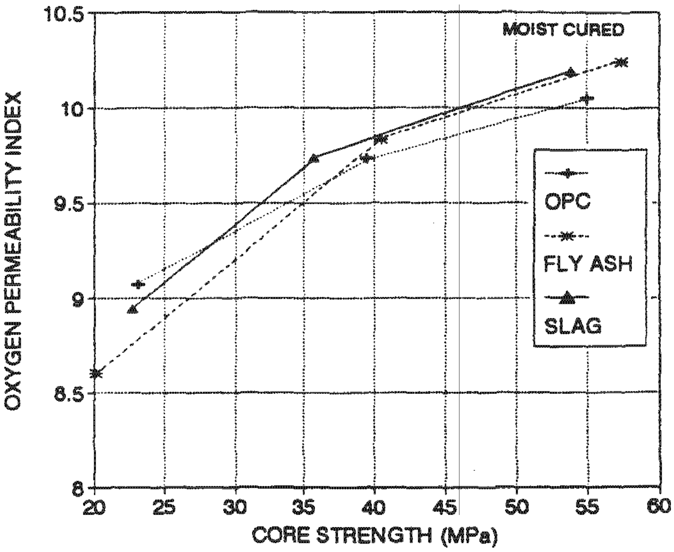


Figure 3.10d : Oxygen Permeability Index vs. Core Strength

Ballim conducted similar oxygen permeability testing using Witwatersrand materials and found OPC concrete to be more impermeable than either fly ash or slag concrete for both moist and dry cured concrete [14]. Grade 40 OPC concrete which was wet cured for 28 days had an oxygen permeability index of 10.40 compared to a value of 10.06 using Western Cape materials. The difference can be mainly attributed to the superior characteristics of the Witwatersrand aggregate which are well graded and shaped. Concrete made with angular greywacke stone and poorly graded Cape Flats sand resulted in excessive bleeding particularly for the grade 20 and 40 concrete. The presence of large bleed channels and lenses of bleed water trapped under coarse aggregate particles was thought to be responsible for the high permeability of OPC concrete. The presence of sufficient fines in fly ash and slag concrete mixes resulted in low bleed rates in the material thereby reducing the large capillaries.

c. Water Sorptivity

Water sorptivity results indicated that absorption rates of concrete reduced with increasing grade of concrete and the extent of active moist curing. Wet curing and moist curing produced similar sorptivity results while dry cured concrete had significantly higher sorptivity values. The sorptivity test essentially measures a surface phenomenon and should therefore be sensitive to early age drying effects which affect the microstructural porosity gradient in the concrete. Differences in sorptivity values for wet and dry cured concrete were between 25% and 70% and would indicate that the test method may be effective in assessing curing effectiveness on site.

OPC concrete had higher sorptivity values than either fly ash or slag concrete when moist or wet cured, showing the advantages that cement extenders can give when correctly treated. Dry cured concrete showed the opposite trend with fly ash and slag concretes having higher sorptivities than OPC concrete. The difference in sorptivity results between moist and dry cured concrete indicates how the technical benefits of using cement extenders can be lost if poorly treated on site. It should be noted that the dry curing regime used ie. 50% R.H. and 23°C is not considered extreme for South African standards where rapid drying may occur in summer with high temperatures and wind speeds coupled with low humidities. Details of the water sorptivity results for OPC, fly ash and slag concrete are shown in Figure 3.11a - 3.11c.

Dry curing had a marked effect on increasing sorptivity and could reduce the quality of a grade 60 concrete to that of a grade 30 concrete compared with wet cured concrete. The presence of a poor quality surface layer has major implications on the durability potential of the material even if the internal concrete is dense and unaffected by the poor curing. Grade 20 concrete had fairly high sorptivity values even with ideal curing conditions, indicating that good surface quality requires a dense material of medium to high grade. Comparing the moist cured sorptivity results it would appear that OPC and fly ash concrete had similar sorptivity results while slag concrete had lower sorptivity values. Figure 3.11d shows the comparisons between OPC, fly ash and slag concrete for moist cured concrete.

Ballim found that OPC concrete had lower sorptivity than either fly ash or slag concrete for grade 30 and 40 concrete and similar results for grade 50 concrete when cured underwater for 1, 3, 7 and 28 days [14]. Concrete surface samples 15 mm thick were used by Ballim for this testing. These results are contrary to the findings of other researchers who found that cement extenders such as fly ash, slag and silica fume reduced the sorptivity of well cured concrete [7,8,19,]. Ballim did show however that for internal slices at 15-30 mm and 30-45 mm from the concrete surface the sorptivity of fly ash and slag concrete was lower than OPC concrete.

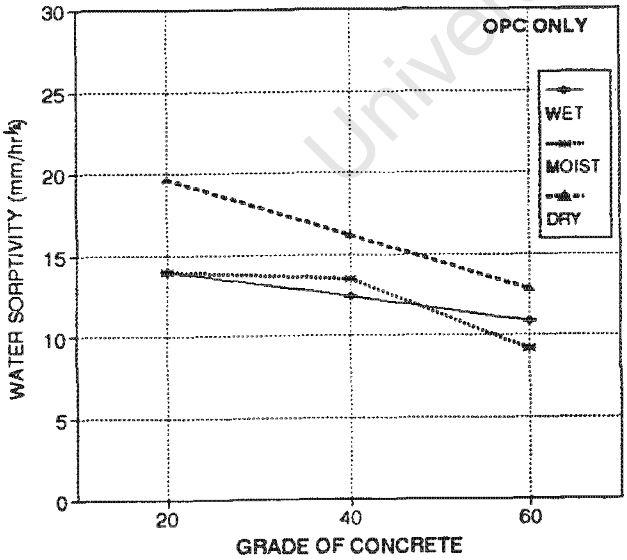


Figure 3.11a : Water Sorptivity vs. Grade of OPC Concrete

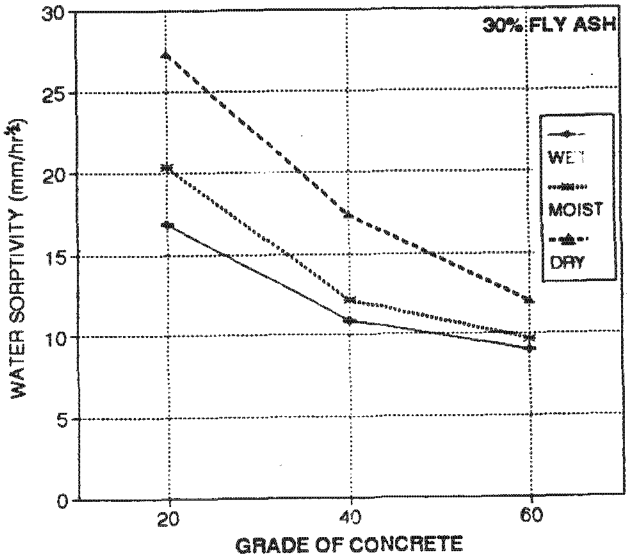


Figure 3.11b : Water Sorptivity vs. Grade of Fly Ash Concrete

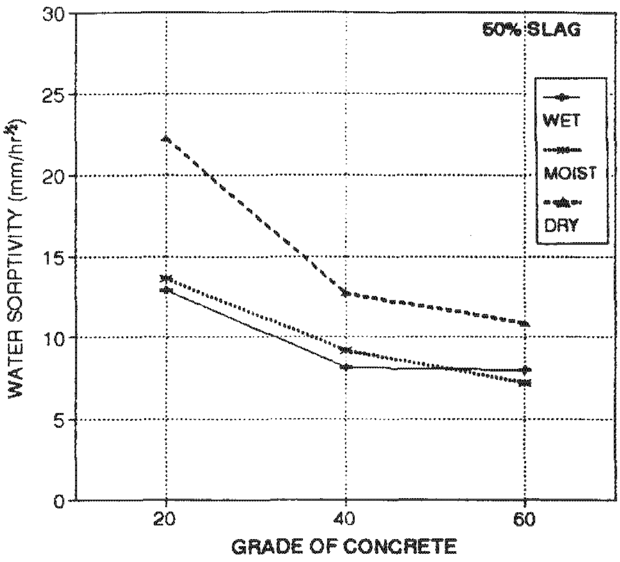


Figure 3.11c : Water Sorptivity vs. Grade of Slag Concrete

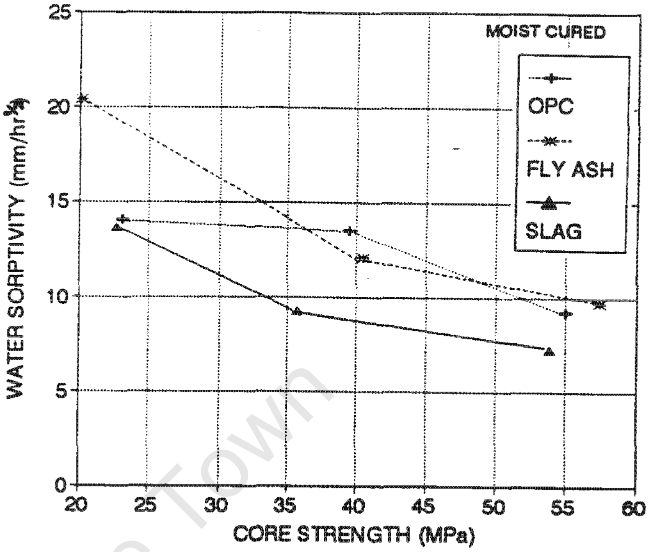


Figure 3.11d: Water Sorptivity vs. Core Strength

d. Chloride Conductivity

Chloride conductivity reduced with increasing grade of concrete but 28 day results was more affected by the extent of curing and the type of concrete. Moist cured concrete had slightly higher chloride conductivity than similar wet cured concrete while dry cured concrete had significantly higher chloride conductivity than either moist or dry cured concrete. Dry cured concrete had chloride conductivity values more than twice those of similar wet cured concrete particularly for fly ash and slag concrete. Chloride conductivity results for OPC, fly ash and slag concrete at 28 days are shown in Figure 3.12a - 3.12c.

The conduction process causes electromigration of chloride ions which move through all pores of sufficient size without favouring the larger pores as with the permeation process. The chloride conduction test therefore provides a good indication of the overall diffusivity of the material being sensitive to changes in pore structure which might appear to be insignificant when using the permeation process. Chloride conductivity was found to be extremely sensitive to pore structure changes caused by vaying amounts of curing and by using different types of concrete.

Chloride conductivity of well cured fly ash or slag concrete was lower than OPC concrete but when dry cured all three concrete types had similar chloride conductivity results. Dry curing increased the chloride conductivity of fly ash and slag concrete significantly but the inherently good chloride resistance of these materials meant that the results were no worse than similar OPC concrete. OPC concrete was less affected by curing but had relatively high conductivity values even when wet cured. Figure 3.12d shows the chloride conductivity results for moist cured OPC, fly ash and slag concrete where the advantages of cement extenders are clearly apparent.

Chloride conductivity results by Taylor et al showed similar relationships with extent of curing and type of cement [38]. Silica fume has also been shown to reduce the chloride conductivity of concrete substantially in comparison with OPC concrete [39]. High strength concrete of grade 75 with 10% condensed silica fume had extremely low chloride conductivity values in the region of 0.25 mS/cm. The chloride conductivity results represent the resistance of the concrete to chloride conduction at 28 days and do not allow for long-term effects such as continued hydration and chloride binding so direct comparisons between concrete types may be misleading.

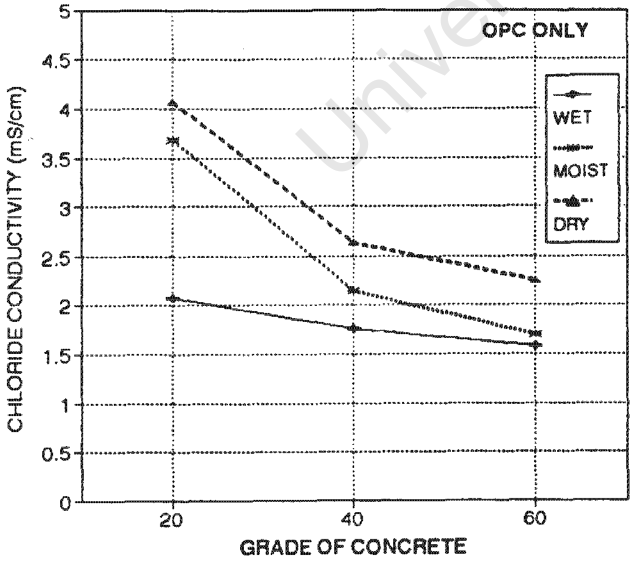


Figure 3.12a : Chloride Conductivity vs. Grade of OPC Concrete

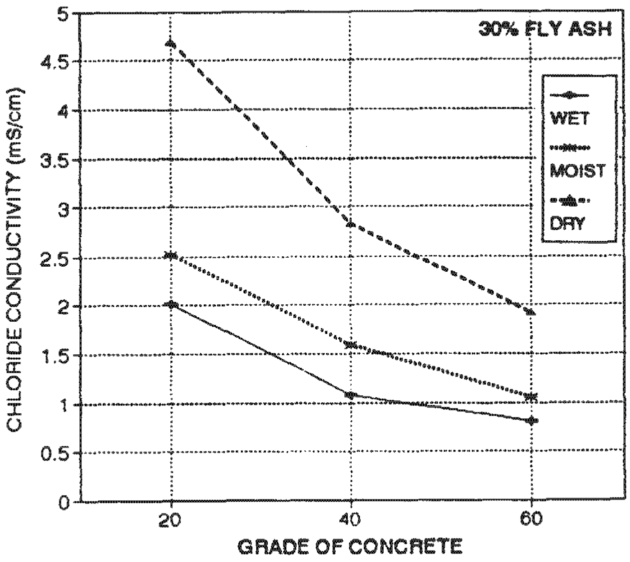


Figure 3.12b : Chloride Conductivity vs. Grade of Fly Ash Concrete

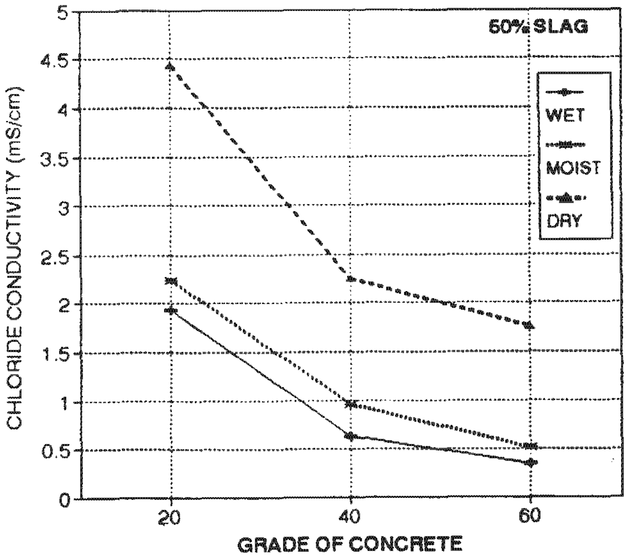


Figure 3.12c : Chloride Conductivity vs. Grade of Slag Concrete

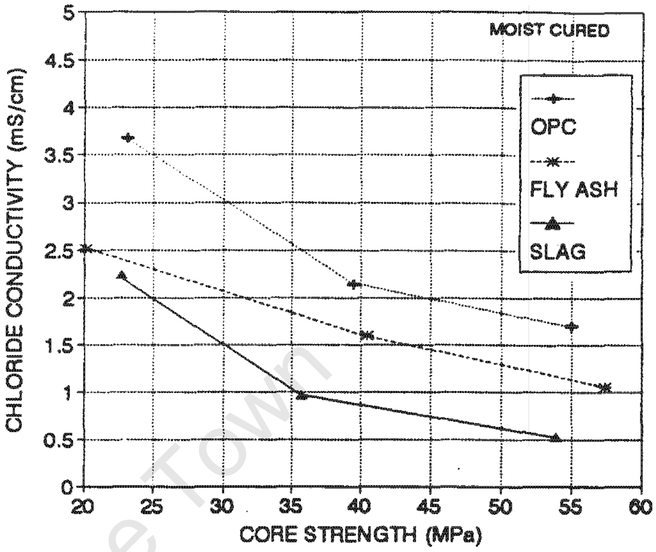


Figure 3.12d : Chloride Conductivity vs. Core Strength

e. Abrasion Resistance

Abrasion resistance (defined as concrete having a low coefficient of abrasion) was found to depend on the grade of concrete and the extent of curing and was less affected by the type of concrete. Dry cured concrete was considerably less resistant to abrasion than similar wet or moist cured concrete particularly for fly ash and slag concrete. Moist and wet cured OPC concrete had similar abrasion resistance but moist cured fly ash and slag concrete had lower abrasion resistance than wet cured concrete. The slower cement hydration associated with cement extenders such as fly ash and slag was probably responsible for the discrepancy. Details of the abrasion resistance coefficients measured for OPC, fly ash and slag concrete are shown in Figure 3.13a - 3.13c.

OPC concrete had higher abrasion resistance than either fly ash or slag concrete when wet, moist or dry cured. Slag concrete in particular had low abrasion resistance and when dry cured the concrete had poor abrasion resistance. A comparison of moist cured OPC, fly ash and slag concrete is shown in Figure 3.13d.

A strong relationship appears to exist between abrasion resistance and compressive strength. This relationship is consistent with the nature of the abrasion test which employs sand-blasting to physically abrade the material. An increase in strength from 20 to 60 MPa would generally reduce the abrasion resistance coefficient by 50% and in some cases by over 60%. Clearly a high strength material is required to ensure high abrasion resistance whilst adequate curing may also be of assistance. Grade 60 concretes showed little difference in abrasion resistance when comparing OPC, fly ash and slag concrete.

Similar trends noticed in the abrasion resistance results have been found by other researchers; Senbetta and Malchow showed that moist curing increased the abrasion resistance of concrete when compared to air curing [40], while Ho found that the abrasion resistance of OPC concrete increased with increasing grade and increasing active curing [41]. The abrasion resistance results found by Ho were generally higher than for Western Cape material with coefficients of 0.190 for a grade 40 fog cured concrete. The higher abrasion resistance could have been due to the effect of the Australian aggregates used in the investigation as the quality of the coarse aggregate has a major influence on the amount of abrasion [42].

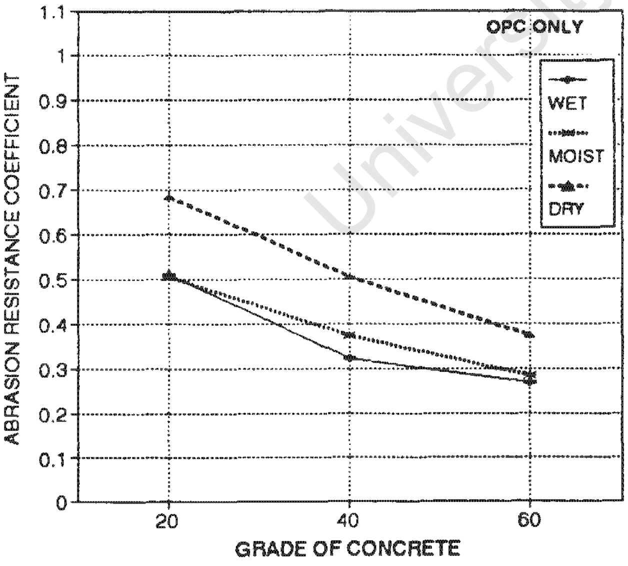


Figure 3.13a : Abrasion Resistance vs. Grade of OPC Concrete

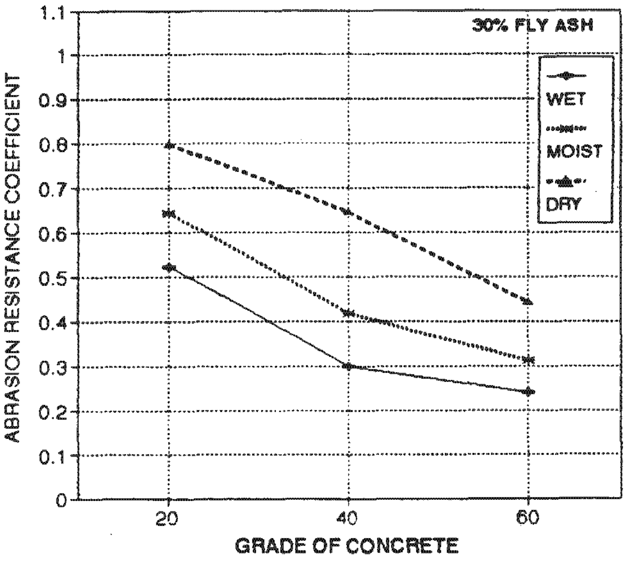


Figure 3.13b : Abrasion Resistance vs. Grade of Fly Ash Concrete

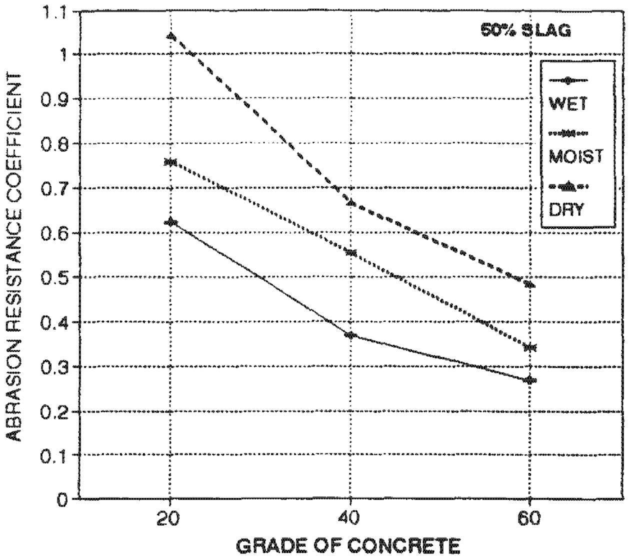


Figure 3.13c : Abrasion Resistance vs. Grade of Slag Concrete

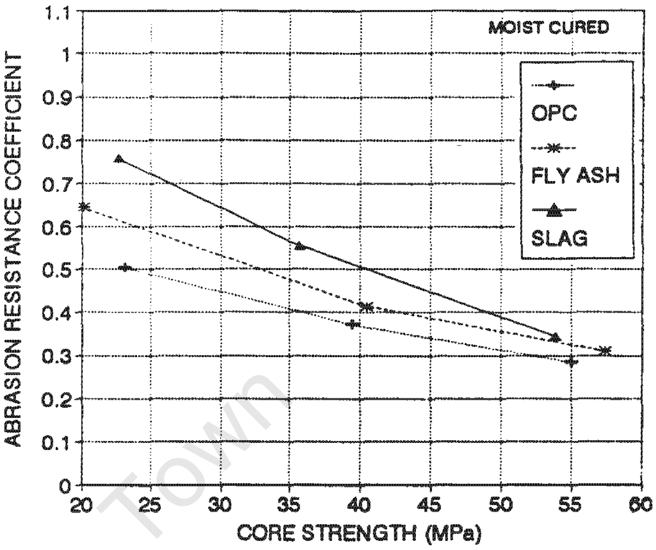


Figure 3.13d : Abrasion Resistance vs. Core Strength

f. Concrete Resistivity

Concrete resistivity results recorded after 28 days were dependent on the type of concrete and increased marginally with increasing strength while no consistent trend was observed with respect to different curing regimes. All concrete blocks were tested under fairly damp conditions as the moist and dry cured samples were left outdoors after seven days where they were exposed to occasional rain and evening dewfall. Details of the resistivity results for OPC, fly ash and slag concrete are shown in Figures 3.14a - 3.14c.

Resistivity values for the different concrete types fell within narrow bands with OPC concrete having values between 6 and 8 kOhmcm, fly ash concrete having values between 8 and 10 kOhmcm and slag concrete having values between 14 and 18 kOhmcm. The higher resistivity of slag concrete was probably due to the lower diffusivity of the material and the lower conductivity of the pore water solution. The resistivity test was however unable to distinguish between the two effects and other methods would be required to establish the individual effect of each factor. A comparison between moist cured OPC, fly ash and slag concrete is shown in Figure 3.14d.

Interpretation of resistivity results was difficult because resistivity depends upon the diffusivity of the material, conductivity of the pore water solution and moisture condition of the material. Separating each of these parameters is not possible with the in situ test so the results can only be used for comparative purposes. The test method is however a useful monitoring method particularly under stable moisture conditions such as saturated conditions where no preconditioning of the material is required before testing. Testing saturated concrete eliminates moisture content variations and is therefore able to quantify the pore structure more reliably.

Resistivity measurements should ideally be performed on saturated concrete before extensive exposure since readings are affected by moisture profiles, chloride contamination and carbonation [43,44]. The higher resistivity results of fly ash and slag concrete suggest that potential corrosion rates of reinforcement will be lower than those in OPC concrete, with less likelihood of corrosion damage. This may only be true for macrocell corrosion however since localized pitting corrosion may not be significantly affected by the resistivity of the surrounding concrete.

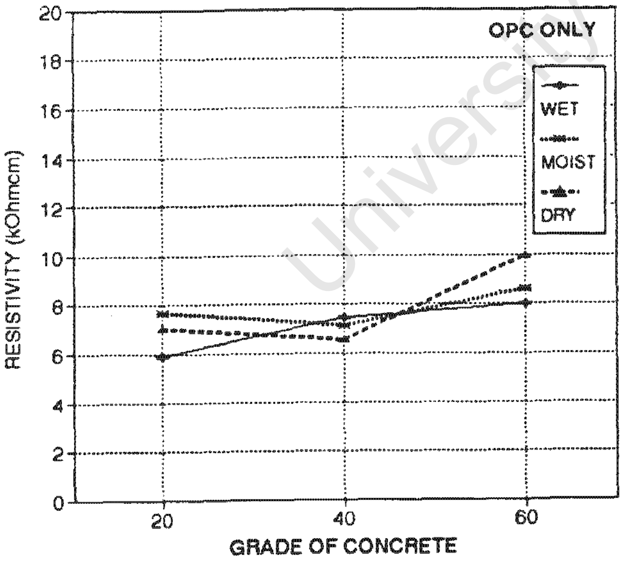


Figure 3.14a : Concrete Resistivity vs. Grade of OPC Concrete

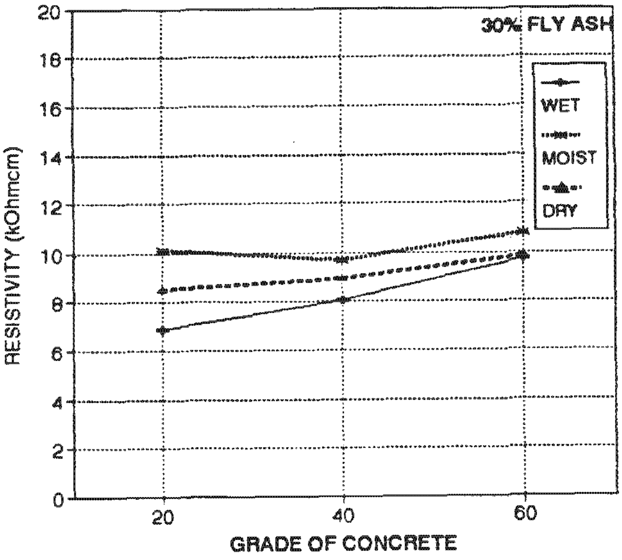
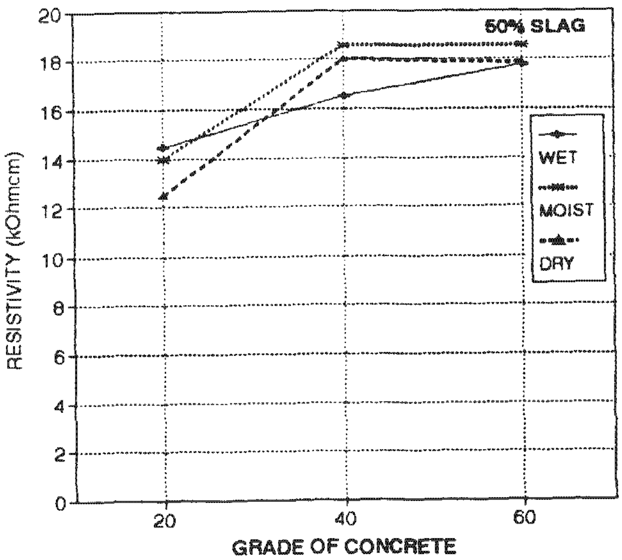


Figure 3.14b : Concrete Resistivity vs. Grade of Fly Ash Concrete



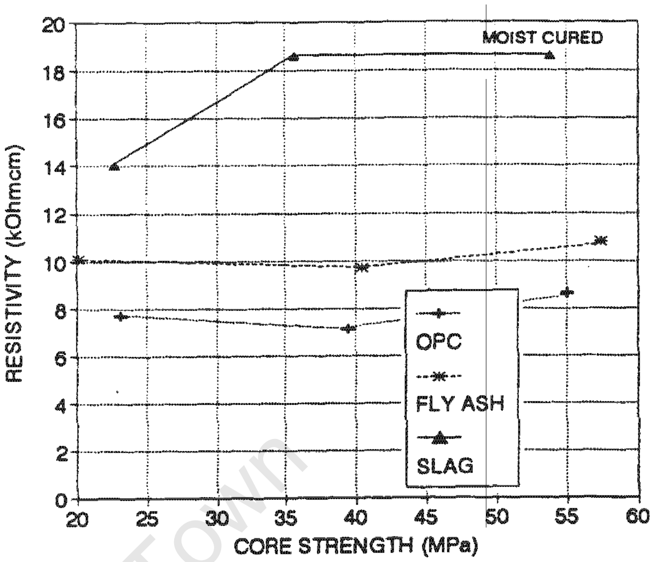


Figure 3.14c : Concrete Resistivity vs. Grade of Slag Concrete

Figure 3.14d : Concrete Resistivity vs. Core Strength

3.4.3 Summary of Concrete Characterization Results

Concrete characterization results appeared to follow similar trends with respect to grade of concrete, extent of curing and type of concrete. Increasing the grade of concrete resulted in better durability potential although the improvement was not as pronounced as expected and certainly does not justify the current emphasis on strength being used as a measure of concrete quality. The extent of active moist curing had a marked effect on potential durability and even on concrete strength particularly for fly ash and slag concrete. The improvement of concrete durability by using materials such as fly ash and slag can only be fully utilised when the concrete is well cured as the materials are sensitive to poor curing.

A marine durability index which combines the results of each durability index to produce an overall index has been proposed [45]. The marine durability index would allow different concretes to be compared but would have little theoretical value. Such an index might give a better assessment of the potential durability of marine concrete but the complex relationships of the deterioration mechanisms must be known to calculate a suitable marine durability index. This index would also be valid for certain exposure conditions only so the application might be of a limited nature. The use of durability index tests as indicators of potential durability will be discussed in more detail in Chapter Seven.

Comparisons between different durability index tests can show the consistency of test results but it should be noted that the tests measure different transport mechanisms which may produce varying responses from the concrete. The relationship between oxygen permeability index and water sorptivity for wet, moist and dry cured concrete is shown in Figure 3.15. A fairly consistent relationship was found between the two parameters with slag having lower sorptivity than either OPC or fly ash concrete for similar oxygen permeability index.

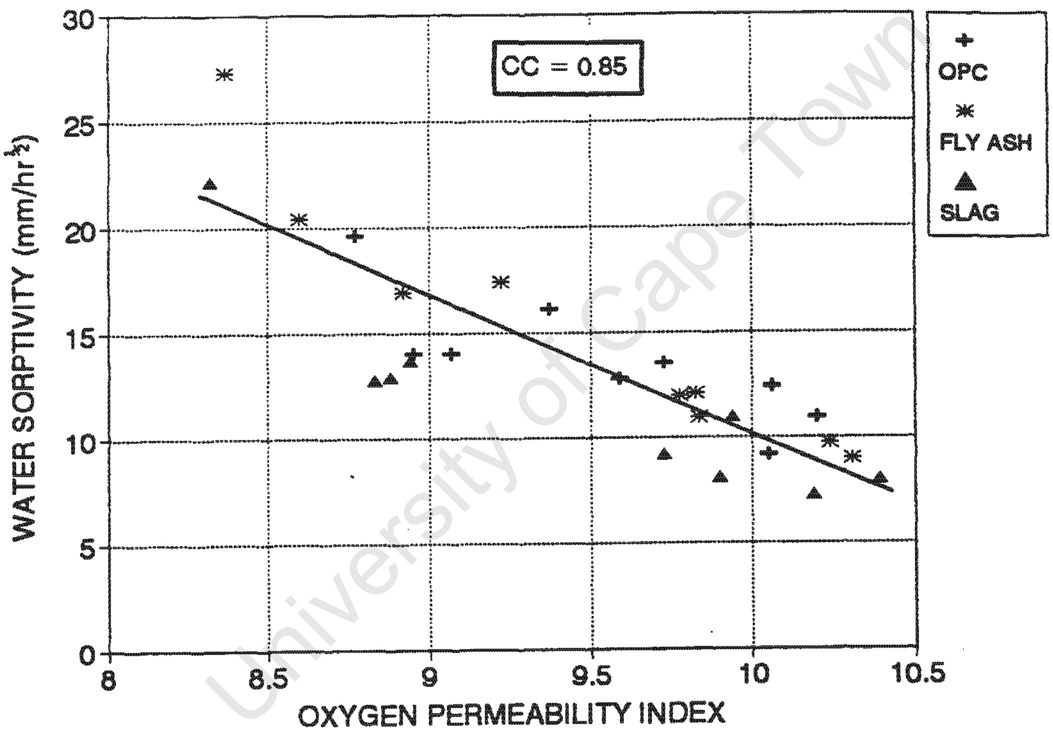


Figure 3.15 : Water Sorptivity versus Oxygen Permeability Index

Figure 3.16 shows the relationship between chloride conductivity and oxygen permeability index where a consistent trend between the two parameters was apparent. Fly ash and OPC concrete had similar chloride conductivity when compared to oxygen permeability index with the exception of a few outliers. Slag concrete had lower chloride conductivity with respect to oxygen permeability than either OPC or fly ash concrete except for two outliers.

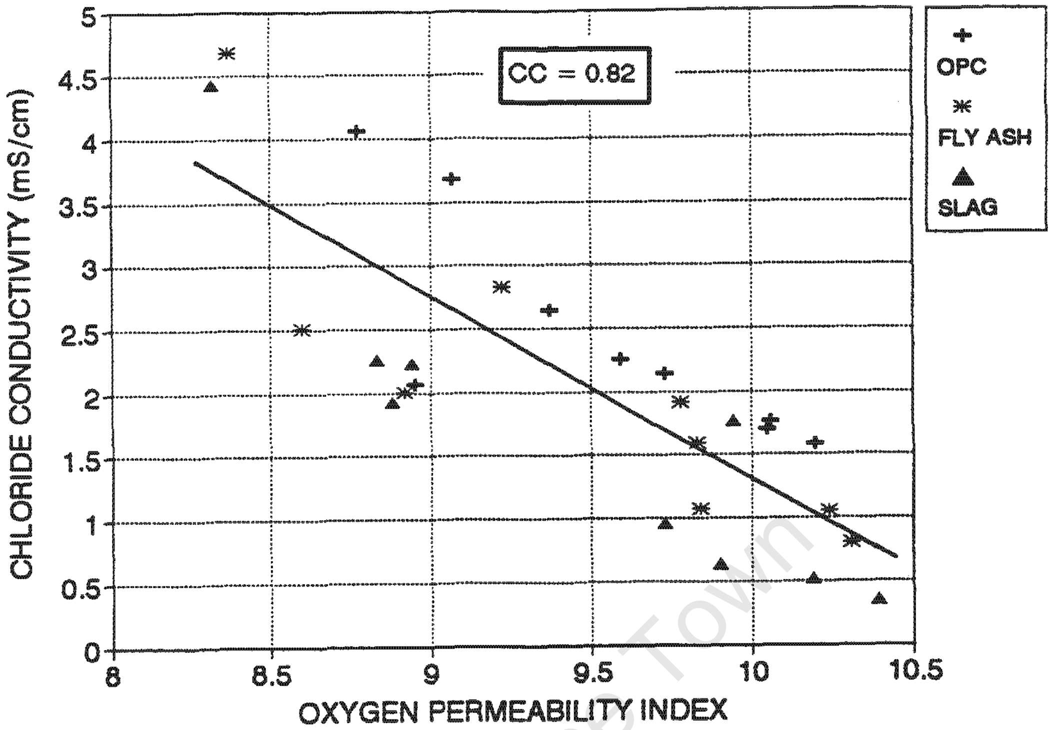


Figure 3.16 : Chloride Conductivity versus Oxygen Permeability Index

The best correlation between durability index tests was found with chloride conductivity and water sorptivity where all three concrete types followed a similar trend. Figure 3.17 shows the relationship between chloride conductivity and water sorptivity. The correlation was less consistent at lower strengths with the grade 20 concrete being fairly inconsistent. The overall linear regression correlation coefficient was 0.812 which indicates a fairly good correlation. The relationship between the two tests does not necessarily mean that only one test is required to measure both properties since each test has different sensitivities to effects such as cement type and extent of curing. The major transport mechanism appropriate to the circumstances being considered should therefore dictate which durability index test is appropriate. Comparisons between durability index test results are important in that correlations indicate some degree of consistency between the tests in measuring the potential durability of the material.

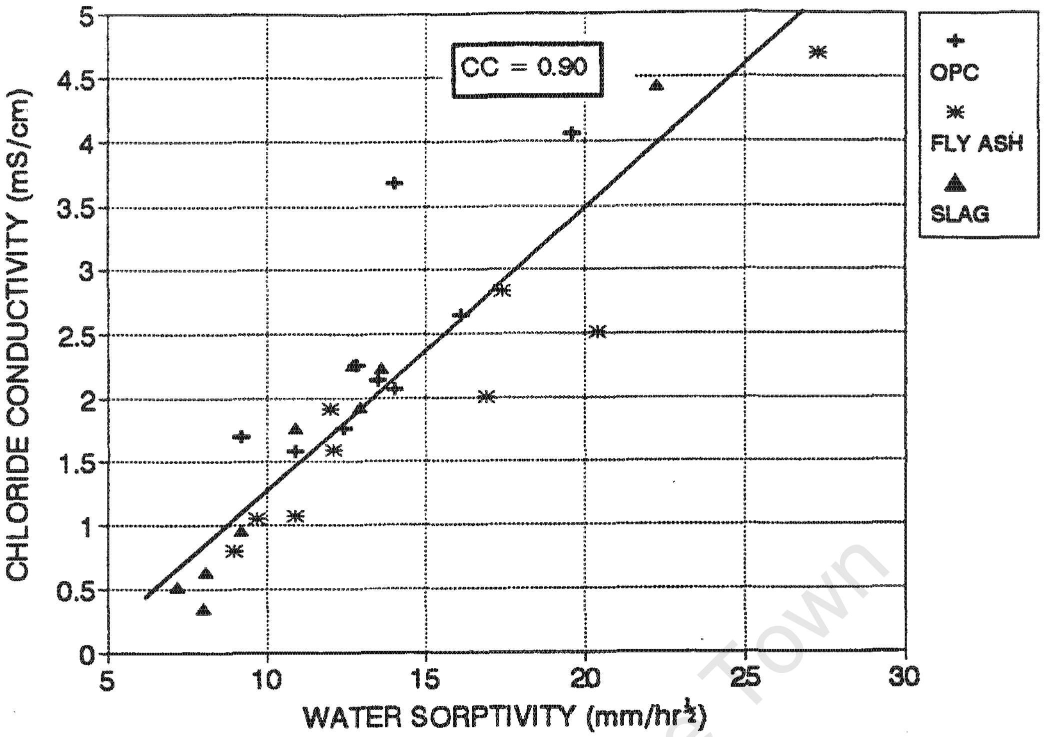


Figure 3.17 : Chloride Conductivity versus Water Sorptivity

A poor correlation was found between chloride conductivity and resistivity results as shown in Figure 3.18. A rough trend of increasing resistivity with decreasing chloride conductivity was apparent but the influence of concrete type on resistivity tended to group the results in clusters. Clearly resistivity testing is influenced by parameters such as moisture content and pore water conductivity which are not directly related to potential durability. These internal and external effects make the resistivity test unsuitable for use as an early age indicator of potential durability.

Water sorptivity and abrasion resistance are both dependent on the concrete surface quality and it was not surprising that a clear relationship was evident between the two properties. The water sorptivity and abrasion resistance results are plotted against one another in Figure 3.19. OPC and fly ash concrete exhibited a similar relationship while slag concrete had lower abrasion resistance for a given value of sorptivity. It would appear that slag concrete has inherently lower physical abrasion resistance than either OPC or fly ash concrete which could affect the durability performance under certain exposure conditions.

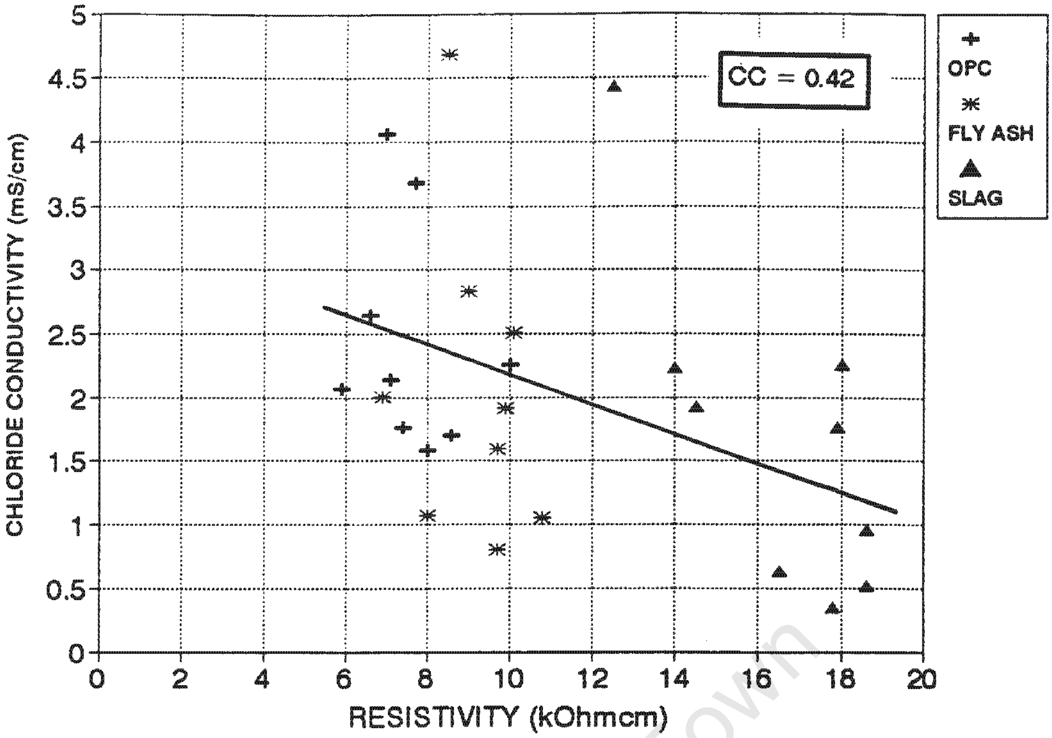


Figure 3.18 : Chloride Conductivity vs. Concrete Resistivity

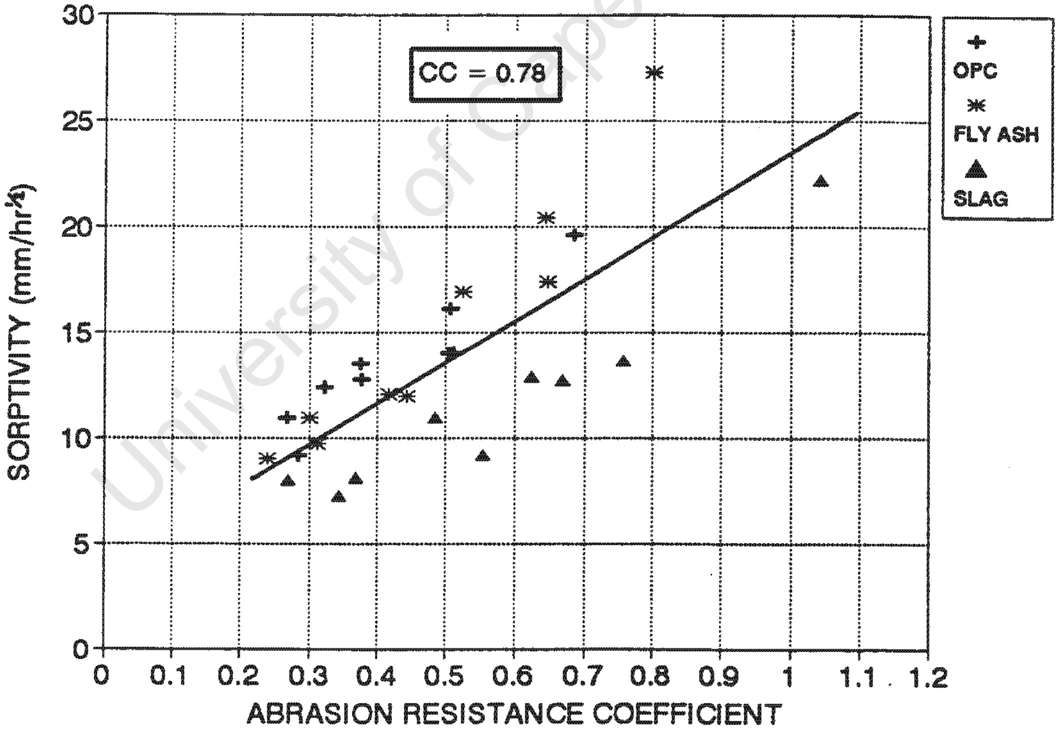


Figure 3.19 : Water Sorptivity vs. Abrasion Resistance Coefficient

If the comparative performance of the concrete as measured by the concrete characterization testing is analysed recommendations can be made about producing concrete with a good durability potential. Results from this work indicate that potential durability as measured with durability index tests decreases significantly below a grade of 40 MPa even if adequate curing is guaranteed. Concrete durability is enhanced by extended moist curing particularly with cement extenders such as fly ash and slag. Potential durability is reduced when concrete is poorly cured, particularly fly ash and slag concrete which were sensitive to curing.

Durability index tests such as oxygen permeability, sorptivity and chloride conductivity could be used as part of a system of performance specifications during construction to ensure good durability potential. The chloride conductivity should ideally be used at the design phase of a marine structure so that the selection of concrete materials can be optimized. Sorptivity testing has been shown to be sensitive to the effects of curing and could be used to measure the curing effectiveness during construction. Testing of concrete for oxygen permeability would provide useful information about the quality of the covercrete zone and is sensitive to major defects caused by inadequate compaction, segregation and cracking. The oxygen permeability test may also be used an an indicator of carbonation resistance of concrete as shown in Chapter Four.

3.5 Validating Concrete Characterization Results

Concrete characterization results need to be related to the actual deterioration processes which affect concrete so that the durability performance of concrete can be related to the original properties of the material. This is important since concrete properties change in the medium and long-term and concretes react in specific ways to deterioration processes. These different material responses will undoubtedly affect the durability performance of concretes. If the durability performance of concrete can be related back to the original properties minimum requirements for durability can be specified for different environments.

Three different approaches are proposed for determining the durability performance of concrete; short-term exposure testing of characterized concrete samples, durability performance of concrete structures in service and accelerated durability testing of concrete in the laboratory. The advantage of using exposure samples is that predictions can be made in the medium-term but the reliability of extrapolations used to predict durability performance is not certain. Measuring the durability performance of concrete structures in service provides information about the material performance but determining the original material properties is difficult. Accelerated durability testing of concrete provides information quickly but the accelerated testing may change the deterioration process or alter the material response to the process. A combination of all three approaches was investigated in this work in an attempt to estimate the durability performance of concrete in the marine environment. Exposure testing of characterized concrete samples is discussed in Chapter Four, measuring the durability performance of marine structures is investigated in Chapter Five and a number of accelerated durability test methods for marine concrete are reported in Chapter Six.

3.6 Conclusions

Durability index tests such as oxygen permeability, sorptivity and chloride conductivity have a sound theoretical basis and are shown to be sensitive to concrete microstructural changes which affect durability. The tests measure transport mechanisms such as absorption, diffusion and permeation using simple, rapid and reliable test techniques which can be used in site laboratories. Other concrete tests such as core strength, abrasion resistance and resistivity were found to be useful for assessing specific aspects related to the potential durability of concrete but are not suitable for use as durability index tests (although the abrasion resistance test shows promise).

The extensive series of tests documented in this chapter provided valuable information and allowed the sensitivity of the tests to be assessed. The appropriateness of the tests as indicators of potential durability was not determined during this preliminary work. The chloride conductivity test was thought to be the most likely indicator of potential chloride resistance since the test may be related to chloride ingress into concrete. While general trends were apparent between durability tests such as permeability, sorptivity and chloride conductivity, cognizance should be taken that different transport mechanisms were assessed resulting in unique relationships for each test.

Fly ash and slag concrete had better potential durability than OPC when properly cured but poor curing negated the benefits of using cement extenders. OPC concrete had better abrasion resistance than either fly ash or slag concrete but the differences were negligible at higher grades. All grade 20 concrete had poor durability potential and a grade 40 material was required to produce relatively good short-term characteristics. Similar trends were observed when the permeability, sorptivity and chloride conductivity results were compared which indicates that the tests are able to consistently quantify the pore structure characteristics of concrete which affect durability.

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3.8 APPENDICES

DURABILITY INDEX TEST	CURING TYPE	OPC ONLY			30% FLY ASH			50% SLAG		
		20 MPa	40 MPa	60 MPa	20 MPa	40 MPa	60 MPa	20 MPa	40 MPa	60 MPa
CUBE fcu (MPa)	WET	19.7	38.1	58.6	18.0	41.7	65.1	21.2	40.9	64.1
CORE STRENGTH (MPa)	WET	18.8	42.4	56.2	20.8	40.3	59.1	21.0	40.5	57.7
	MOIST	23.1	39.5	55.0	20.2	40.5	57.4	22.7	35.7	53.9
	DRY	19.7	41.0	55.7	18.8	35.1	48.7	18.2	34.4	49.1
OXYGEN PERMEABILITY INDEX	WET	8.95	10.06	10.20	8.92	9.84	10.31	8.88	9.90	10.39
	MOIST	9.07	9.73	10.05	8.60	9.83	10.24	8.94	9.73	10.19
	DRY	8.77	9.37	9.59	8.37	9.22	9.78	8.32	8.83	9.94
WATER SORPTIVITY (mm/hr$^{1/2}$)	WET	14.0	12.4	10.9	16.9	10.9	9.0	12.9	8.1	8.0
	MOIST	14.0	13.5	9.2	20.4	12.1	9.7	13.6	9.2	7.2
	DRY	19.6	16.1	12.8	27.3	17.4	12.0	22.2	12.7	10.9
CHLORIDE CONDUCTIVITY (mS/cm)	WET	2.07	1.76	1.58	2.01	1.07	0.81	1.93	0.63	0.35
	MOIST	3.68	2.14	1.70	2.51	1.59	1.05	2.23	0.96	0.52
	DRY	4.07	2.64	2.25	4.69	2.83	1.92	4.43	2.25	1.76
ABRASION RESISTANCE COEFF.	WET	0.511	0.322	0.267	0.524	0.300	0.239	0.623	0.368	0.268
	MOIST	0.504	0.374	0.284	0.644	0.416	0.312	0.758	0.554	0.344
	DRY	0.684	0.506	0.375	0.799	0.646	0.443	1.043	0.666	0.483
CONCRETE RESISTIVITY (kOhmcm)	WET	5.9	7.4	8.0	6.9	8.0	9.7	14.5	16.5	17.8
	MOIST	7.7	7.1	8.6	10.1	9.7	10.8	14.0	18.6	18.6
	DRY	7.0	6.6	10.0	8.5	9.0	9.9	12.5	18.0	17.9

Table 3A1 : Concrete Characterization Results at 28 Days

CONCRETE TYPE	CONCRETE GRADE	SAMPLE NUMBER	TYPE OF CURING		
			WET	MOIST	DRY
OPC CONCRETE	20 MPa	1	19.0	22.0	19.7
		2	18.7	24.1	19.6
		AVG	18.8	23.1	19.7
	40 MPa	1	43.4	38.3	43.7
		2	41.3	40.7	38.3
		AVG	42.4	39.5	41.0
	60 MPa	1	54.8	56.6	53.4
		2	57.6	53.4	57.9
		AVG	56.2	55.0	55.7
30 % FLY ASH CONCRETE	20 MPa	1	19.0	20.2	18.7
		2	22.6	20.2	19.0
		AVG	20.8	20.2	18.8
	40 MPa	1	41.5	39.7	33.5
		2	39.1	41.2	36.6
		AVG	40.3	40.5	35.1
	60 MPa	1	61.1	59.8	48.5
		2	57.1	55.0	48.9
		AVG	59.1	57.4	48.7
50 % SLAG CONCRETE	20 MPa	1	21.1	22.4	18.1
		2	20.8	23.0	18.4
		AVG	21.0	22.7	18.3
	40 MPa	1	43.1	36.5	36.5
		2	37.9	35.0	32.3
		AVG	40.5	35.7	34.4
	60 MPa	1	52.8	51.3	49.4
		2	62.7	56.6	48.8
		AVG	57.7	53.9	49.1

Table 3A2 : Core Strength Results at 28 Days (MPa)

CONCRETE TYPE	CONCRETE GRADE	SAMPLE NUMBER	TYPE OF CURING		
			WET	MOIST	DRY
OPC CONCRETE	20 MPa	1	8.99	9.08	8.78
		2	8.93	9.06	8.77
		AVG	8.95	9.07	8.77
	40 MPa	1	10.06	9.75	9.37
		2	10.07	9.72	9.37
		AVG	10.06	9.73	9.37
	60 MPa	1	10.11	10.18	9.53
		2	10.30	9.95	9.67
		AVG	10.20	10.05	9.59
30 % FLY ASH CONCRETE	20 MPa	1	8.83	8.94	8.27
		2	9.04	8.41	8.52
		AVG	8.92	8.60	8.37
	40 MPa	1	9.79	9.97	9.19
		2	9.89	9.72	9.25
		AVG	9.84	9.83	9.22
	60 MPa	1	10.29	10.36	9.80
		2	10.34	10.15	9.78
		AVG	10.31	10.24	9.78
50 % SLAG CONCRETE	20 MPa	1	9.05	9.02	8.36
		2	8.76	8.88	8.29
		AVG	8.88	8.94	8.32
	40 MPa	1	9.90	9.76	8.88
		2	9.90	9.70	8.78
		AVG	9.90	9.73	8.83
	60 MPa	1	10.55	10.16	9.85
		2	10.28	10.22	9.85
		AVG	10.39	10.19	9.85

Table 3A3 : Oxygen Permeability Index Results at 28 Days

CONCRETE TYPE	CONCRETE GRADE	SAMPLE NUMBER	TYPE OF CURING		
			WET	MOIST	DRY
OPC CONCRETE	20 MPa	1	12.8	15.0	24.9
		2	15.2	13.0	14.4
		AVG	14.0	14.0	19.6
	40 MPa	1	12.2	13.7	16.5
		2	12.6	13.2	15.7
		AVG	12.4	13.5	16.1
	60 MPa	1	10.6	9.4	13.4
		2	11.1	9.0	12.3
		AVG	10.9	9.2	12.8
30 % FLY ASH CONCRETE	20 MPa	1	19.3	23.9	21.8
		2	14.5	16.8	32.8
		AVG	16.9	20.4	27.3
	40 MPa	1	11.6	11.6	18.0
		2	10.1	12.6	16.7
		AVG	10.9	12.1	17.4
	60 MPa	1	8.7	9.6	11.8
		2	9.3	9.8	12.2
		AVG	9.0	9.7	12.0
50 % SLAG CONCRETE	20 MPa	1	11.7	14.0	21.2
		2	14.2	13.3	23.2
		AVG	12.9	13.6	22.2
	40 MPa	1	8.6	9.1	13.4
		2	7.7	9.2	12.1
		AVG	8.1	9.2	12.7
	60 MPa	1	7.1	7.0	11.1
		2	8.9	7.4	10.6
		AVG	8.0	7.2	10.9

Table 3A4 : Water Sorptivity Results at 28 Days (mm/√hr)

CONCRETE TYPE	CONCRETE GRADE	SAMPLE NUMBER	TYPE OF CURING		
			WET	MOIST	DRY
OPC CONCRETE	20 MPa	1	1.98	3.81	4.05
		2	2.17	3.54	4.09
		AVG	2.07	3.68	4.07
	40 MPa	1	1.77	2.48	2.60
		2	1.75	1.80	2.67
		AVG	1.76	2.14	2.64
	60 MPa	1	1.60	1.68	1.89
		2	1.56	1.71	2.62
		AVG	1.58	1.70	2.25
30 % FLY ASH CONCRETE	20 MPa	1	1.84	2.49	3.88
		2	2.17	2.54	5.50
		AVG	2.01	2.51	4.69
	40 MPa	1	0.96	1.37	2.45
		2	1.18	1.81	3.20
		AVG	1.07	1.59	2.83
	60 MPa	1	0.90	1.02	2.04
		2	0.73	1.09	1.80
		AVG	0.81	1.05	1.92
50 % SLAG CONCRETE	20 MPa	1	1.84	2.09	4.06
		2	2.01	2.37	4.80
		AVG	1.93	2.23	4.43
	40 MPa	1	0.57	1.01	2.15
		2	0.70	0.91	2.34
		AVG	0.63	0.96	2.25
	60 MPa	1	0.36	0.57	1.72
		2	0.35	0.47	1.81
		AVG	0.35	0.52	1.76

Table 3A5 : Chloride Conductivity Results at 28 Days (mS/cm)

CONCRETE TYPE	CONCRETE GRADE	SAMPLE NUMBER	TYPE OF CURING		
			WET	MOIST	DRY
OPC CONCRETE	20 MPa	1	0.589	0.452	0.723
		2	0.472	0.711	1.020
		3	0.375	0.368	0.496
		4	0.608	0.485	0.496
		AVG	0.511	0.504	0.684
	40 MPa	1	0.380	0.387	0.551
		2	0.322	0.388	0.448
		3	0.328	0.348	0.500
		4	0.258	0.374	0.525
		AVG	0.322	0.374	0.506
	60 MPa	1	0.318	0.278	0.355
		2	0.299	0.284	0.494
		3	0.235	0.315	0.275
		4	0.216	0.259	0.375
		AVG	0.267	0.284	0.375
30 % FLY ASH CONCRETE	20 MPa	1	0.660	0.889	0.697
		2	0.575	0.644	0.633
		3	0.395	0.470	0.710
		4	0.466	0.573	1.157
		AVG	0.524	0.644	0.799
	40 MPa	1	0.273	0.461	0.812
		2	0.351	0.315	0.461
		3	0.287	0.467	0.596
		4	0.287	0.419	0.716
		AVG	0.300	0.416	0.646
	60 MPa	1	0.284	0.325	0.355
		2	0.206	0.269	0.475
		3	0.239	0.307	0.507
		4	0.227	0.345	0.435
		AVG	0.239	0.312	0.443
50 % SLAG CONCRETE	20 MPa	1	0.753	0.933	1.498
		2	0.569	0.798	0.886
		3	0.617	0.628	0.866
		4	0.552	0.674	0.922
		AVG	0.623	0.758	1.043
	40 MPa	1	0.406	0.573	0.698
		2	0.335	0.522	0.615
		3	0.355	0.554	0.762
		4	0.374	0.567	0.589
		AVG	0.368	0.554	0.666
	60 MPa	1	0.306	0.335	0.468
		2	0.268	0.375	0.483
		3	0.241	0.368	0.491
		4	0.255	0.298	0.489
		AVG	0.268	0.344	0.483

Table 3A6 : Abrasion Resistance Coefficients at 28 Days

CONCRETE TYPE	CONCRETE GRADE	SAMPLE NUMBER	TYPE OF CURING		
			WET	MOIST	DRY
OPC CONCRETE	20 MPa	1	6.4	7.3	7.0
		2	5.6	8.1	7.0
		3	5.8	7.6	7.1
		AVG	5.9	7.7	7.0
	40 MPa	1	7.5	6.9	8.2
		2	7.1	7.5	6.0
		3	7.7	6.8	5.6
		AVG	7.4	7.1	6.6
	60 MPa	1	8.5	9.8	9.2
		2	7.3	7.5	10.9
		3	8.3	8.5	9.8
		AVG	8.0	8.6	10.0
30 % FLY ASH CONCRETE	20 MPa	1	7.0	10.5	8.7
		2	6.5	8.8	7.7
		3	7.1	10.9	9.0
		AVG	6.9	10.1	8.5
	40 MPa	1	8.6	8.8	8.4
		2	7.3	9.5	9.6
		3	8.1	10.7	8.9
		AVG	8.0	9.7	9.0
	60 MPa	1	10.5	12.2	8.4
		2	10.2	10.5	10.3
		3	8.5	9.8	10.9
		AVG	9.7	10.8	9.9
50 % SLAG CONCRETE	20 MPa	1	16.4	12.7	10.7
		2	14.6	14.5	12.8
		3	12.4	14.8	13.9
		AVG	14.5	14.0	12.5
	40 MPa	1	15.8	18.3	18.3
		2	16.5	17.7	18.8
		3	17.1	19.7	17.0
		AVG	16.5	18.6	18.0
	60 MPa	1	19.4	17.6	18.1
		2	17.3	19.8	17.3
		3	16.8	18.3	18.3
		AVG	17.8	18.6	17.9

Table 3A7 : Concrete Resistivity at 28 Days (kOhmcm)

CHAPTER FOUR

MARINE EXPOSURE TESTING OF CONCRETE

4.1 Literature Survey

The complex chemical and physical interactions between seawater and the constituents of concrete make laboratory simulations of marine exposure extremely difficult. There is therefore a need for marine exposure studies where the effects of the marine environment on concrete can be monitored under well defined exposure conditions. Field studies of marine concrete allow deterioration mechanisms to be accurately assessed under normal exposure conditions which is not possible with accelerated laboratory tests. The disadvantage of field exposure of concrete is that the deterioration may take many years to proceed to a measurable extent. Results from field investigations are essential however for determining the long-term performance of concrete and validating findings from early age durability tests.

Simulated marine exposure has been attempted in many laboratories where the multivariate effect of exposure is reduced by considering one variable in isolation. With the widespread interest in chloride induced corrosion this has generally consisted of subjecting concrete specimens to chloride solutions either from external or internal sources [1,2,3]. Research done by Buenfeld and Newman and by Conjeaud have shown that the interaction of other sea water salts with concrete should not be ignored and chloride solutions cannot simulate marine exposure accurately [4,5]. Exposure of concrete to chloride solutions may however be useful for comparative purposes or when simulating the effect of deicing salts on road bridges.

The assumption that marine exposure of concrete involves mainly the chemical reaction of sea water with concrete appears to be commonly accepted. Several studies have been done exposing concrete to sea water under controlled conditions in tanks [6,7]. This assumes that effects such as wave action, abrasion, tidal action and surface drying are insignificant and do not accelerate the deterioration process. Controlled seawater exposure may provide useful comparative information about concrete but the conditions may be fairly mild in comparison to real exposure conditions on site. Gjorv et al used sea water tanks at Trondheim harbour and exposed concrete samples in sea water for periods up to three years whilst measuring chloride penetration into the concrete [6]. Predictions of service life were made using this data which assumes that the test conditions represent the worst exposure environment for marine structures. Results published by Sandberg and Tang confirm this assumption for concrete in

Sweden where submerged concrete was found to have higher chloride contents than either tidal or spray zone concrete [8]. This effect appears to occur in mild environments where the concrete is sheltered from wave action but conflicting evidence has been found in other environments [9]. Figures 4.1a and 4.1b show the chloride profiles obtained from concrete exposed to the spray, tidal and submerged zones in Sweden and Singapore respectively.

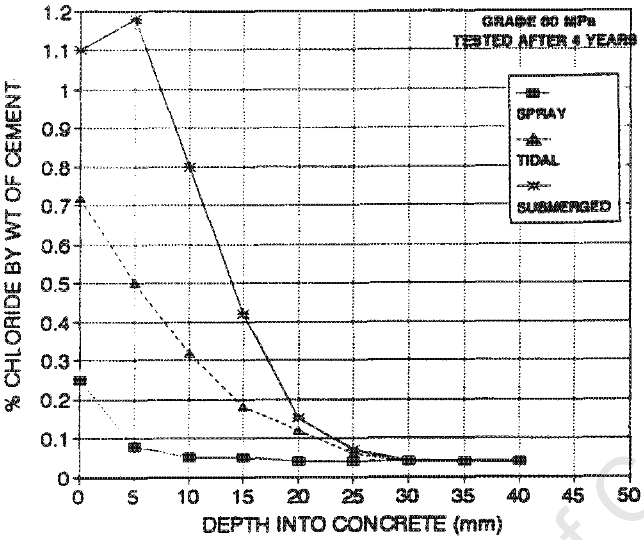


Figure 4.1a : Chloride Profiles Recorded in Sweden [8]

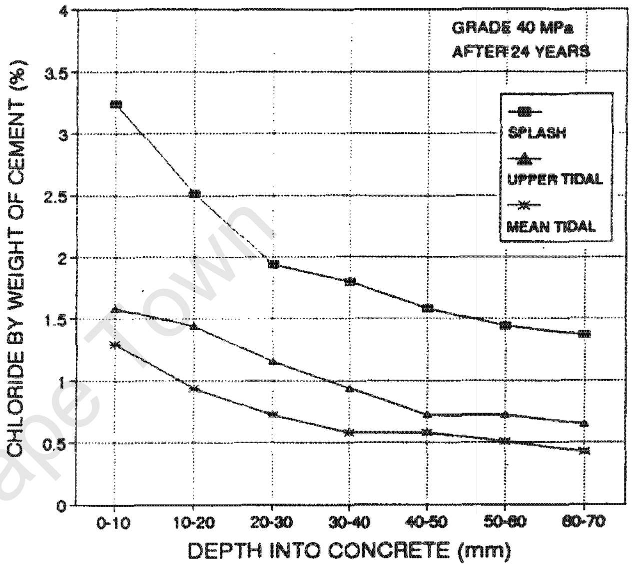


Figure 4.1b : Chloride Profiles Recorded in Singapore [9]

Buenfeld and Newman stated that surface blocking of concrete may occur by the deposition of insoluble salts such as brucite and aragonite which interfere with the ingress of chlorides [10]. This surface barrier was found to be relatively thin and may be broken down by attrition from wave action, thermal effects or cracking. Concrete that is not sheltered from physical agencies causing surface damage may therefore lose this protection resulting in easier access for chlorides compared to protected concrete. Concrete exposed to rough seas or hot climates have been found to have more rapid ingress of chlorides in the tidal zone than lower down in the submerged zone. This observation may be ascribed to breakdown of the protective surface layer [9].

Several marine exposure stations for concrete durability studies are protected in estuaries and make use of cages and barriers which reduce the severity of marine exposure. The BRE marine exposure site at Shoeburyness on the Thames Estuary uses a large holding tank for tidal zone samples that effectively eliminates severe drying and wave action [11]. The Dundee marine exposure site on the Tay Estuary is also sheltered with small concrete specimens being stacked in steel cages suspended in the sea [12]. These conditions do not represent the extreme conditions found on site in many instances, and the use of small specimens also limits macro effects which occur on site concrete such as cracking from thermal and shrinkage effects. Little appreciation for differences in marine exposure conditions is shown in the literature particularly with regard to wave action, sea temperature and surface drying.

Malhotra et al used fairly large concrete specimens for marine exposure testing in the tidal zone of the Bay of Fundy in order to simulate realistic marine conditions [13]. The specimens were sufficiently large to be free standing and did not require protective devices which might interfere with processes such as abrasion and freeze thaw action. Samples could be extracted by in situ coring and drilling to assess the durability performance. Steel reinforcement could also be embedded in the concrete so that corrosion monitoring could be undertaken thus making each sample a small structure which could be studied in isolation. This approach is expensive but should ensure conditions similar to those experienced by marine structures in service

Exposure conditions in the marine spray zone vary considerably depending on factors such as wave action, wind speed, distance from the sea, humidity and rainfall as shown in Figure 4.2. Local effects such as orientation and configuration of structures in the spray zone also have an influence on the severity of exposure. The interaction of these factors makes estimates of spray zone exposure difficult and little guidance is given in relevant codes of practice. A distinction is generally made between direct seawater exposure of concrete and exposure to sea spray but no further classification is given for designers of marine structures.

The effect of the marine spray zone has been investigated by several researchers, since the vast majority of marine structures are not in direct contact with sea water. Jaegermann reported results of concrete on the Mediterranean coast near Haifa [14], Mustafa and Yusof exposed concrete specimens on the West coast of Malaysia [15], Sakai and Sasaki investigated precracked concrete on the coast at Hokkaido Island [16] and work has been done in South Africa in Durban harbour [17]. Comparisons between exposure sites are difficult due to climatic differences resulting in varying wind-borne aerosol levels, humidity and temperature. Comparisons between exposure sites are subjective since there does not appear to be a standard method for measuring the level of wind-borne salt spray.

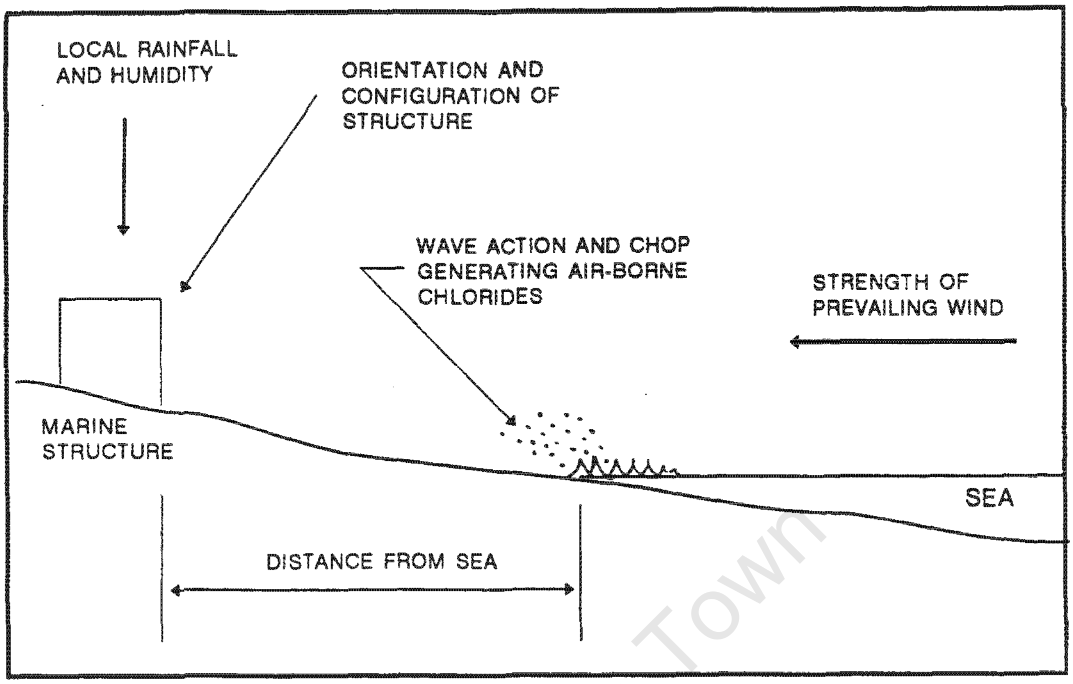


Figure 4.2: Marine Spray Zone Exposure - Factors Affecting Severity

Early field exposure investigations of marine concrete samples generally concentrated on physical phenomena such as strength, freeze-thaw resistance and dimensional stability. More recent studies have investigated more relevant durability-related properties of concrete such as chloride ingress and reinforcement corrosion. The increased use of cement extenders and additives has resulted in many comparative durability investigations under marine conditions. In situ tests such as resistivity have been used in some of these projects but the primary indicator of durability performance of marine concrete has been assumed to be the rate of chloride penetration which is usually expressed in terms of the diffusion coefficient.

The diffusion of chloride ions through concrete under non-steady state conditions may be described by Fick's second law of diffusion as follows.

$$\frac{dC}{dt}=D_c\frac{d^2C}{dx^2}\dots\dots\dots\dots\dots\dots\dots(4.1)$$

The partial differential equation may be solved by applying the following boundary conditions and assumptions [18].

1) The surface concentration (Cs) reaches a constant value almost immediately after exposure to the diffusant.
2) The internal concentration (Cx) is zero at some point internally in the concrete.
3) The diffusivity of the material remains constant with depth and time of exposure.
4) The concrete pore structure is saturated throughout the diffusion process.
5) Chloride diffusion through concrete is a two component system with no significant interactions between the components.

The solution of Fick's second law of diffusion may therefore be written as follows.

$$C_x=C_s\left(1-\mathrm{Erf}\left[\frac{x}{2\sqrt{D_c t}}\right]\right)\ldots\ldots\ldots\ldots\ldots\ldots(4.2)$$

where C_x is the internal concentration (%), C_s is the surface concentration (%), x is the depth in the concrete (cm), D_c is the diffusion coefficient (cm^2/s), Erf is the mathematical error function (see Appendix Table 4A13) and t is the time of diffusion in seconds.

Table 4.1: Suitability of Boundary Conditions For Solution of Fick's Law

BOUNDARY CONDITION	IDEALIZED STATE (ASSUMED)	OBSERVED BEHAVIOUR
1	Surface concentration reaches a constant value at the start of exposure	Surface concentration may only stabilize after several years of exposure
2	Concentration is zero at some point internally in the concrete	Slender structures may not have zero concentrations with time and initial levels often exist
3	Diffusivity of concrete is constant with depth and time of exposure	Diffusivity of concrete decreases with depth and time of exposure
4	The concrete pore structure is saturated throughout the diffusion process	The concrete pore structure may only be partially saturated near the surface
5	No interaction occurs between concrete constituents and the diffusant	Significant physical and chemical interactions occurs between concrete and diffusant

The error function solution of Fick's Law has been popular for making predictions of service life by estimating the chloride content at a certain depth and time. Recently considerable doubt has been cast about the theoretical soundness of the approach [19,20]. Predictions of long-term chloride levels have been shown to have poor correlations with recorded data on site [8]. The reasons for the lack of confidence in the method are mainly due to the lack of compliance of observed diffusion characteristics with the stated boundary conditions implicit in the solution of Fick's Law (Table 4.1). Nevertheless the method still remains popular and diffusion coefficients of concrete are regularly quoted by researchers as measures of the resistance of concrete to chlorides. The solution of Fick's law is generally valid at any particular instant in time and may be useful provided the integrated nature of the diffusion coefficient is understood.

Bamforth stated in a review of chloride profiles from other sources that "the surface level is established within weeks of exposure, and changes very little thereafter" [21]. Surface concentrations may however fluctuate with seasonal variations and there is some evidence to suggest that surface concentrations actually increase with time [20,22]. The diffusivity of concrete has been shown to change with increasing depth and with time due to microstructural variations with depth and chemical reactions which alter the concrete pore structure [19,23,24,25]. The diffusivity of concrete is affected by the continued hydration of concrete, chloride binding of calcium aluminate phases and pore blocking due to deposition of insoluble salts at the concrete surface. Chatterji suggests that self-dessication of concrete results in internal concrete being unsaturated even for submerged concrete. Such partially saturated conditions will therefore affect the diffusion of chlorides [19]. Concrete in the tidal and spray zones is sometimes only partially saturated near the surface and chloride penetration occurs by a combination of water absorption and diffusion. Chatterji also maintains that ionic diffusion may not be described by Fick's Law alone as this applies to non-ionic materials [26]. For ionic diffusants Fick's Law must be modified, according to Chatterji, to take account of the increase of diffusivity of ions with dilution as expressed in the Nernst equation.

The assumptions made in the solution of Fick's Law are not valid for site concrete in the marine environment and even controlled diffusion tests in the laboratory do not fully comply with the boundary conditions. Clearly, applying Fick's Law of diffusion for chloride penetration into concrete has several drawbacks which demands some justification for continuing with the approach. Whilst the theoretical basis for applying the method to chloride diffusion has been shown to be not strictly valid the method does correlate well with measured data. Observed chloride profiles in site concrete may be fitted well with the error function solution to Fick's Law and a coefficient is produced which describes the shape of the curve. The diffusion coefficient has been shown to be an accurate indicator of the material resistance to chloride pentration. Useful predictions of service life have been formulated by Mangat and Molloy using Fick's Law modified to allow for a changing diffusion coefficient with time [25].

Chloride penetration into concrete has been shown to be dependent on concrete type and grade while cracking may allow rapid ingress of chlorides into the internal concrete [27,28,29]. Cement extenders such as fly ash and slag were generally found to have considerably greater resistance to chloride ingress than OPC concrete. Marine exposure results of concrete at Folkestone reported by Bamforth showed that diffusion coefficients of fly ash and slag concrete decreased steadily during the six year period [27]. Diffusion coefficients of OPC and silica fume concrete showed negligible decrease with time and were substantially higher than fly ash and slag concrete after six years exposure. These long-term trends have major implications on the durability performance of these concrete types. The surface concentration of fly ash and slag concrete was substantially higher than OPC and silica fume concrete which was ascribed to the higher volume of binder and superior chloride binding characteristics of the fly ash and slag concrete. Results from Bamforth's work for diffusion coefficient and surface concentration with time are shown in Figure 4.3a and 4.3b.

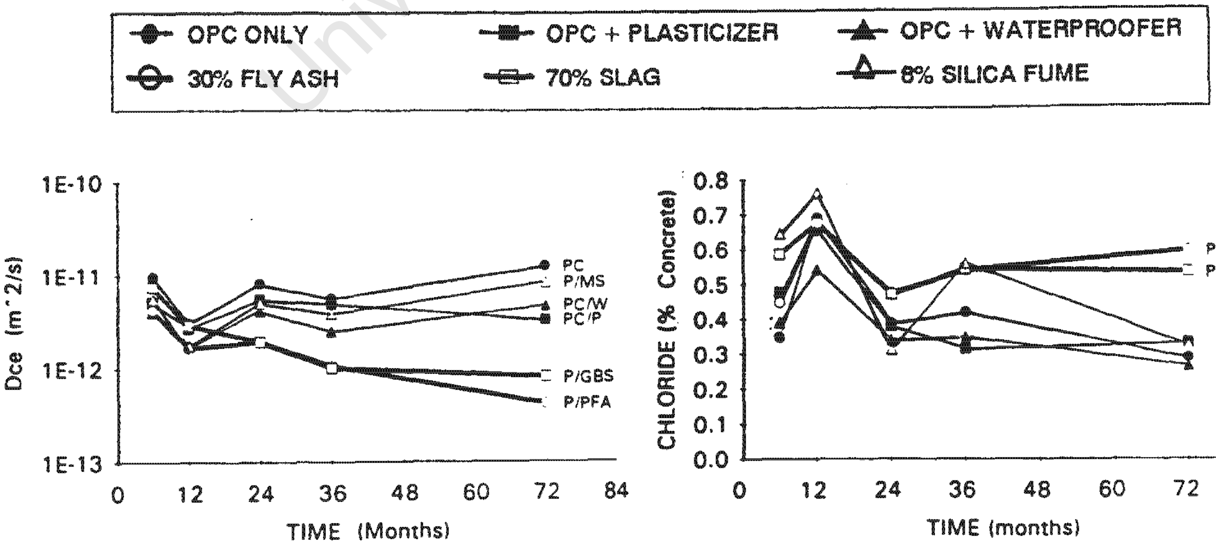


Figure 4.3a : Diffusion Coefficient, D_c versus Time [27]

Figure 4.3b : Surface Concentration, C_s versus Time [27]

Formulating a theoretical chloride diffusion model is complex and a multicomponent interactive model is required which allows for factors such as partially saturated conditions, variable surface concentrations, internal chemical reactions and macrodefects. Several theoretical models have been developed but at present this method is not sufficiently refined for practical applications [30,31]. The models usually require detailed information regarding material properties, constructional effects and environmental and loading conditions which can only be obtained using sophisticated monitoring equipment. It would therefore appear that an empirical approach of predicting long-term chloride contents is the most practical method until more sophisticated theoretical models have been developed [32]. It must be accepted that this is essentially a pragmatic approach where the chloride penetration is defined and predicted using the diffusion coefficient defined in equation 4.2.

With all the data from field exposure sites it should be possible to make useful comparisons of durability performance and predictions of concrete durability. Unfortunately this is complicated by different materials, casting procedures, specimen sizes, curing techniques, severity of exposure and testing of durability performance of concrete. This variability could be reduced if standard methods of characterizing marine concrete, assessing the severity of marine exposure and measuring durability performance of concrete could be established and used by all researchers. At present field studies of marine concrete performance are only useful locally, resulting in increasing numbers of exposure stations conducting essentially similar research. A coordinated research approach would clearly be more beneficial for all parties.

Reviewing the results of several investigations into the durability performance of marine concrete indicates that some factors consistently produce durable concrete with good chloride resistance. Important factors include the use of cement extenders such as fly ash and slag, low water/cement ratio, effective moist curing, good compaction, and prevention of cracking. What is still required is a better understanding of the mechanisms of deterioration of marine concrete structures and determination of the kinetics of the reactions. This information is needed to predict the durability of marine concrete structures and to propose minimum specifications for adequate durability performance.

4.2 Marine Exposure Sites

Due to the variability of marine exposure it was decided to use two marine exposure sites together with a control site at the University of Cape Town. The marine sites represented extremes of marine exposure being either sheltered (Simonstown) or exposed (Granger Bay).

4.2.1 Simonstown

The site at Simonstown was chosen because of its protected location being exposed to mild wave action and sheltered from the prevailing South Easterly wind. The sheltered nature of the site is partly due to the Naval Dockyard on the seaward side of the bay and the whole area is protected from the open ocean within False Bay. Sea temperatures ranged between 13 and 20°C due to warm waters which are trapped in the shallow bay. Tidal zone specimens were located just below the high tide mark while spray zone samples were located 3m above mean sea level facing North West. Figure 4.4a shows the general layout of Simonstown harbour showing the protected nature of the site while Figure 4.4b shows the location of tidal and spray zone samples on site.

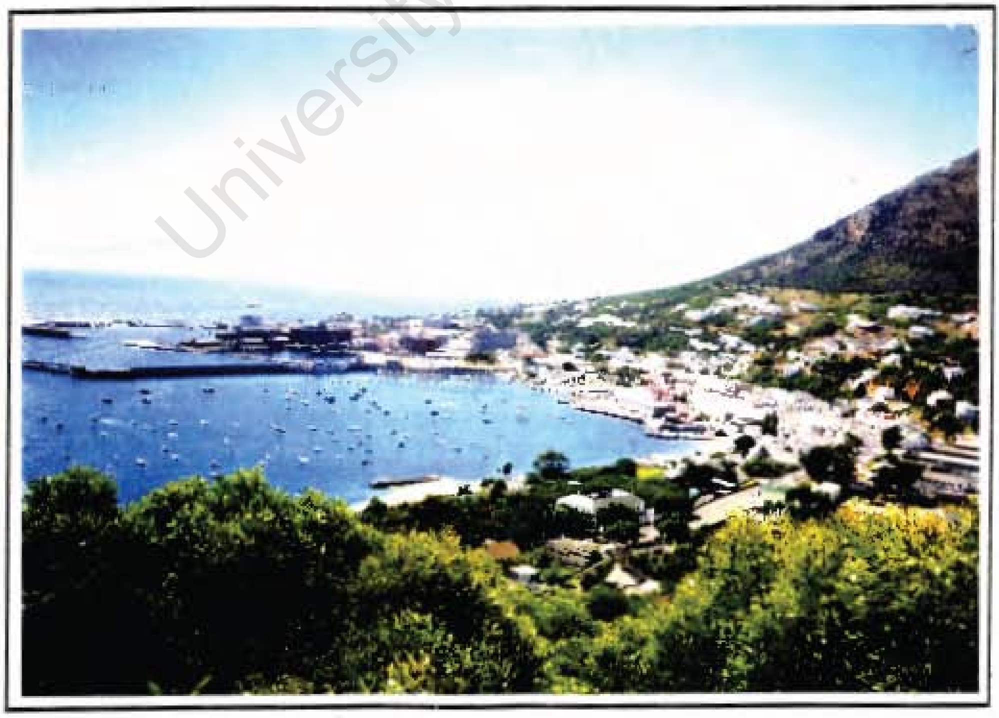


Figure 4.4a : View of Simonstown Harbour

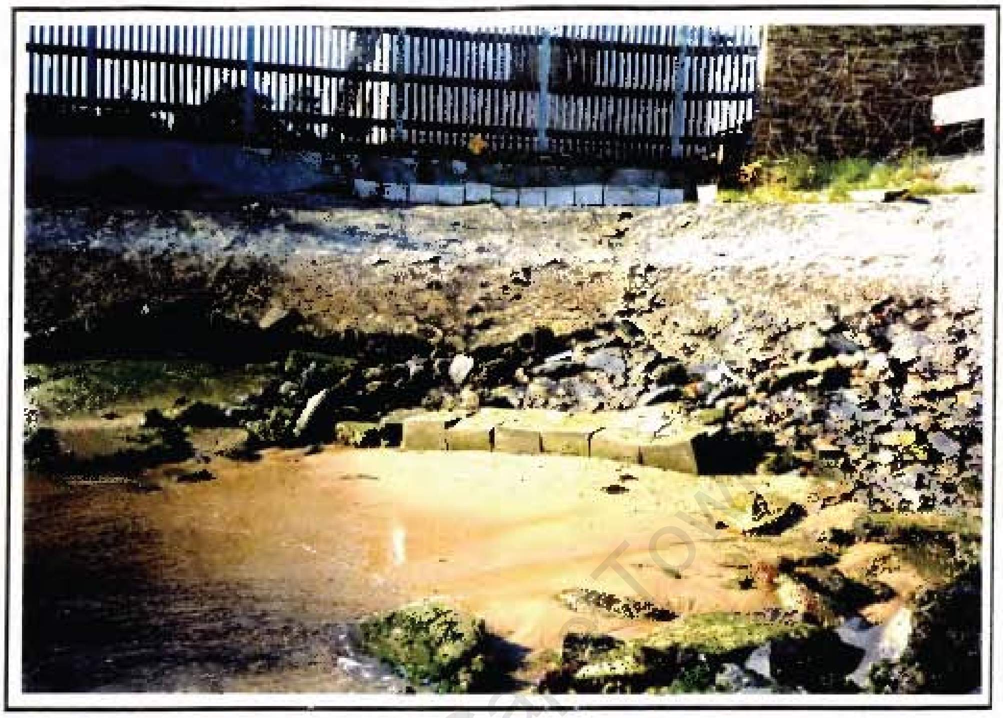


Figure 4.4b: View of Simonstown Tidal and Spray Zone Samples

4.2.2 Granger Bay

The exposure site at Granger Bay was chosen due to the exposed nature of the area which is prone to heavy wave action and strong winds. The site is located along the edge of Table Bay, outside the breakwater and exposed to the open Atlantic Ocean. Sea temperatures in Table Bay ranged from 12 to 15°C due to the presence of the cold Benguela current off the Cape West coast and the upwelling of cold waters near the shoreline. Tidal zone samples were located just below the high tide mark while the spray zone samples were located on top of a sea wall 3m above mean sea level facing in a North Westerly direction. All concrete blocks were chained down to prevent them being washed away during winter storms when waves often crash over the top of the sea wall. During summer wave action is more moderate with little overtopping of the sea wall by waves except in exceptional cases. Figure 4.5a shows conditions at Granger Bay during a winter storm while Figure 4.5b shows the location of the spray zone samples on top of the sea wall (the tidal zone samples were located at the base of the sea wall).

Figure 4.5a: View of Granger Bay During Winter

Figure 4.5b: Location of Spray Zone Samples at Granger Bay

4.2.3 Laboratory Controls

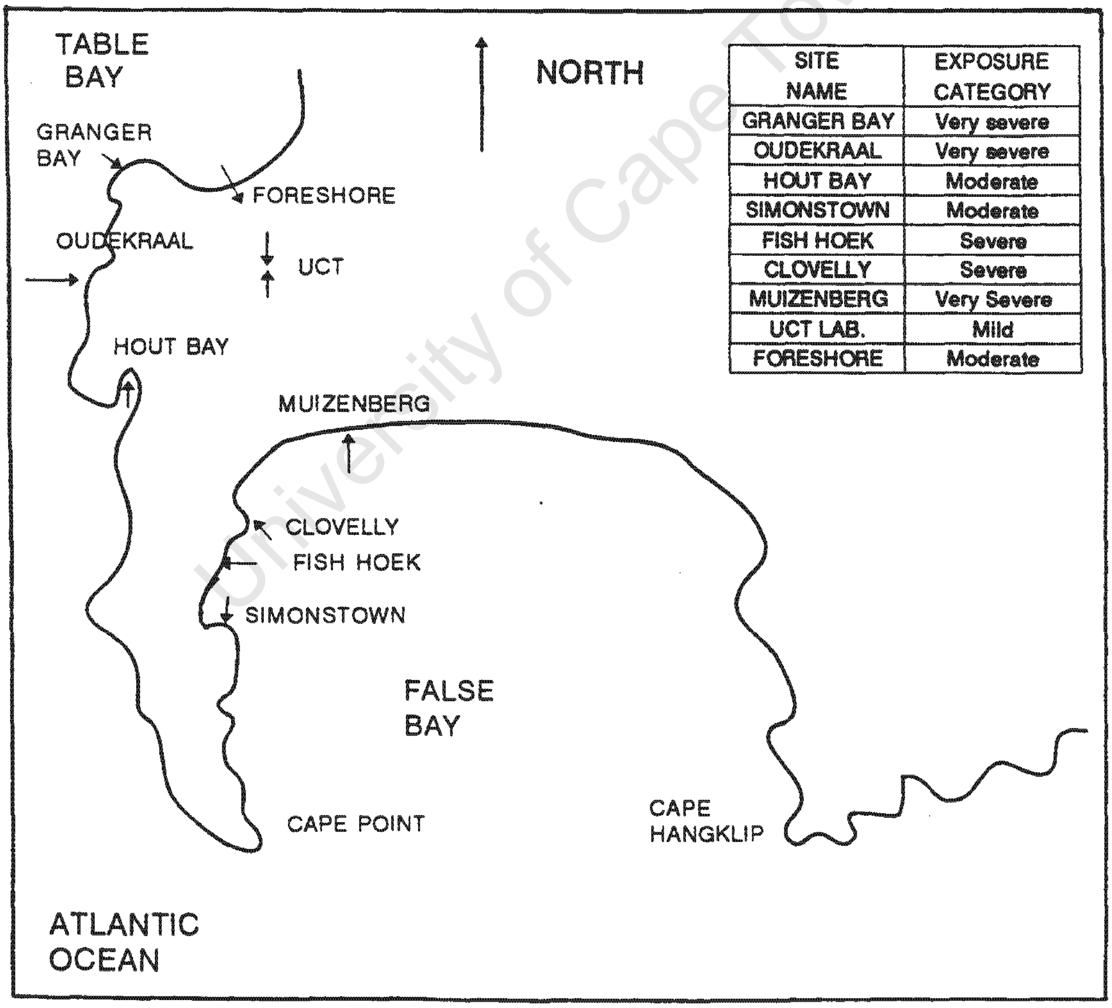
Laboratory control specimens were kept at the University of Cape Town under mild inland conditions. The site is located 6 km from Table Bay to the North and 18 km from False Bay in the South. Concrete specimens were either stored outdoors or in the laboratory under water at a temperature of between 15 and 20°C.

4.2.4 Severity of Marine Exposure

Exposure conditions vary considerably in the marine environment depending on a wide range of factors such as micro and macroclimate, position relative to the sea, orientation and exposure to prevailing winds. Research has been done measuring chloride aerosol levels using gauze targets but this is too time consuming to do at several sites and only the surface deposition of chlorides is determined [15,16]. It was believed that more information could be gathered about the severity of exposure by using concrete specimens which were exposed for a period of one year at a number of marine sites and only tested at the end of the exposure period. Nine locations around the Cape Peninsula were selected for the work as shown in Figure 4.6. The locations, orientations and local exposure conditions of the concrete panels are given in Appendix Table A 11. Sites were chosen to get a wide range of exposure conditions and a simple exposure rating system was devised which divided the marine spray zone into four categories according to the potential severity of exposure.

Marine Exposure Zone	Description	Distance From Sea
Very Severe	Exposed to heavy wave action and strong onshore winds with some direct splashing	< 50m
Severe	Exposed to heavy wave action and strong onshore winds	< 1000m
Moderate	Sheltered coastal location with moderate wave and wind action	< 1000m
Mild	Inland marine conditions	> 1000m <30000m

The classification system chosen represented a practical compromise between the complexity of factors which influence the marine spray zone and the need for a simple system with limited data input. Exact definitions of factors such as heavy wave action and strong onshore winds do not necessarily improve the accuracy of the system as conditions may change or accurate meteorological data may not be available. Some environments may also be exposed to extreme conditions for only a few months of the year such as at Granger Bay while other sites such as Muizenberg may have continously extreme conditions. Conservative assumptions must therefore be used to determine the worst case scenario for each site being considered. More sophisticated methods may be investigated once sufficient information has been generated but the high local variations in exposure militate against a precise classification system.



SITE NAME	EXPOSURE CATEGORY
GRANGER BAY	Very severe
OUDEKRAAL	Very severe
HOUT BAY	Moderate
SIMONSTOWN	Moderate
FISH HOEK	Severe
CLOVELLY	Severe
MUIZENBERG	Very Severe
UCT LAB.	Mild
FORESHORE	Moderate

Figure 4.6 : Location of Panels Around the Cape Peninsula

4.3 Experimental Programme

Concrete blocks were taken to site 28 days after casting and placed in either the tidal or spray zones. Twelve hours before being taken to site the concrete was saturated with water to ensure that the concrete did not absorb excessive sea water on initial exposure on site. The concrete was therefore allowed to equilibrate with ambient conditions from a saturated condition. The concrete blocks were tested at regular intervals using either insitu or semi-destructive testing by coring and drilling.

4.3.1 In situ Testing

In situ testing of the concrete blocks was done at 3, 6, 9, 12, 18 and 24 months using the following methods.

a) Concrete Resistivity

In situ resistivity testing was done using a portable Wenner probe and digital resistivity meter as developed by Millard [33]. Concrete specimens were lightly brushed to remove any surface deposits which might interfere with resistivity measurements and readings taken across the width of each block. Resistivity is dependent on the internal moisture content, diffusivity of the material and the pore solution conductivity. The resistivity results could therefore be used to assess changes in the concrete pore structure resulting from continued hydration and salt penetration. Resistivity measurements are adversely affected by carbonation, wetting profiles and highly conductive surface layers so care had to be taken to minimize these effects on site. Readings were taken during periods of stable weather.

b) Ultrasonic pulse velocity

Ultrasonic pulse velocity (UPV) measurements were taken to assess the general soundness of the material and to indirectly monitor the strength development of the concrete. Measurements were taken across the width in the middle third of each concrete block to minimise local variations which occur in the concrete between the top and bottom.

c) Visual Assessment

Visual assessment of the concrete surface was done with respect to weathering, abrasion damage and rust staining. The assessment was made in a subjective, descriptive manner and no attempt was made to measure the deterioration observed. Details of the subjective rating system are given in Table 4A3.

4.3.2 Concrete Core Samples

Two concrete cores of 68 mm diameter were extracted at an age of twelve months using a portable coring machine. The core holes were patched with a high strength cementitious mortar. This testing after one years exposure was done to determine if concrete characterization techniques used at 28 days could be used to determine durability performance of concrete in service. The outer 25 mm of each core was used for durability index testing while the inner 30 - 90 mm portion of each core was tested for core strength. Details of the test methods used to determine oxygen permeability, water sorptivity, chloride conductivity, abrasion resistance and core strength are given in Chapter Three.

4.3.3 Concrete Powder Samples

Drilled concrete powder samples were extracted from the concrete blocks after 12, 18 and 24 months using a rotary hammer drill. Three 20 mm holes were drilled and the powder combined to form one sample for depths of 5 - 25 mm, 25 - 50mm, 50 - 75 mm, 75 - 100 mm from the concrete surface. Drill holes at any time interval were a minimum of 75 mm from previously drilled holes or block surfaces. The depth increments were fairly deep considering that the concrete would only be exposed to the marine environment for two years but it was felt that chlorides should be tested at depths relevant to marine concrete cover depths which vary between 50 and 100 mm. It was realised however that some high strength concrete would have negligible chloride beyond 50 mm after two years. Drilled powder samples were used in preference to cores due to the time advantages of the former method and concern about washing out chlorides during the coring process. Initial testing of the technique indicated that negligible contamination from the sides of the drill hole occurred when drilling at depths up to 100 mm provided the drill bit was maintained perpendicular to the concrete surface.

Chloride contents were measured in accordance with BS 1881 Part 124 : 1988 except that the acid soluble chlorides were analysed using a potentiometric titration rather than using a volumetric back titration with thiocyanate [34]. The potentiometric titration was done using an auto titrator having stabilized the acidified concrete powder with a buffer solution of sodium acetate to a pH of approximately 4.0. The potentiometric titration requires a weak acidic solution to measure a distinct equivalence point. Chloride contents were expressed in terms of the cement content of the original concrete mix. Each chloride result was the average of three titrations with outliers being discarded using the method given by ASTM E178-80 [35].

4.3.4 Severity of Marine Spray Zone

In a separate investigation, sixteen panels of size 150x150x50 mm were cast during the summer of 1994 using a grade 40 OPC concrete in accordance with details given in Chapter 3. The specimens were stripped after 24 hours and air-cured for six days at 23°C and 50% R.H. and saturated in tap water for twelve hours before being taken to site at eight days. The panels were saturated before exposure to allow the concrete to equilibrate with ambient conditions without excessive initial absorption taking place. Exposure of the concrete panels to the marine environment was done by attaching the specimens to vertical surfaces of marine structures. Details of the arrangement of the concrete panels on site are shown in Figure 4.7.

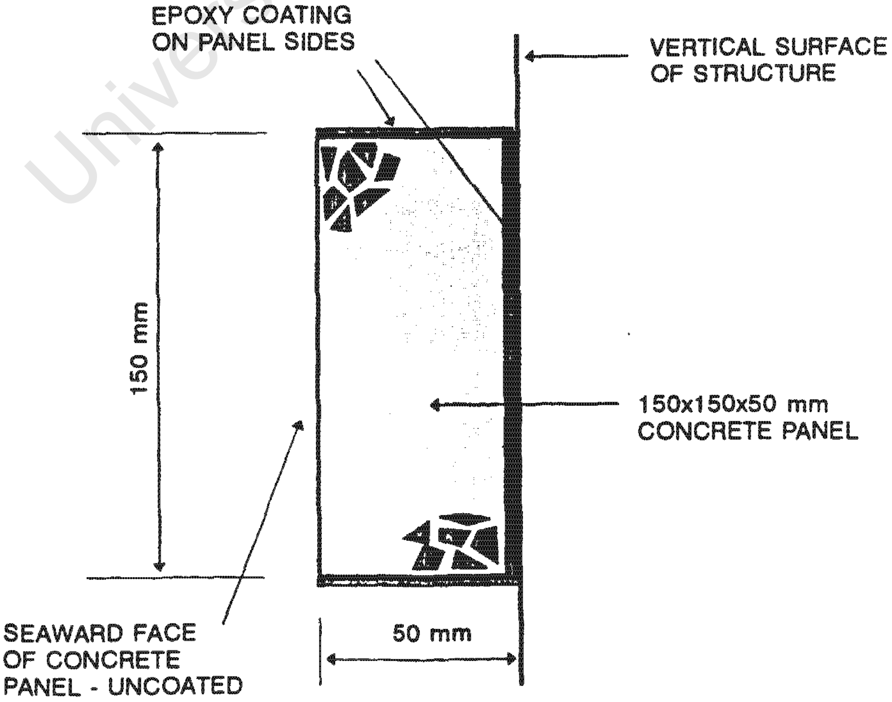


Figure 4.7 : Details of Marine Spray Zone Panels

The severity of the marine spray zone was determined by measuring the chloride contents in concrete panels after twelve months exposure to the marine environment. Drilled powder samples were extracted from the concrete at 5mm depth increments between 0 and 25 mm and analysed for chlorides in accordance with 4.3.2a. The test method assumes that the relative penetration of chlorides into concrete is an accurate indicator of the severity of exposure. One panel was lost (P5) during the twelve month period when the brick wall on which it was attached was demolished by a motor vehicle.

4.4 Concrete Exposure Results

4.4.1 In situ Results

In situ results for concrete blocks exposed to the marine tidal and spray zones, and laboratory controls are given below.

a) Concrete Resistivity

Resistivity values for all concrete types were initially below 20 kOhmcm at 28 days but with exposure the resistivity of fly ash and slag concrete increased while OPC concrete resistivity remained fairly constant. Concrete in the tidal zone had lower resistivity than similar concrete in the spray zone due to the saturated conditions and higher levels of chloride penetration in the tidal zone. The effect of the concrete grade and extent of initial moist curing was not significant in comparison to variations caused by concrete type and environment. Resistivity results for concrete exposed to tidal conditions is shown in Figure 4.8a and 4.8b with full details in Appendix Table 4A1.

Concrete in the tidal zone at Simonstown was exposed to mild conditions with little surface drying. These steady conditions produced consistent trends with regard to resistivity values. In the spray zone at Simonstown moisture conditions in the concrete were dependent on seasonal weather fluctuations due to the lack of splash action from the sea. Resistivity values were therefore affected by ambient conditions, increasing during hot, dry summers and decreasing during cold, wet winters. This trend was less pronounced in the spray zone at Granger Bay due to heavy wave action of the sea which produced regular splash action.

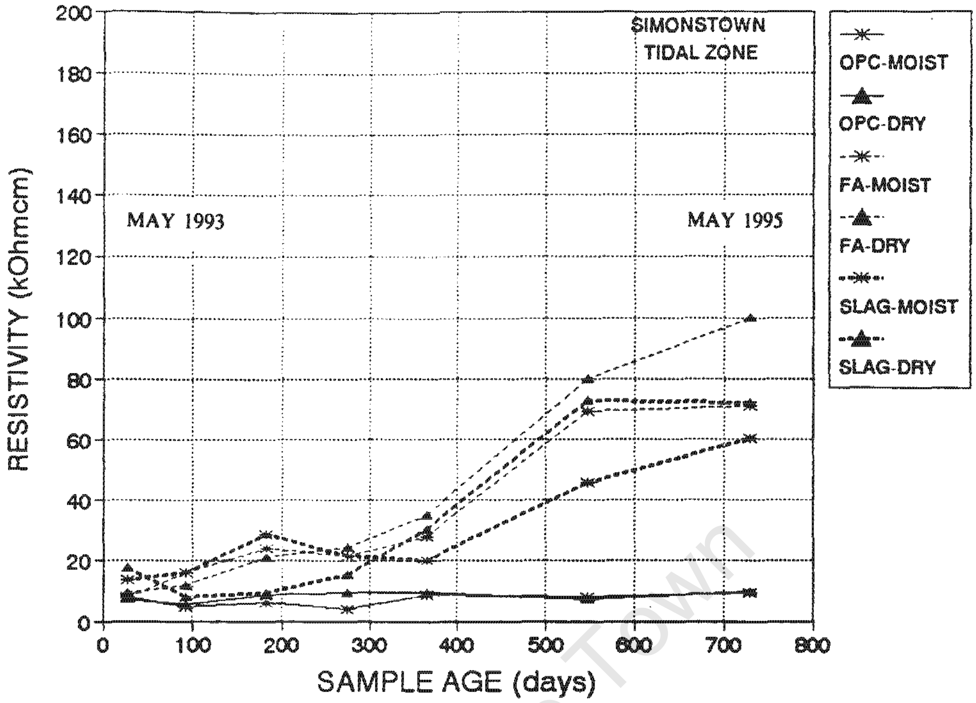


Figure 4.8a: Resistivity versus Time - Simonstown Tidal Zone

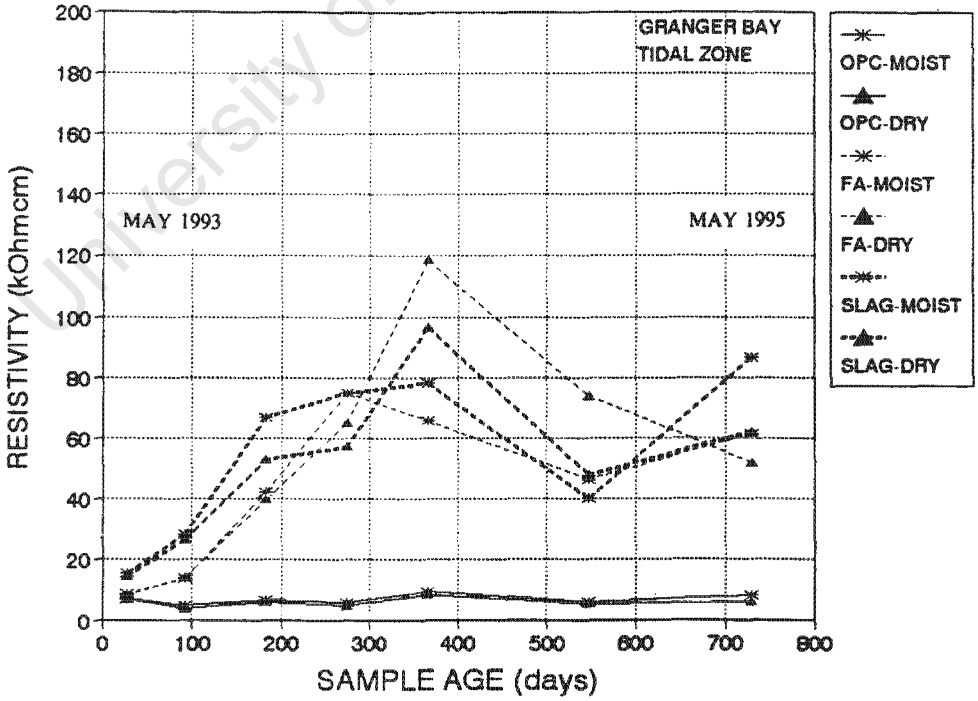


Figure 4.8b: Resistivity versus Time - Granger Bay Tidal Zone

b) Ultrasonic Pulse Velocity Results

UPV results increased with time for all environments and concrete types, with fly ash and slag concretes tending to have higher values than OPC concrete after two years. The grade of concrete had a significant effect on UPV results but surface changes resulting from initial curing and long-term drying had only a marginal effect. UPV results for dry cured slag concrete at Granger Bay were significantly lower than other concretes possibly due to abrasion damage which was severe for this type of concrete. Concrete in the tidal zone had slightly higher UPV values than similar concrete in the spray zone at both sites. UPV results for tidal zone samples are shown in Figure 4.9a and 4.9b.

After two years exposure UPV values appeared to have stabilized, suggesting that there was negligible further cement hydration and pozzalanic action. UPV values might decrease in the future as the concrete experiences damage from wet/dry cycles, abrasion and drying shrinkage which might affect the integrity of the concrete. Full UPV results are given in Appendix Table 4A2.

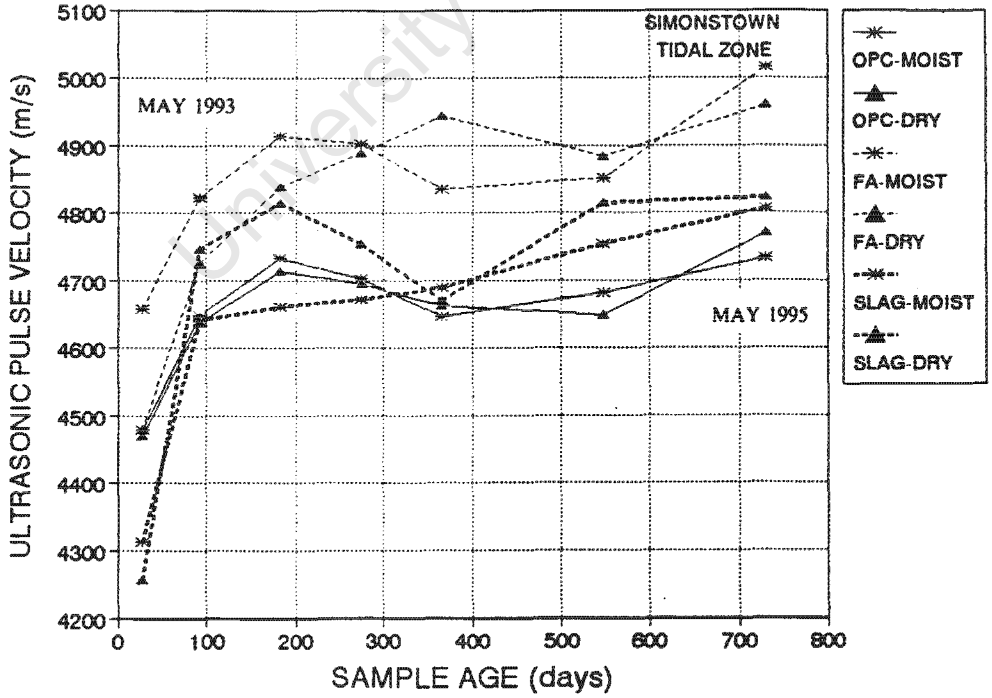


Figure 4.9a: UPV versus Time - Simonstown Tidal Zone

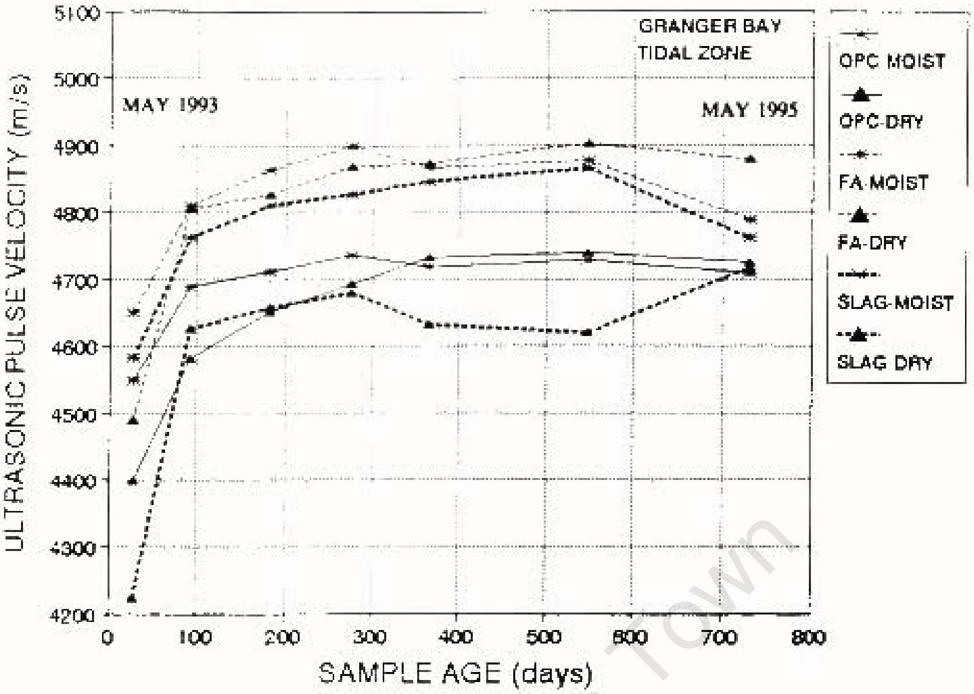


Figure 4.9b: UPV versus Time - Granger Bay Tidal Zone

c) Visual Appearance

Visual examination of concrete blocks after two years exposure revealed a wide range of surface conditions. Concrete in the spray zone at Simonstown showed very slight weathering in the form of slight surface roughness except for grade 20 fly ash and slag concrete which showed moderate weathering. In the tidal zone at Simonstown several specimens had moderate levels of weathering with exposed fine aggregate while grade 20 slag concrete had suffered major weathering with some exposed coarse aggregate. The general condition of the concrete in the tidal zone at Simonstown is shown in Figure 4.10a.

All concrete blocks at Granger Bay showed evidence of abrasion damage, particularly specimens in the tidal zone. The amount of abrasion damage of concrete in the tidal zone appeared to decrease with increasing portland cement content and length of moist curing. Grade 40 dry cured slag concrete was worst affected and had suffered extreme abrasion damage with exposed aggregate and corner damage. Specimens in the spray zone showed similar trends to those found in the tidal zone but were not as severely affected, being damaged only periodically during severe winter storms. Evidence of physical damage can be seen in Figure 4.10b while visual appearance results are given in Appendix Table 4A3.

Figure 4.10a: Condition of Simonstown Tidal Zone Samples

Figure 4.10b: Abrasion Damage of Granger Bay Tidal Zone Samples

4.4.2 Concrete Core Results (12 months)

Details of core strengths, oxygen permeability, sorptivity, chloride conductivity, abrasion resistance and carbonation results measured after 12 months exposure are given below.

a) Core Strength

Core strength results were affected by the extent of initial curing even though the sample was taken from between 30 and 100 mm from the surface. Moist curing produced higher core strengths than dry curing particularly for fly ash and slag concrete. This effect was most noticeable for control concrete but was also apparent for marine exposure concrete. Details of the core strength results are given in Appendix Table 4A4 and in Figure 4.11a and 4.11b.

Fly ash concrete generally had higher core strengths than OPC concrete if well cured; when dry cured the results were similar or higher. The results were however not sufficiently consistent to draw firm conclusions. Exposure to the marine environment appeared to assist the strength development of concrete with tidal zone concrete being on average 10% higher strength than wet cured control samples. Even concrete exposed to the marine spray zone had similar core strengths to wet cured control concrete.

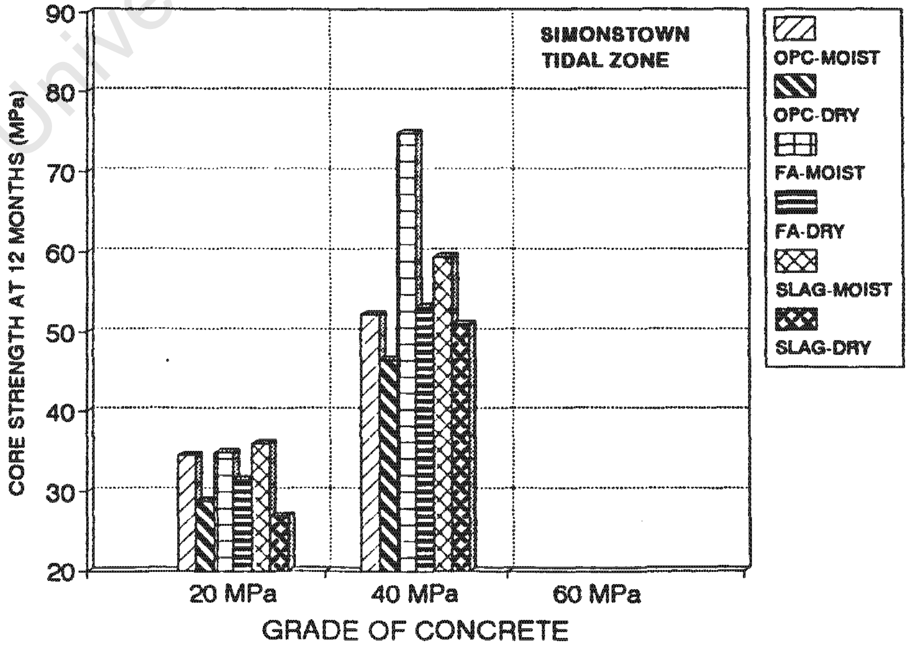


Figure 4.11a : Core Strength Results - Simonstown Tidal Zone

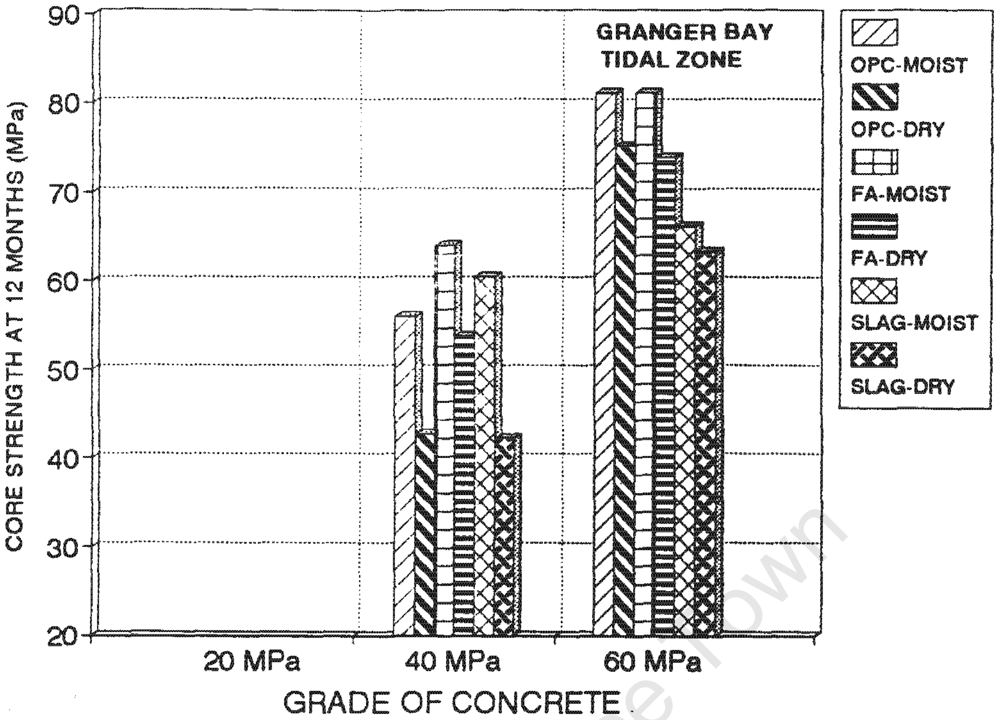


Figure 4.11b : Core Strength Results - Granger Bay Tidal Zone

b) Oxygen Permeability

Control samples tested for oxygen permeability showed similar trends after 12 months to those observed at 28 days where permeability decreased with increasing strength, extended moist curing and with the use of fly ash and slag. Concrete exposed to the marine environment produced similar trends except the effect of curing was less obvious. Spray zone concrete had higher permeability than tidal zone concrete due to the drier conditions which retarded cement hydration in the covercrete zone. This trend was more significant at Simonstown than at Granger Bay as the heavy wave action at Granger Bay resulted in regular wetting of the spray zone blocks. Details of the oxygen permeability results are given in Appendix Table 4A5 and Figures 4.12a and 4.12b.

Concrete in the tidal zone at Granger Bay had higher permeability than similar concrete at Simonstown due to the effects of abrasion which prevented the build up of surface deposits on the concrete. Laboratory wet cured concrete had similar permeability to concrete in the tidal zone at Granger Bay suggesting that no protective layer formed by surface deposition had occurred to the Granger Bay concrete.

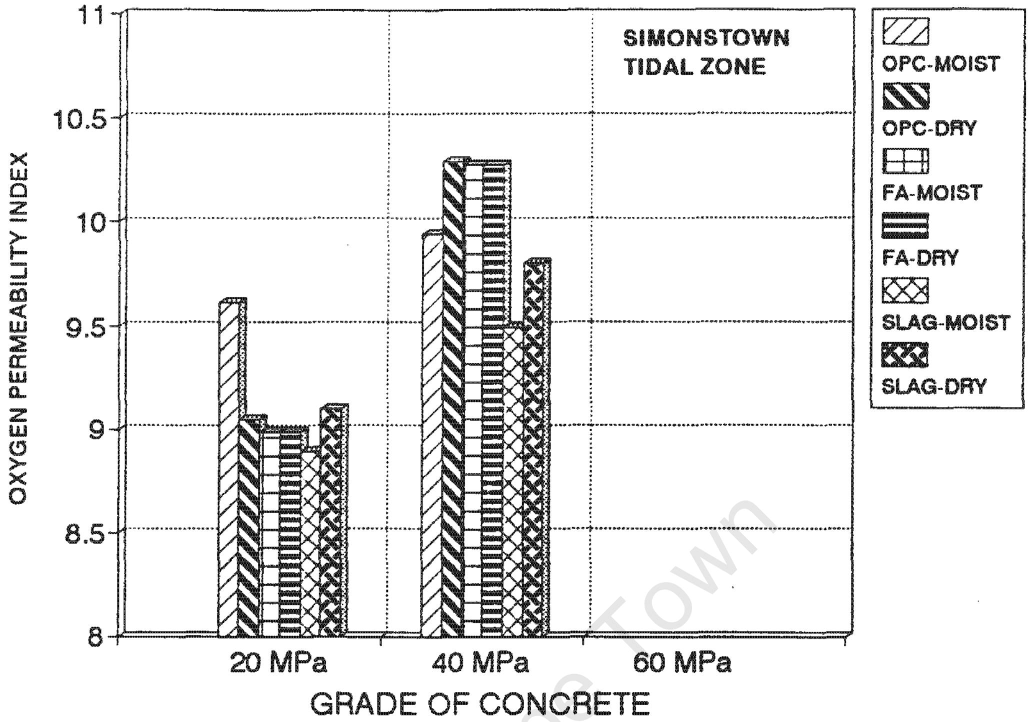


Figure 4.12a : Oxygen Permeability Index Results - Simonstown Tidal Zone

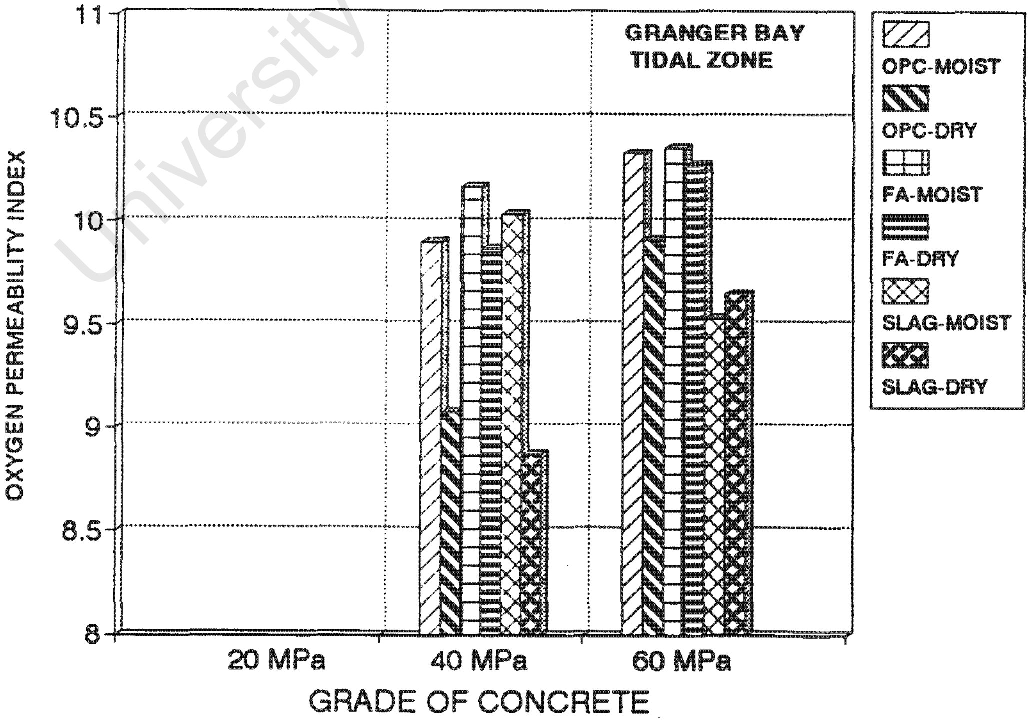


Figure 4.12b : Oxygen Permeability Index Results - Granger Bay Tidal Zone

c) Water Sorptivity

Carbonation and chloride penetration into concrete affect the surface properties and may interfere with the water absorption process resulting in reductions of sorptivity. Wet cured control concrete had higher sorptivity than either moist or dry cured concrete as the wet cured material had virtually no carbonation. Dry cured concrete with higher depths of carbonation had lower sorptivity than similar moist cured concrete. Details of the water sorptivity results are given in Appendix Table 4A6 and Figures 4.13a and 4.13b.

Marine exposure samples showed similar trends to those found at 28 days where water sorptivity decreased with increasing strength and use of fly ash and slag but increased for moist cured concrete. Water sorptivity of concrete from the tidal zone at Granger Bay was higher than similar concrete at Simonstown due to the effects of surface abrasion. Concrete tested from the tidal zone at Granger Bay had sorptivity values on average 70% higher than concrete from Simonstown.

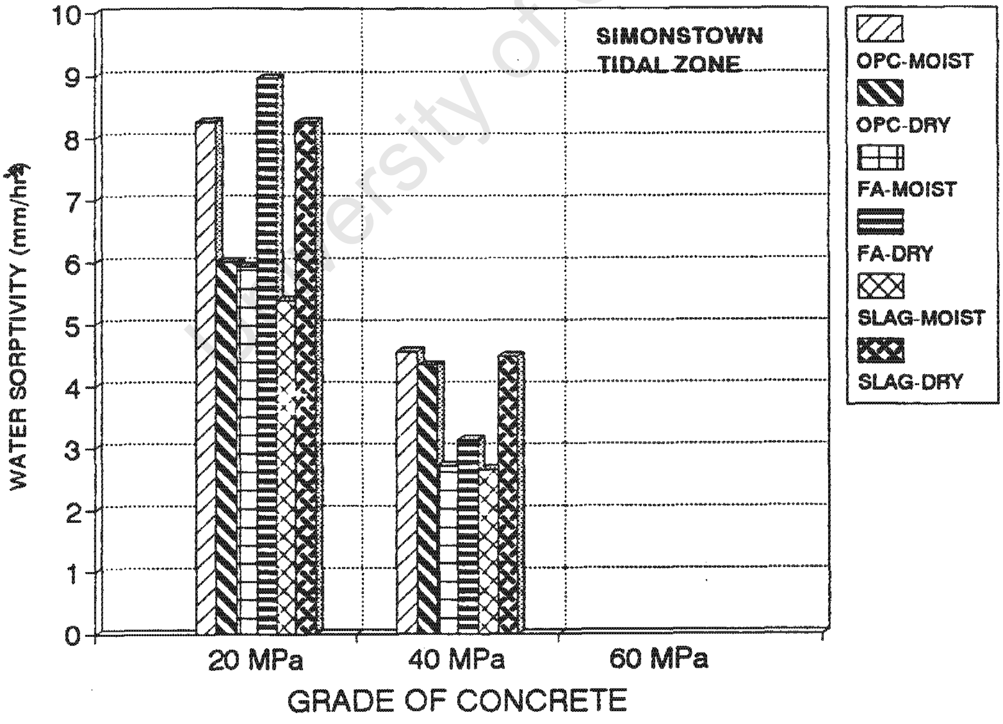


Figure 4.13a : Water Sorptivity Results - Simonstown Tidal Zone

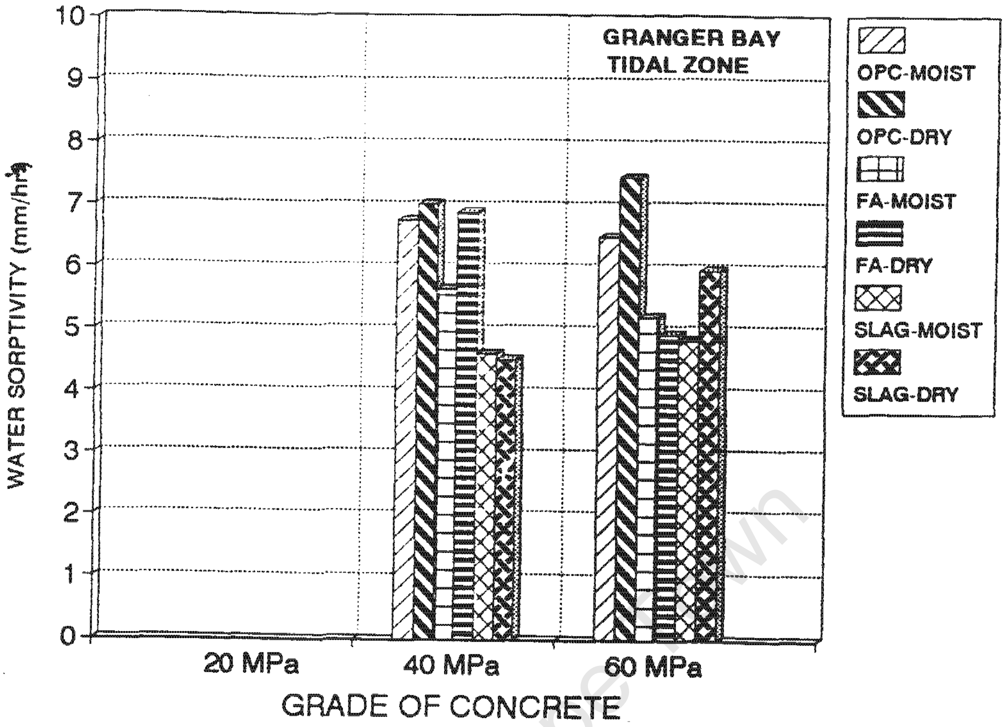


Figure 4.13b : Water Sorptivity Results - Granger Bay Tidal Zone

d) Chloride Conductivity

After one year the chloride conductivity of control concrete decreased with increasing strength, increasing durations of moist curing and by the inclusion of fly ash and slag. Concrete containing fly ash and slag had lower chloride conductivity than OPC concrete even when poorly cured. Concrete exposed to the marine environment showed similar trends but spray zone concrete at Simonstown had higher chloride conductivity results due to the relatively dry conditions. Details of chloride conductivity results are given in Appendix Table 4A7 and Figures 4.14a and 4.14b.

Tidal zone concrete at Simonstown had significantly lower chloride conductivity than concrete in the tidal zone at Granger Bay due to differences in surface abrasion. Chloride conductivity values for Simonstown concrete were on average 46% lower than those for Granger Bay which may be attributed to the surface blocking effect arising from deposition of insoluble salts on the surface of protected concrete.

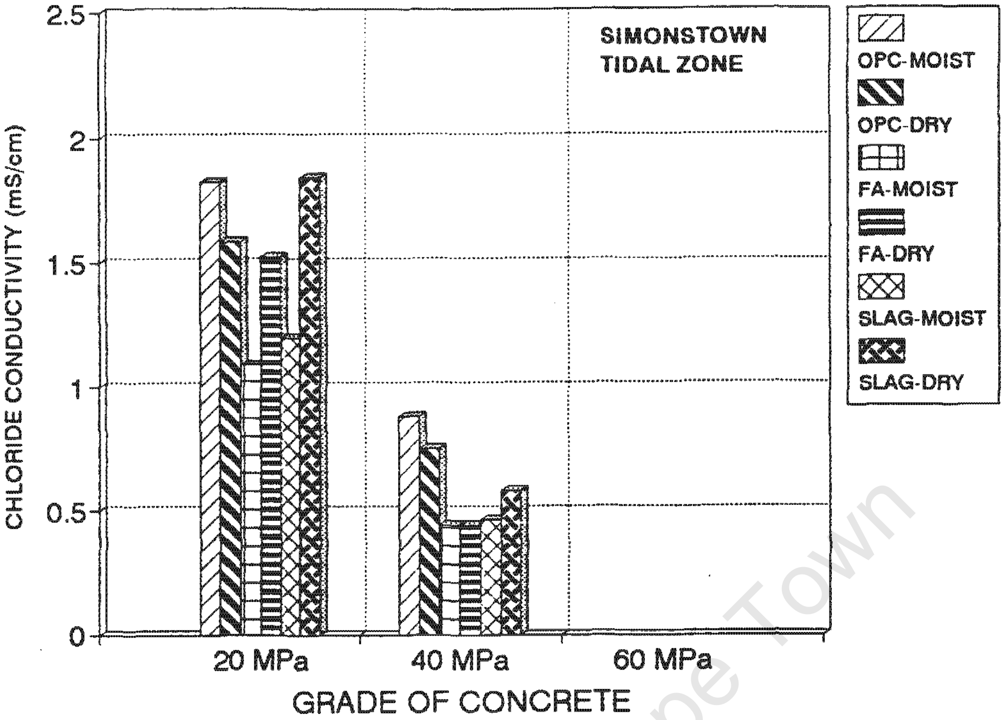


Figure 4.14a: Chloride Conductivity Results - Simonstown Tidal Zone

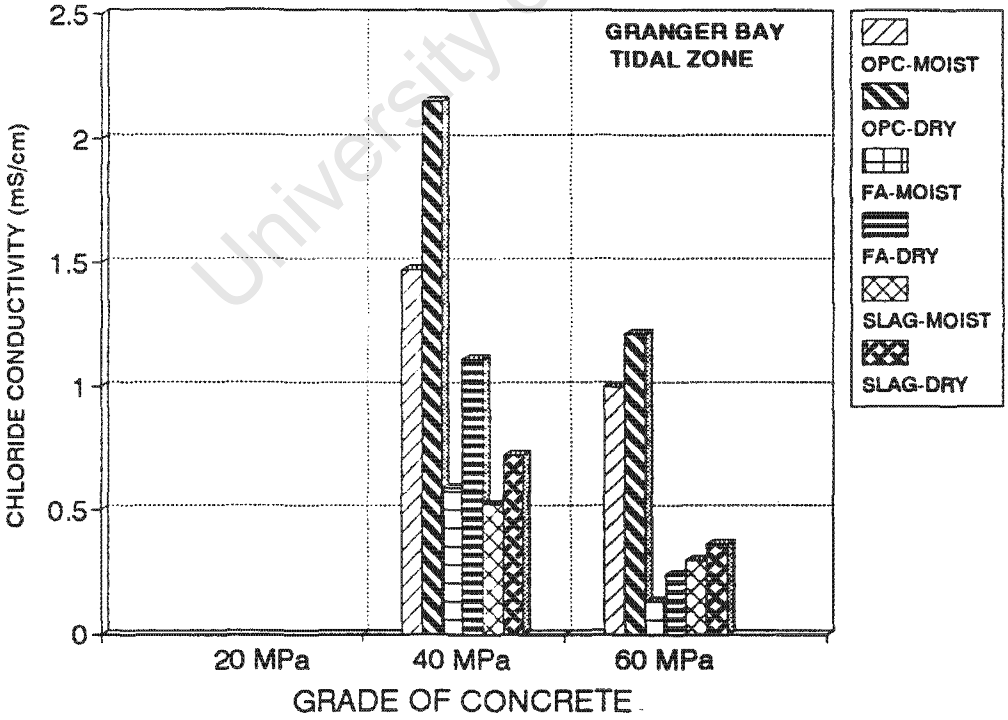


Figure 4.14b: Chloride Conductivity Results - Granger Bay Tidal Zone

e) Abrasion Resistance

Increased abrasion resistance was found for increasing strength, length of moist curing and amount of OPC used in the concrete when tested after twelve months. The only exception to these trends was wet cured OPC concrete which had lower abrasion resistance than moist and dry cured concrete. The continuous wet curing appeared to have produced a relatively weak surface skin with little carbonation. Details of abrasion resistance results are given in Appendix Table 4A8 and Figures 4.15a and 4.15b.

Results from Simonstown showed that spray zone concrete had lower abrasion resistance than concrete in the tidal zone. Fly ash concrete had marginally lower abrasion resistance compared to OPC concrete while slag concrete was significantly less resistant. The effect of initial curing was only obvious for the spray zone samples where dry cured concrete was less resistant to abrasion than moist cured concrete. Abrasion results from concrete at Granger Bay were difficult to compare to Simonstown and control results due to the extent of abrasion damage which had already occurred thereby exposing harder internal concrete.

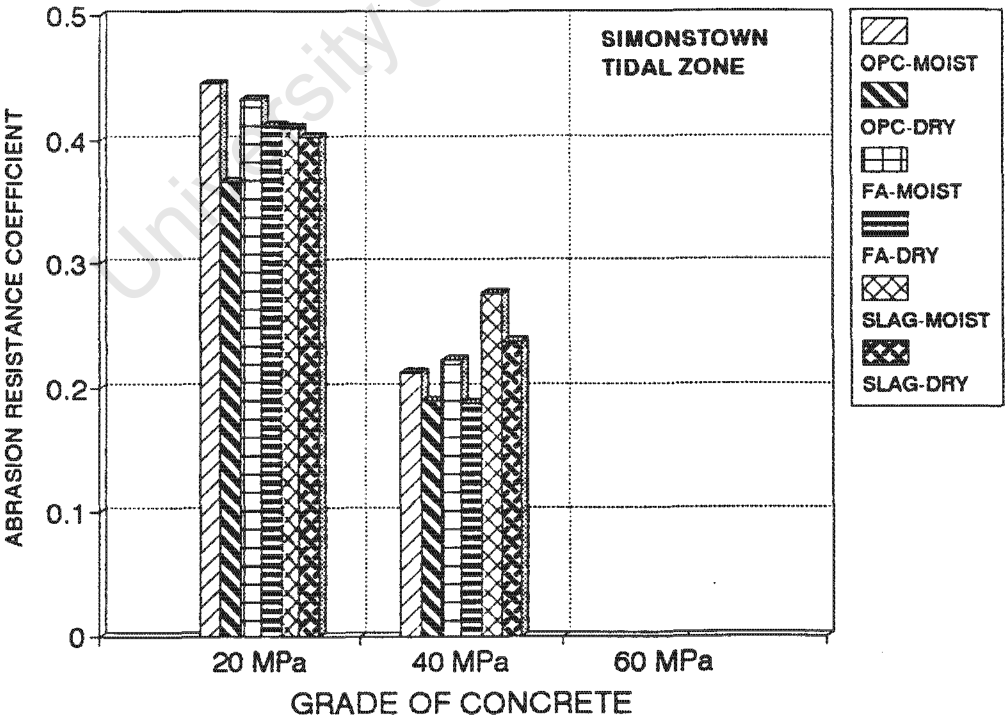


Figure 4.15a: Abrasion Resistance Results - Simonstown Tidal Zone

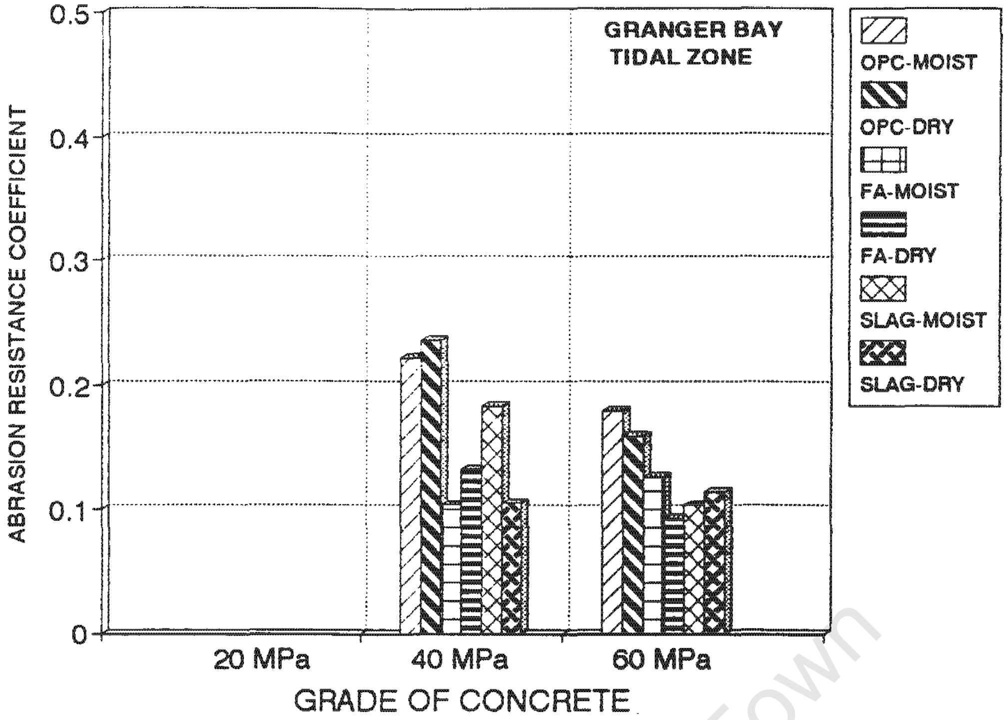


Figure 4.15b: Abrasion Resistance Results - Granger Bay Tidal Zone

f) Carbonation Depth

Carbonation depths were found to be higher in relatively dry exposure sites such as the Simonstown spray zone and the outdoor controls. Concrete exposed to relatively wet conditions had low carbonation depths due to the saturated nature of the concrete which restricted the access of carbon dioxide. Slag concrete was most vulnerable to carbonation followed by fly ash and OPC concrete respectively. Concrete which had been dry cured initially had higher carbonation depths than moist cured concrete. Details of the carbonation results are given in Appendix Table 4A9 and Figure 4.16a and 4.16b.

Concrete in the tidal zone at Granger Bay had lower carbonation depths compared to similar concrete at Simonstown. This may have been due to the abrasion damage of Granger Bay concrete where the outer carbonated layer had been effectively stripped off by abrasion. Carbonation depths of concrete in the tidal zone were all less than 3 mm after twelve months exposure which made comparisons arbitary. Accurate predictions of carbonation can be produced using oxygen permeability testing before exposure but allowance must be made for ambient conditions of exposure. Figure 4.16c shows the relationship between oxygen permeability index and carbonation depth after twelve months exposure outdoors at the University of Cape Town.

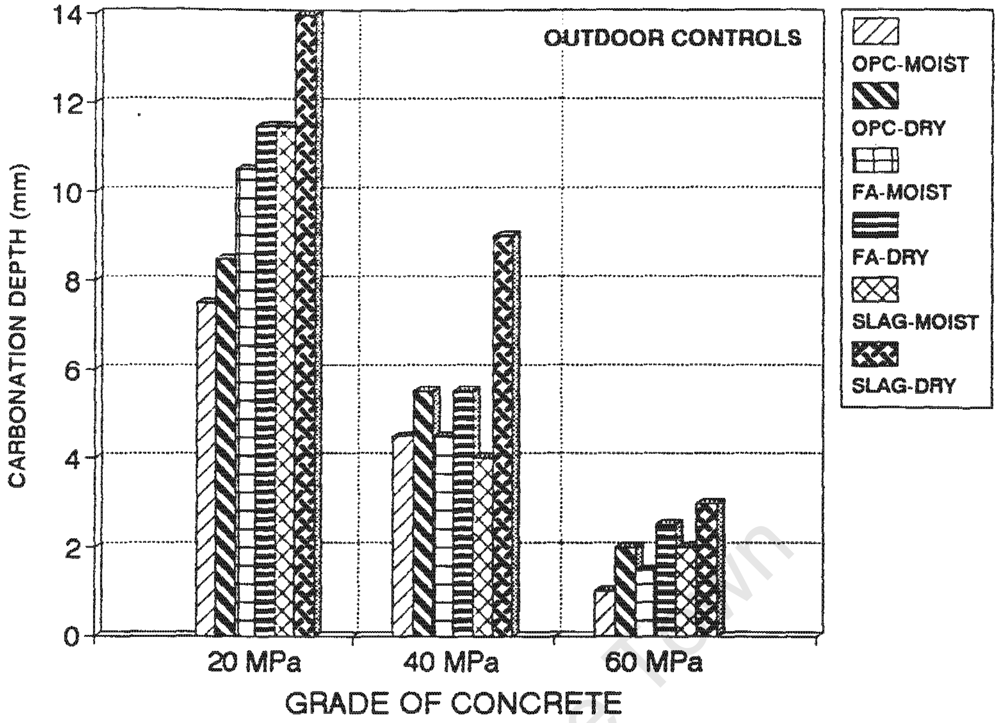


Figure 4.16a: Carbonation Depth - Outdoor Controls

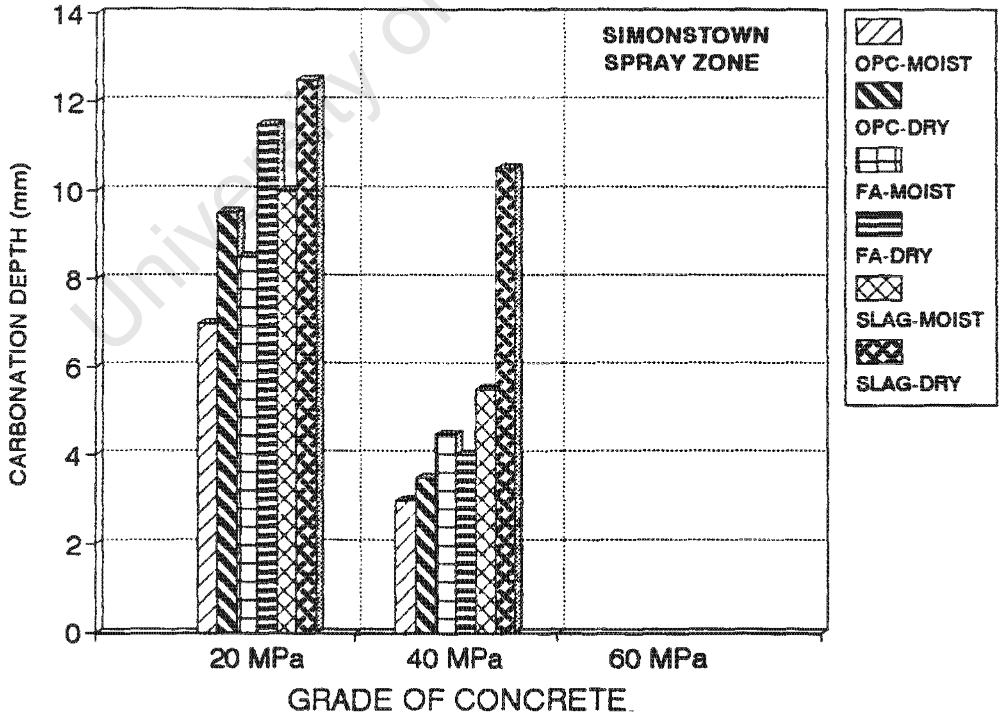


Figure 4.16b: Carbonation Depth - Simonstown Spray Zone

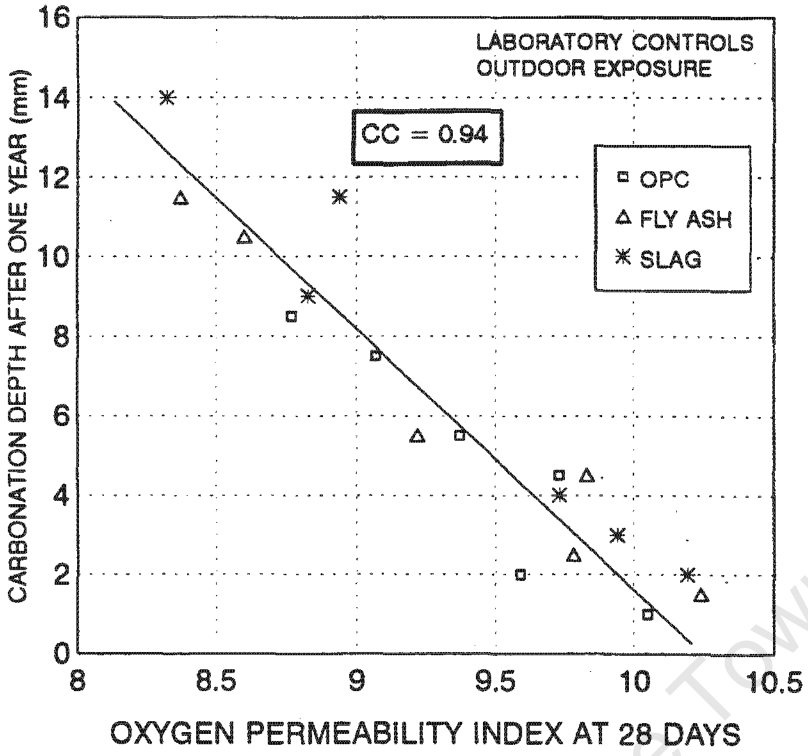


Figure 4.16c: Carbonation Depths versus Oxygen Permeability Index

4.4.3 Concrete Drill Powder Results

Results of testing done at Simonstown and Granger Bay after 12, 18 and 24 months are given below.

a) Chloride Content

Measured chloride contents in the concrete blocks were found to rise with increasing length of exposure to the marine environment. Several factors influenced the rate of penetration of chlorides into concrete. These factors are given in decreasing order of importance: concrete type, marine environment, concrete grade and length of initial moist curing. After two years exposure chloride penetrations depths in excess of 100 mm for low grade material were recorded while high quality concrete had penetration depths of less than 25 mm. Chloride contents together with diffusion coefficients and surface concentrations determined from Fick's Law for concrete specimens in the tidal and spray zones at Simonstown and Granger Bay are given in Appendix Table 4A10a-d

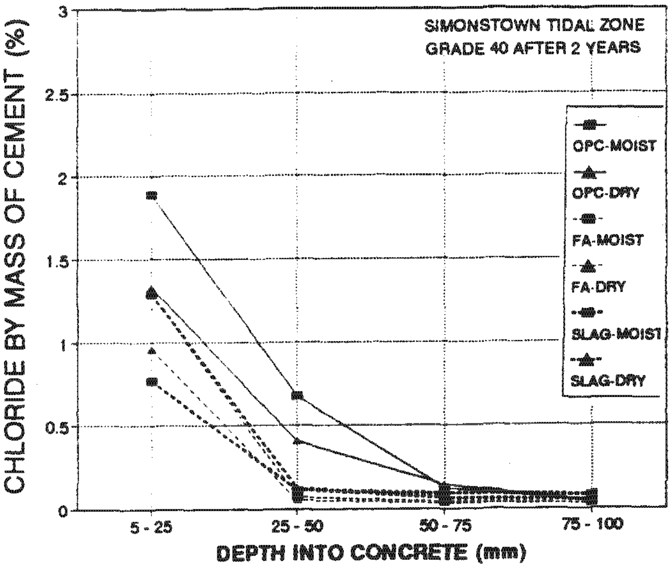


Figure 4.17a : Chloride Profiles
- Simonstown Tidal Zone

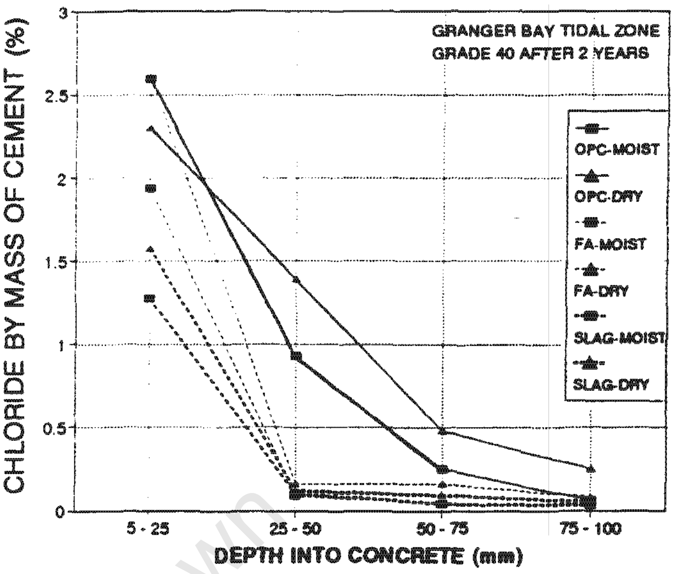


Figure 4.17b: Chloride Profiles
- Granger Bay Tidal Zone

OPC concrete had relatively poor chloride resistance in comparison with fly ash and slag concrete. Slag concrete had marginally lower chloride contents than fly ash concrete but both concrete types generally had minimal chloride contents at depths greater than 25 mm after two years exposure. Comparisons of grade 20 concretes showed that fly ash and slag concrete had chloride contents up to three times lower than similar OPC concrete. Dry cured grade 20 concrete with cement extenders had low chloride levels internally. These low chloride levels were not expected given the poor characterisation performance of these materials. Figure 4.17a and 4.17b show chloride profiles for grade 40 concrete in the tidal zone at Simonstown and Granger Bay.

The severity of marine exposure significantly affected chloride contents in concrete with the tidal zone at Granger Bay being the most severe environment. Concrete in the tidal zone at Granger Bay suffered regular abrasion damage which would have prevented the formation of protective deposits and allowed unrestricted access of chlorides into the concrete. Similar concrete at Simonstown under tidal conditions had lower chloride contents particularly for OPC concrete where mild conditions did not cause much surface damage and allowed some organic growth. The spray zone at Simonstown was very mild and chloride contents were uniformly low while the Granger Bay spray zone was far more severe resulting in chloride contents similar to those found in the tidal zone at Simonstown. The extremely low chloride levels recorded from concrete in the spray zone at Simonstown were at the limit of the sensitivity of the chloride analysis (<0.10 %). This resulted in fluctuating internal chloride levels with time which often did not follow the general trend of increasing concentrations with time (see Appendix Table 4A10b). Comparison of chloride profiles from grade 40 OPC concrete for the different exposure zones are shown in Figure 4.18a and 4.18b.

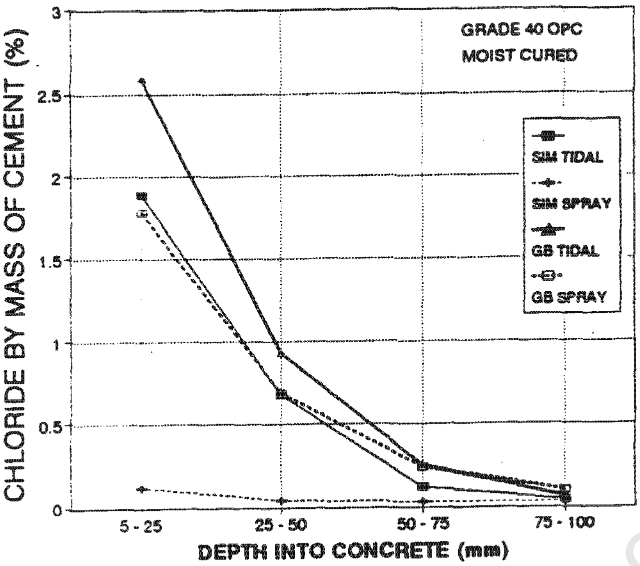


Figure 4.18a : Chloride Profiles - Moist Cured

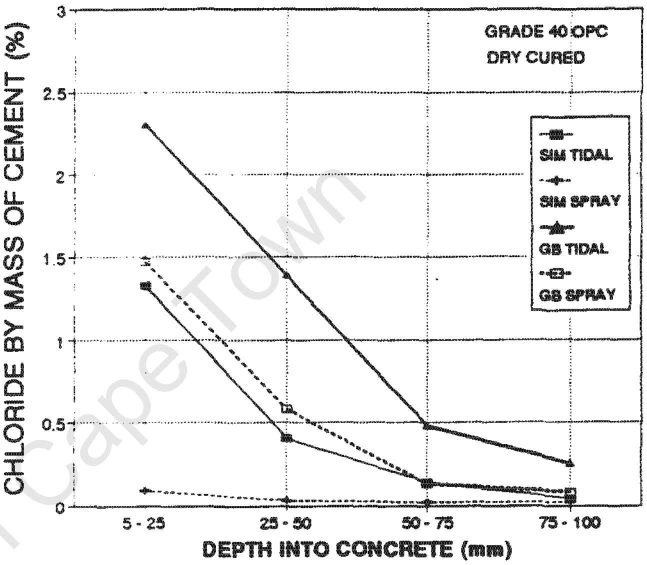


Figure 4.18b: Chloride Profiles - Dry Cured

Higher concrete grades had lower chloride contents but the effect was not as significant as concrete type. Grade 60 OPC concrete had similar chloride contents to those of grade 20 fly ash and slag concrete which shows the advantage of using cement extenders rather than simply increasing the cement content as is the current practice. Moist cured concrete generally had lower chloride contents than similar dry cured concrete but this trend was not always consistent. The moist exposure conditions may have allowed some delayed curing to occur which might have assisted the dry cured concrete in particular. The effect of initial curing on chloride contents in concrete was less significant with depth which is to be expected given that curing and early age drying generally affect the microstructural development of the outer "curing affected zone" which extends between 30 and 50 mm from the concrete surface. The effect of different concrete grades and types of curing on the consequent penetration of chlorides is shown in Figures 4.19a and 4.19b for OPC concrete in the tidal zone at Simonstown and Granger Bay respectively.

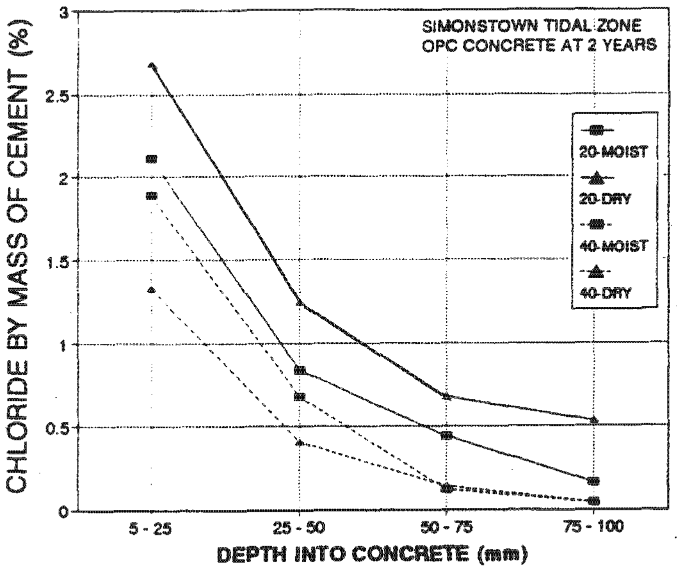


Figure 4.19a: Chloride Profiles - Simonstown OPC Concrete

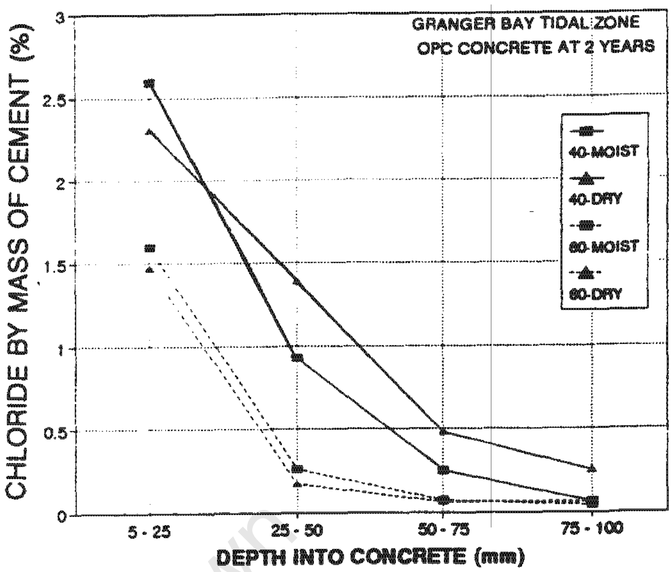


Figure 4.19b: Chloride Profiles - Granger Bay OPC Concrete

Using the solution of Fick's Law, chloride profiles were defined in terms of two parameters: the diffusion coefficient and the surface concentration. Diffusion coefficients were found to reduce with length of exposure, particularly for fly ash and slag concrete. Grade 40 OPC concrete in the tidal zone at Granger Bay remained relatively constant while similar concrete at Simonstown decreased slightly between 12 and 24 months. The decrease in diffusion coefficient with time has been reported by several researchers and is generally ascribed to chloride binding, continued cement hydration and surface pore blocking [22,27]. The relative contribution of each of these mechanisms is not known but it may be assumed that concrete in the tidal zone at Granger Bay had minimal pore blocking due to the severe abrasion of the concrete surfaces.

The change in diffusion coefficient with time is important for predictions of long-term chloride penetration and data at later ages is needed to confirm emerging trends. Bamforth has conducted similar marine exposure work for longer periods and this data provides useful information about long-term trends [27]. After six years exposure to the marine splash zone OPC concrete had similar diffusion coefficients to those measured at six months while fly ash and slag concrete had decreased steadily over the six years period. A review of literature on the subject showed diffusion coefficients decrease with time for all three concrete types but fly ash and slag concrete decrease more rapidly than OPC concrete and may decrease by two orders of magnitude after 50 years [36]. Diffusion coefficients for concrete in the tidal zone at Simonstown and Granger Bay are shown in Figures 4.20a and 4.20b.

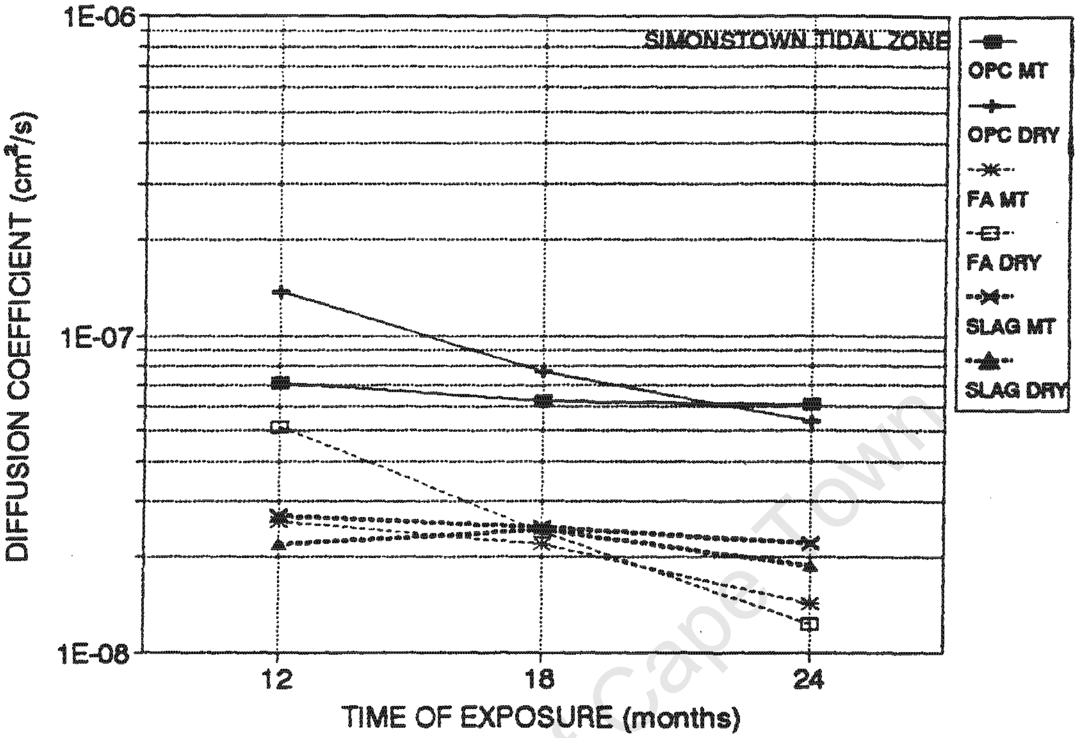


Figure 4.20a: Diffusion Coefficient vesus Time - Simonstown Tidal

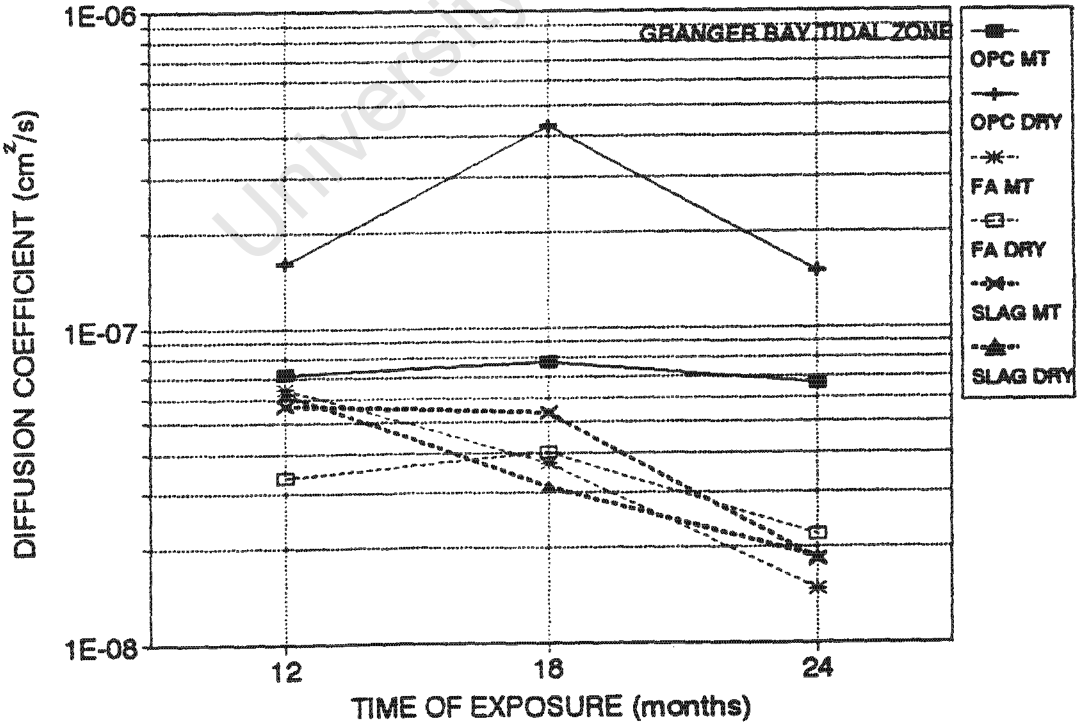


Figure 4.20b: Diffusion Coefficient versus Time - Granger Bay Tidal

The apparent surface concentration, C_s for each profile was determined from the best fit line through the data point, extrapolating back to zero depth. Physical measurement of the actual concentration is complicated by fluctuations caused by climatic conditions and local surface variations which makes calculated values more reliable. Values determined varied considerably but it appeared that surface concentrations increased with time as shown in Figure 4.21a and 4.21b. These results are in agreement with findings which indicate that surface concentrations may take several years to stabilize in some environments [22]. This is in conflict with other researchers who believe that surface concentrations stabilize rapidly after exposure to the sea [21]. Surface concentrations appear to be approaching stable values after two year exposure but further research is required to draw more reliable conclusions for different environments and materials.

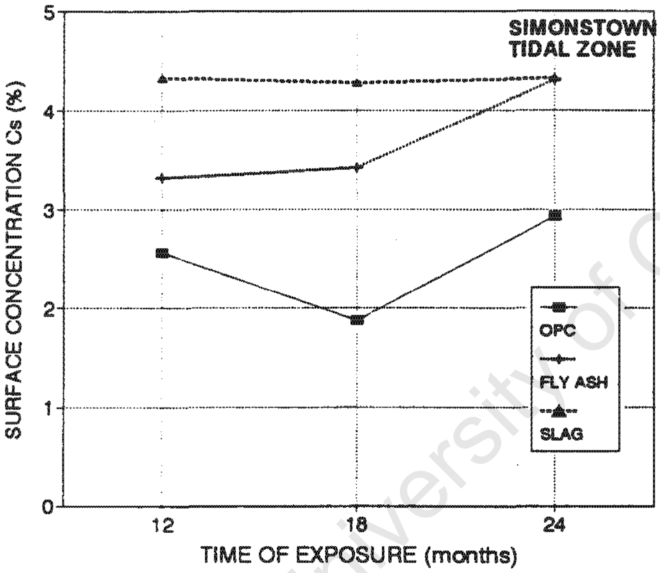


Figure 4.21a : Surface Concentration - Simonstown Tidal Zone

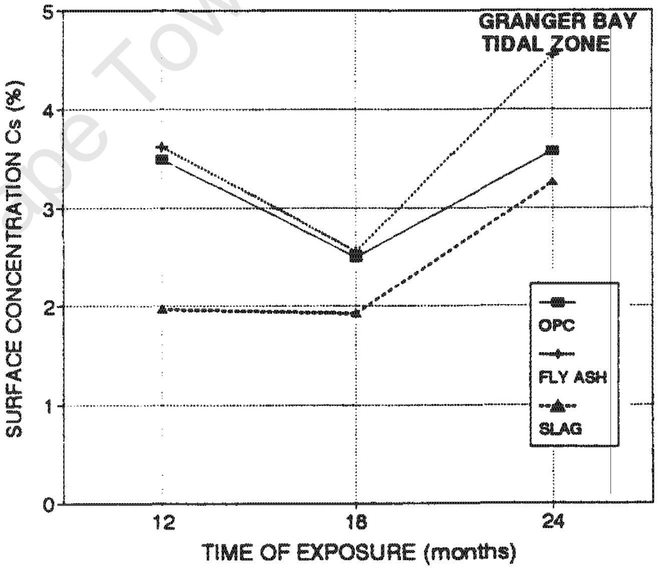
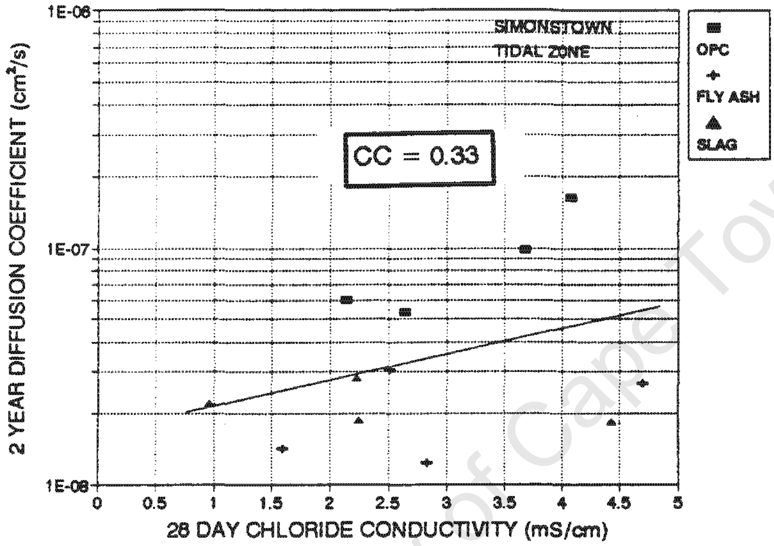
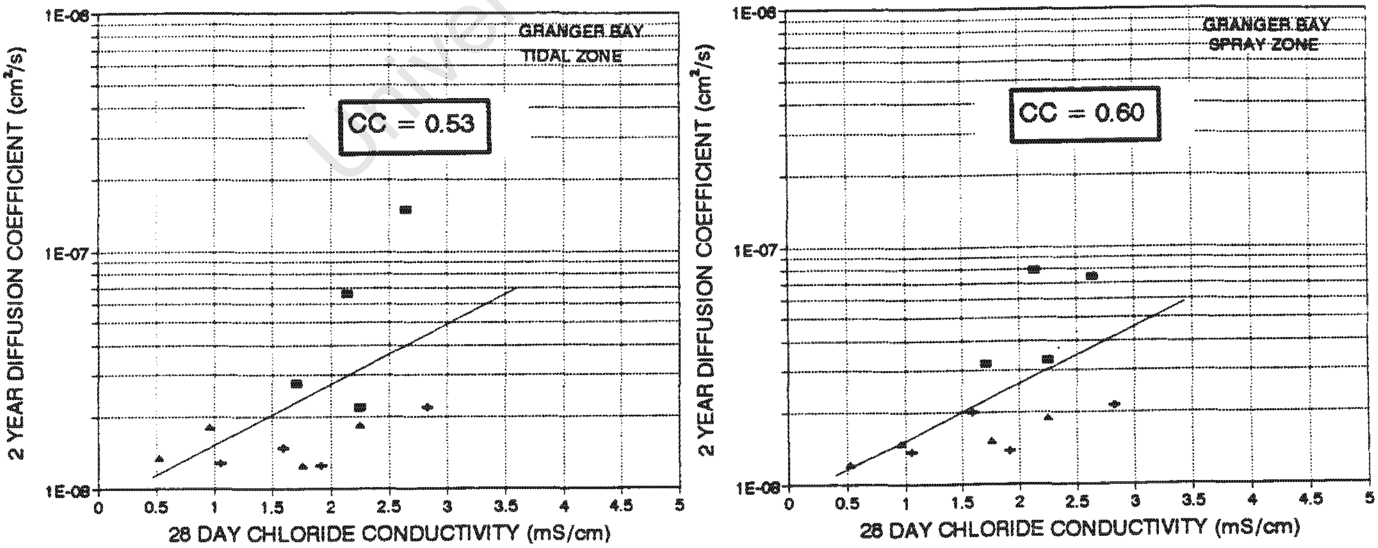


Figure 4.21b: Surface Concentration - Granger Bay Tidal Zone

The reliability of the chloride conductivity test as a predictive technique was assessed by comparing characterization results of moist and dry cured concrete recorded at 28 days with diffusion coefficients measured after two years of exposure in the tidal zone. Figure 4.22 shows the correlation between diffusion coefficient and chloride conductivity for tidal zone concrete at Simonstown and tidal and spray zone concrete at Granger Bay. The relationship between diffusion coefficient and chloride conductivity was poor particularly for Simonstown data where grade 20 and 40 concrete was used. The poor correlation was thought to be due to long-term effects of continued cement hydration and chloride binding which change the concrete pore structure significantly. The correlation between diffusion coefficient and 28 day chloride conductivity was found to be improved when comparing the results of moist cured concrete only. Each concrete type does however appear to have a unique relationship between diffusion coefficient and chloride conductivity. This meant that chloride conductivity values recorded at 28 days could not be directly related to long-term performance in the marine environment. A modification of the chloride conductivity test was therefore required to estimate the potential long-term chloride resistance of concrete.



a) Simonstown Tidal Zone



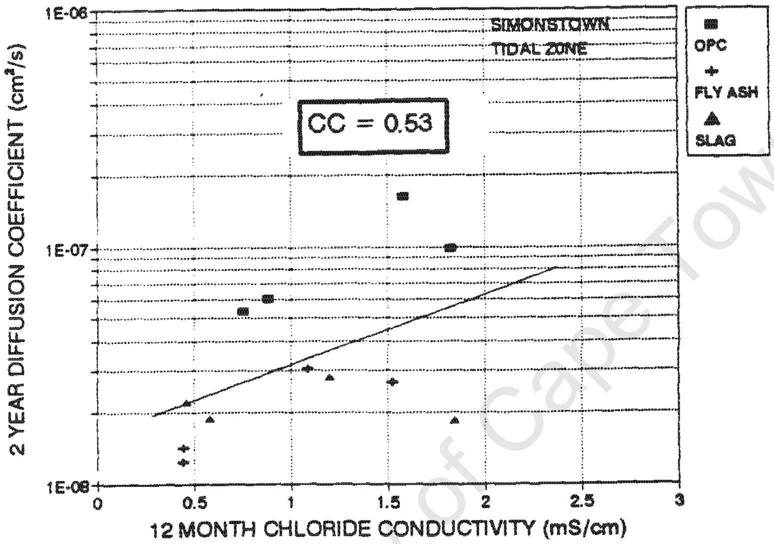
b) Granger Bay Tidal Zone

c) Granger Bay Spray Zone

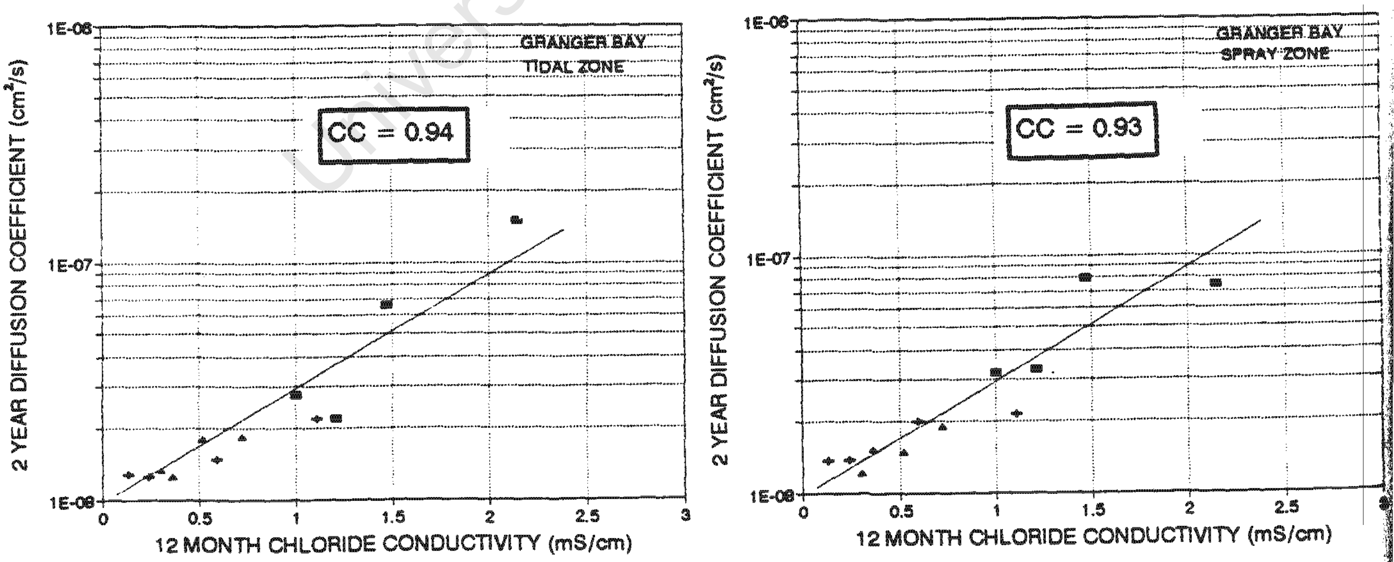
Figure 4.22: Diffusion Coefficient versus 28 day Chloride Conductivity

b) Extended period chloride conductivity tests

Chloride conductivity results recorded after 12 months exposure were much lower than those measured at 28 days with smaller differences in chloride conductivity between moist and dry cured concrete. Plotting chloride conductivity values after twelve months marine exposure against diffusion coefficient produced fairly poor to good correlations as shown in Figure 4.23. The poorer correlation of Simonstown concrete was thought to be due to the inherent variability of the grade 20 concretes which were not used at Granger Bay.



a) Simonstown Tidal Zone



b) Granger Bay Tidal Zone

c) Granger Bay Spray Zone

Figure 4.23: Diffusion Coefficient versus 12 Month Chloride Conductivity

This approach of using 12 month chloride conductivity results was able to quantify the long-term chloride resistance of concrete. It would however be an impractical method for determining the potential durability of marine concrete given the length of time and resources required to determine these values. A more rapid test which can measure the long-term effects of chloride binding and cement hydration is therefore required to characterize each particular concrete type so that chloride conductivity reading recorded at 28 days may be adjusted to allow for long-term effects.

During initial characterization of concrete, described in Chapter Three, a preliminary assessment of the effect of chloride binding and continued cement hydration on the concrete pore structure was undertaken. Concrete core samples extracted from wet cured specimens were initially saturated in a 5M NaCl solution 28 days after casting in accordance with the standard test practice for chloride conductivity testing. The chloride conductivity was measured after two hours saturation in the concentrated salt solution and each sample then retested at 14 day intervals until chloride conductivity values had reached a constant value after 98 days. The observed reduction of chloride conductivity was ascribed to the effects of chloride binding and continued cement hydration of the concrete. Figures 4.24a-c show the reduction of chloride conductivity with time exposed to a 5M NaCl solution while the ratio of chloride conductivity at 98 to 28 days is shown for OPC, fly ash and slag concrete in Figure 4.24d.

The intention of the extended period chloride testing was to simulate the reduction of conductivity which occurs when concrete is exposed to seawater. The test is able to accelerate the chemical interactions by using a 5M NaCl solution and vacuum saturation such that "long-term" conditions may be achieved in as little as three months. The technique results in considerably more rapid reductions than occur under natural marine conditions. The reduction factors determined were for continuously wet cured concrete, but moist and dry cured concrete were expected to exhibit similar if not greater reductions between 28 and 98 days.

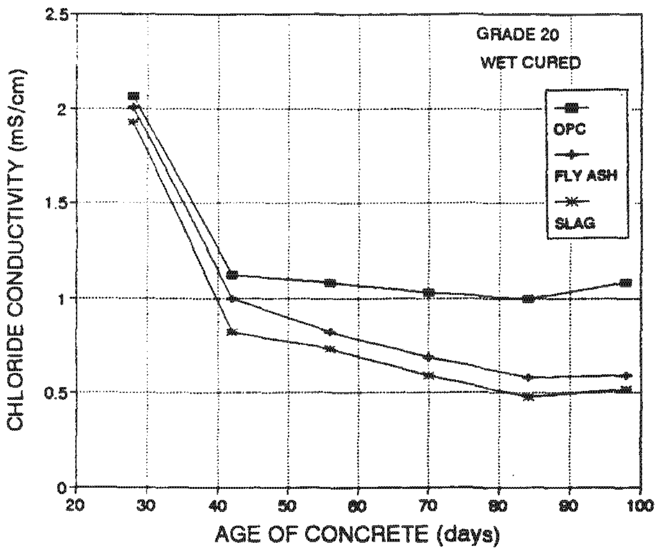


Figure 4.24a: Chloride Conductivity versus Time - Grade 20 Concrete

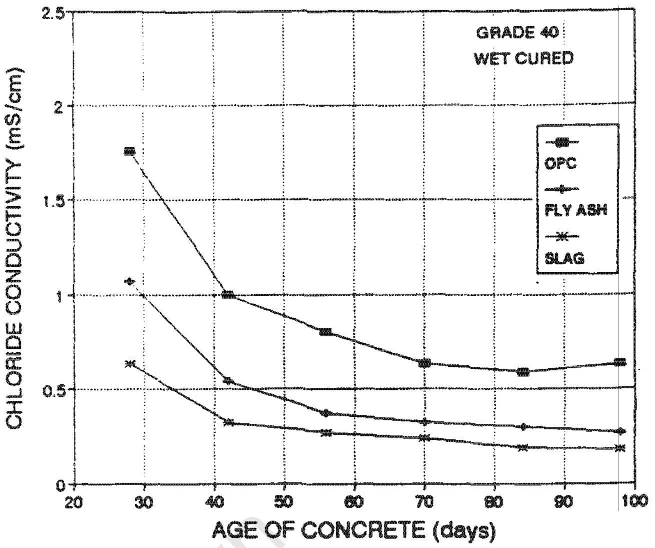


Figure 4.24b: Chloride Conductivity versus Time - Grade 40 Concrete

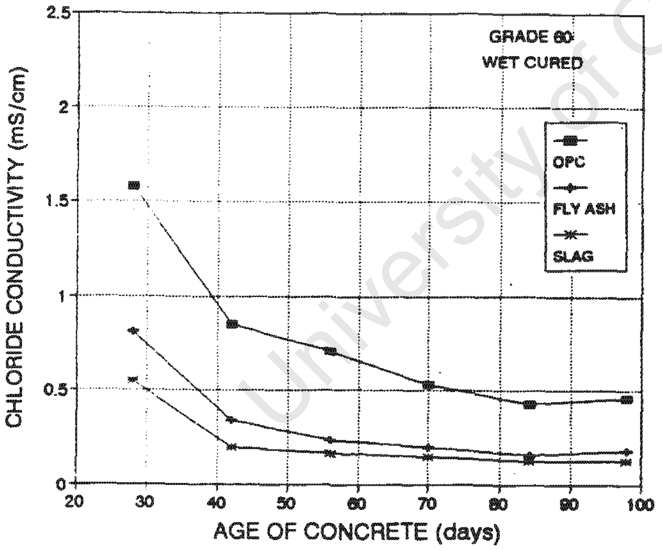


Figure 4.24c: Chloride Conductivity versus Time - Grade 60 Concrete

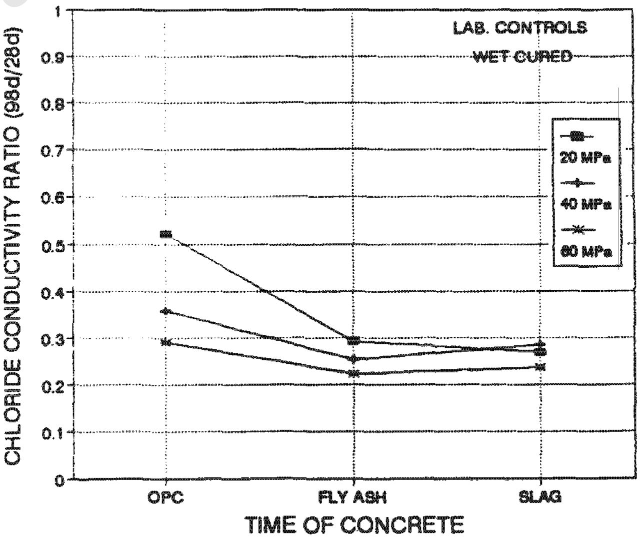
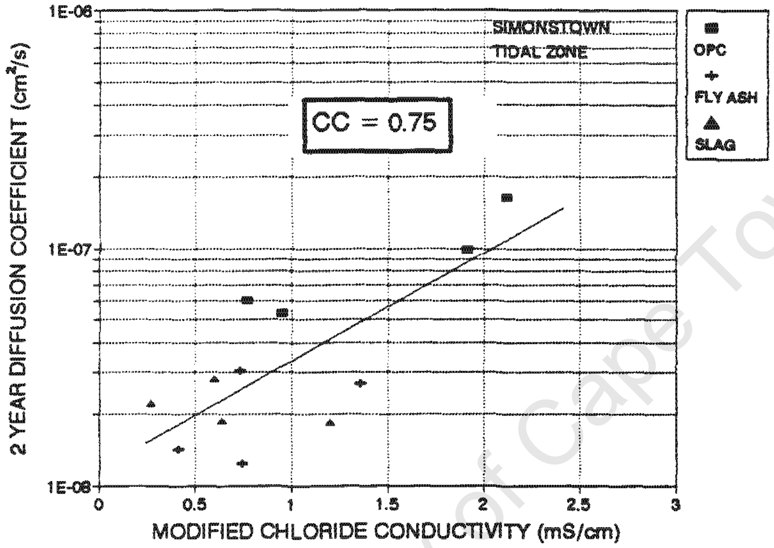
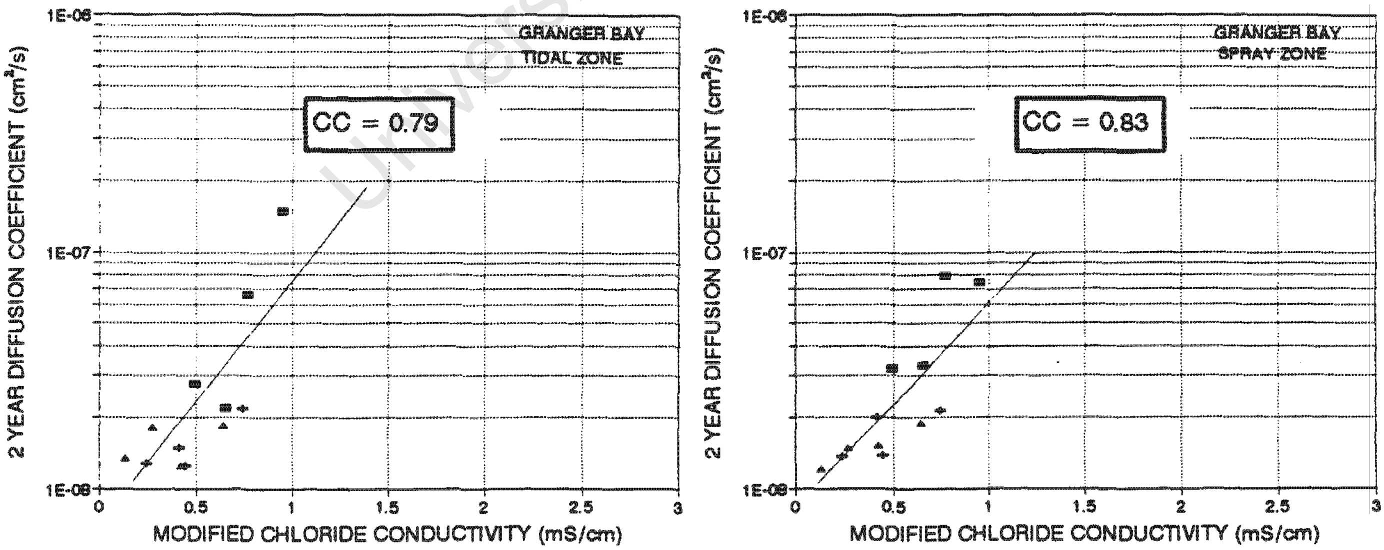


Figure 4.24d: Chloride Conductivity Ratio For Different Concretes

A modified chloride conductivity was determined by multiplying the 28 day chloride conductivity values recorded during initial concrete characterization with the chloride conductivity ratios determined from wet cured concrete. Figure 4.25 show the relationship between diffusion coefficient and the modified chloride conductivity values for the three marine environments. The correlation between diffusion coefficient and modified chloride conductivity was poorer than those achieved using 12 month chloride conductivity results. The correlation coefficients achieved of between 0.75 and 0.83 were however thought to be reasonable for predictive purposes (see further statistical analysis in Chapter Seven). Dry cured fly ash and slag concrete still appear to be penalized but this may be justified in drier environments exposed to occasional wetting and drying action. It should be noted that the test accelerates the chloride binding process and may cause anomalies (salt crystallization and microstructural damage) that do not occur under normal marine conditions.



a) Simonstown Tidal Zone



b) Granger Bay Tidal Zone

c) Granger Bay Spray Zone

Figure 4.25: Diffusion Coefficient versus Modified Chloride Conductivity

4.4.4 Severity of Exposure Results

Measured chloride contents from the concrete panels varied significantly as shown in Appendix Table 4A12. Exposed sites such as Muizenberg and Oudekraal had considerably higher chloride contents than sheltered sites such as Simonstown and Hout Bay. Chloride contents tended to reduce with increasing distance from the sea but some specimens at sheltered sites such as Simonstown had negligible chloride penetration despite being located immediately adjacent to the sea. Inland specimens all had low levels of chloride penetration as would be expected but even concrete placed 18 km from the coast had some chloride deposition which indicates that the limits of the marine environment with respect to concrete structures should extend to at least 30 km.

Chloride profiles for the four exposure categories are shown in Figure 4.26a-d. From the chloride results it would appear that the chosen classification system of marine spray zones produces a reasonable assessment of severity of exposure although little difference was evident between moderate and mild conditions. Differences in chloride profiles appeared to be dependent on the relative rate of deposition of chlorides on the concrete surface, wet-dry cycles and internal saturation levels in the concrete. Some chloride profiles showed a peak value in the 5-10 mm depth range indicating some washing out of chlorides from the concrete surface.

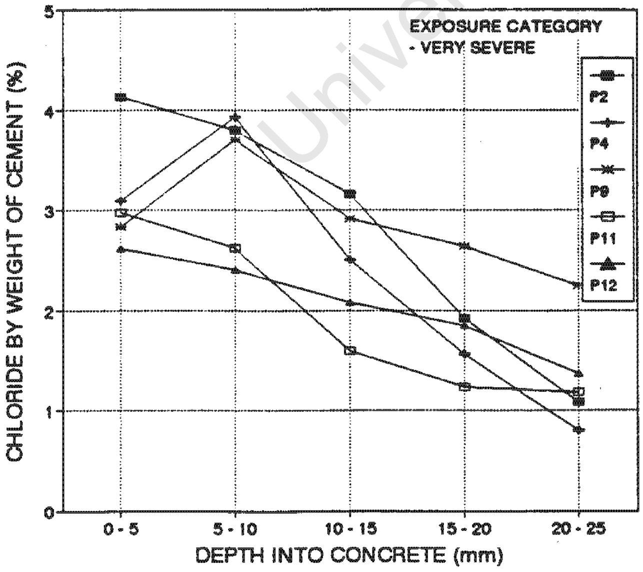


Figure 4.26a: Chloride Profiles for Very Severe Marine Spray Zone

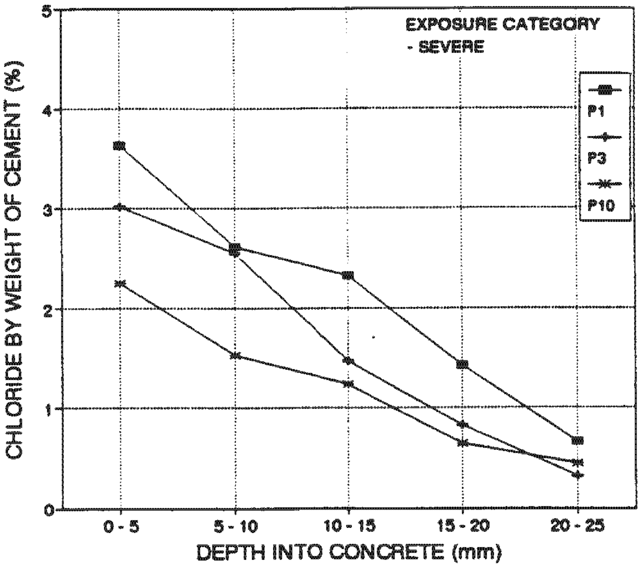


Figure 4.26b: Chloride Profiles for Severe Marine Spray Zone

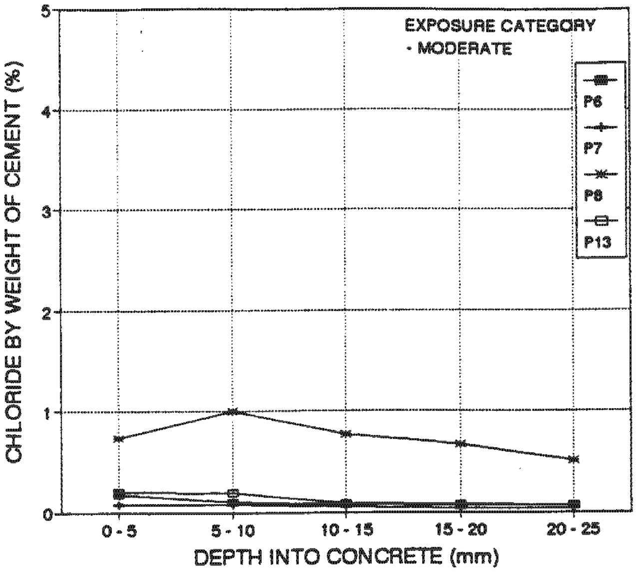


Figure 4.26c: Chloride Profiles for Moderate Marine Spray Zone

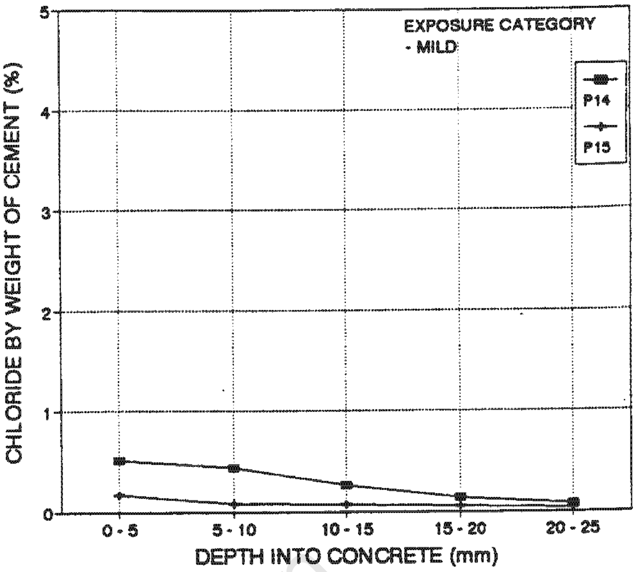


Figure 4.26d: Chloride Profiles for Mild Marine Spray Zone

Using the solution of Fick's Law, the diffusion coefficient and surface concentration were determined for each chloride profile. Both diffusion coefficients and surface concentrations were lower for decreasing levels of exposure but results varied significantly. The reduction of diffusion coefficient when exposure is less severe was consistent with results reported in section 4.4.3a. The diffusion coefficient is not an intrinsic material property such as diffusibility and is therefore affected by both material and environmental factors (such as severity of exposure). The results showed that classifying the marine spray zone merely in terms of distance from the sea is misleading and other factors such as wave action and prevailing wind must be considered. The correlation between marine spray zone category, diffusion coefficient and surface concentration are shown in Figure 4.27a and 4.27b. Methods of classification of marine spray severity may be improved with more experimental data but the chosen system must be sufficiently robust to allow for seasonal variations and local effects.

More data is required to confirm the proposed marine spray zone categories but the method shows promise. Visual observations from site confirm that severe zones have considerably more signs of structural deterioration than sheltered sites. Buildings along the Muizenberg coast line in particular have widespread corrosion damage while similar structures in the spray zone at Simonstown are generally in good condition. While there may be construction and material differences between structures in the two areas it is likely that environmental effects are largely responsible for the observed difference in durability performance.

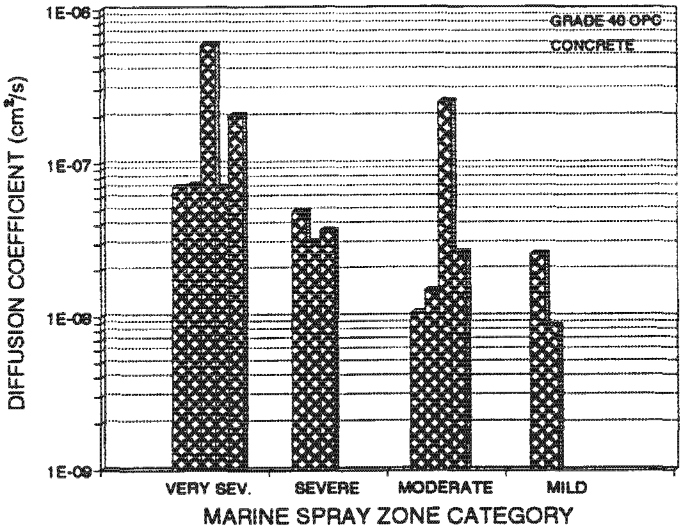


Figure 4.27a: Diffusion Coefficient versus Marine Spray Zone Category

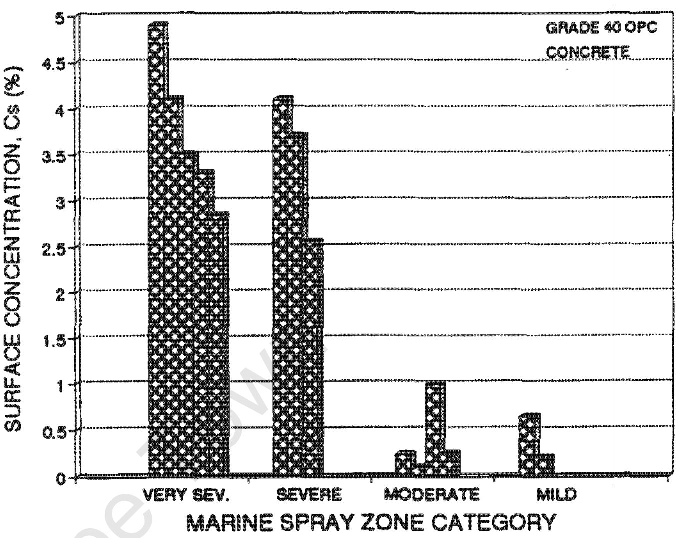


Figure 4.27b: Surface Concentration versus Marine Spray Zone Category

4.5 Conclusions

The wide ranging nature of the investigations into marine exposure of concrete produced a number of important observations. A summary of the conclusions drawn from this work are given below.

4.5.1 Concrete Type

The medium-term durability performance of concrete as measured by chloride ingress rates was improved with concretes containing cement extenders, increasing grades of concrete, and length of initial moist curing. These results confirmed indications of relative potential durability from initial concrete characterisation testing. OPC concrete had poorer chloride resistance than either fly ash or slag concrete and would provide limited protection to reinforcement from marine attack.

Fly ash and slag concrete had poorer abrasion and carbonation resistance than OPC concrete and showed greater signs of surface damage after exposure to the marine environment. The poor physical performance of cement extender materials was particularly evident when the concrete was dry cured initially. Care must therefore be taken when using these materials in concrete applications where weathering resistance is of primary concern.

4.5.2 Marine Environment

Exposure conditions in the marine environment vary considerably and little guidance is given in current design codes for estimating the severity of exposure. Conditions in the tidal zone are less variable and easier to quantify than in the spray zone which can change dramatically even within a fairly localized area. Existing methods of categorising marine exposure are vague and do not provide sufficient guidance for accurately assessing the likely exposure conditions. A more rational system for determining marine spray zone categories was proposed which shows promise.

4.5.3 Measuring Durability Performance

Durability index tests may be useful for characterizing concrete at early ages but after a period of exposure the results are affected by pore structure changes and deterioration mechanisms such as chloride ingress, carbonation and cracking. This may produce misleading results making later-age durability index tests, after deterioration has progressed, unreliable indicators of durability performance of concrete in service. In situ tests provide useful information about concrete properties but are often insensitive to changes in durability performance. The most appropriate method of monitoring the durability performance of concrete in the medium and long-term is to measure those deterioration mechanisms which affect the durability of concrete. For marine concrete structures, chloride ingress is generally regarded as being the most important parameter affecting durability and reliable measurements are necessary from field samples in order to predict rates of chloride ingress into concrete.

4.5.4 Predictions of Durability

To predict the rate of chloride ingress concrete must be characterized in terms of the potential chloride resistance of the material. The most appropriate test for this purpose is the chloride conductivity test but unfortunately the test does not fully characterize the long-term properties of the material. Modification of the chloride conductivity test with a factor which takes into account the long-term effects of chloride binding and cement hydration produced a reasonable correlation with diffusion coefficients determined after two years marine exposure. Predictions based on this relationship show promise and further development of this approach and its inherent reliability is discussed in Chapter Seven.

Of the methods investigated, the most reliable indicator of potential durability was chloride contents of the concrete. Chloride sampling at three time intervals of 12, 18 and 24 months provided information about medium-term trends but more reliable data needs to be generated by testing at later dates. Long-term trends with regard to the durability performance of marine concrete could not be determined given the time constraints and further work is needed to confirm the exposure results recorded after two years. Chloride testing should be done after perhaps three, five and ten years which would provide valuable information about durability performance of concrete. Case studies of marine concrete structures presented in Chapter Five were used to complement these findings from marine exposure samples and provide information about long-term trends.

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4.7 Appendices

SITE / ZONE	CONC. TYPE	INITIAL CURING	CONCRETE RESISTIVITY AT AGE (days)						
			28	91	182	274	365	547	730
LAB. CONT-ROLS	OPC	WET	5	6	6	6	6	6	7
		MOIST	14	9	10	12	12	13	13
		DRY	14	10	13	16	10	13	12
	FLY ASH	WET	8	18	45	73	91	107	98
		MOIST	10	14	107	92	106	145	108
		DRY	9	13	121	89	121	151	117
	SLAG	WET	17	33	41	62	82	97	72
		MOIST	19	37	147	74	104	119	87
		DRY	18	40	155	50	121	140	121
SIM. TIDAL	OPC	MOIST	8	5	6	4	9	8	9
		DRY	7	5	9	9	10	7	10
	FLY ASH	MOIST	8	16	24	22	28	69	71
		DRY	9	12	21	24	35	80	100
	SLAG	MOIST	14	16	29	22	20	46	60
		DRY	18	8	10	16	30	72	72
SIM SPRAY	OPC	MOIST	8	9	6	12	8	14	12
		DRY	6	8	7	20	9	15	12
	FLY ASH	MOIST	9	32	62	142	76	177	187
		DRY	8	28	63	158	67	191	181
	SLAG	MOIST	15	45	76	87	60	180	184
		DRY	18	47	70	115	51	157	141
GR. BAY TIDAL	OPC	MOIST	7	5	7	6	9	6	8
		DRY	7	4	6	5	8	5	6
	FLY ASH	MOIST	9	14	42	75	66	46	61
		DRY	8	14	40	65	119	74	52
	SLAG	MOIST	16	28	67	75	78	40	87
		DRY	15	26	53	57	97	48	62
GR. BAY SPRAY	OPC	MOIST	9	5	10	8	10	10	12
		DRY	9	5	10	9	7	15	15
	FLY ASH	MOIST	9	17	50	60	59	76	83
		DRY	9	23	61	89	101	90	101
	SLAG	MOIST	12	34	36	65	86	74	78
		DRY	12	31	22	46	68	58	74

Table 4A1: Concrete Resistivity Results (kOhmcm)

SITE / ZONE	CONC. TYPE	INITIAL CURING	ULTRASONIC PULSE VELOCITY AT AGE (days)						
			28	91	182	274	365	547	730
LAB. CONT-ROLS	OPC	WET	4579	4713	4640	4867	4703	4765	4769
		MOIST	4587	4710	4667	4799	4660	4728	4732
		DRY	4525	4557	4624	4741	4658	4710	4733
	FLY ASH	WET	4564	4834	4834	5023	4865	4941	4840
		MOIST	4647	4732	4792	4977	4818	4860	4930
		DRY	4461	4658	4797	4992	4807	4845	4900
	SLAG	WET	4498	4757	4875	4886	4881	4913	4908
		MOIST	4493	4718	4908	4954	4924	4925	4885
		DRY	4284	4489	4716	4886	4706	4773	4785
SIM. TIDAL	OPC	MOIST	4479	4649	4733	4703	4648	4683	4733
		DRY	4470	4638	4713	4695	4663	4650	4771
	FLY ASH	MOIST	4659	4823	4913	4902	4837	4852	5016
		DRY	4479	4724	4839	4889	4945	4884	4961
	SLAG	MOIST	4312	4641	4662	4673	4690	4755	4806
		DRY	4257	4747	4815	4755	4671	4815	4822
SIM SPRAY	OPC	MOIST	4505	4683	4771	4669	4713	4756	4650
		DRY	4356	4606	4676	4641	4641	4704	4627
	FLY ASH	MOIST	4482	4837	4845	4913	4860	4937	4860
		DRY	4465	4823	4884	4953	4914	4930	4937
	SLAG	MOIST	4294	4650	4702	4720	4809	4773	4758
		DRY	4313	4599	4758	4711	4744	4773	4723
GR. BAY TIDAL	OPC	MOIST	4550	4689	4712	4737	4719	4728	4709
		DRY	4399	4582	4652	4693	4734	4740	4724
	FLY ASH	MOIST	4651	4809	4865	4899	4868	4877	4789
		DRY	4490	4805	4827	4870	4874	4903	4879
	SLAG	MOIST	4583	4762	4812	4827	4847	4866	4763
		DRY	4223	4625	4658	4680	4632	4620	4717
GR. BAY SPRAY	OPC	MOIST	4578	4589	4625	4625	4750	4757	4630
		DRY	4380	4432	4617	4596	4636	4581	4629
	FLY ASH	MOIST	4616	4647	4785	4822	4772	4787	4742
		DRY	4483	4595	4730	4737	4693	4658	4665
	SLAG	MOIST	4666	4767	4860	4890	4775	4745	4635
		DRY	4204	4380	4508	4541	4458	4464	4451

Table 4A2 : Ultrasonic Pulse Velocity Results (m/s)

CONCRETE TYPE	CONCRETE GRADE	INITIAL CURING	SIMONSTOWN TIDAL	SIMONSTOWN SPRAY	GRANGER BAY TIDAL	GRANGER BAY SPRAY
OPC	20					
		MOIST	MILD	LOW		
		DRY	MILD	LOW		
	40					
		MOIST	LOW	LOW	HIGH	MODERATE
		DRY	LOW	LOW	SEVERE	HIGH
	60					
		MOIST			MODERATE	MILD
		DRY			MODERATE	MILD
FLY ASH	20					
		MOIST	MILD	MILD		
		DRY	MILD	LOW		
	40					
		MOIST	LOW	LOW	SEVERE	HIGH
		DRY	LOW	LOW	HIGH	HIGH
	60					
		MOIST			MODERATE	MODERATE
		DRY			MILD	MILD
SLAG	20					
		MOIST	MODERATE	MILD		
		DRY	MODERATE	MILD		
	40					
		MOIST	MILD	LOW	EXTREME	HIGH
		DRY	LOW	LOW	SEVERE	HIGH
	60					
		MOIST			HIGH	MODERATE
		DRY			HIGH	MODERATE

KEY

CATEGORY	DESCRIPTION OF CONCRETE SURFACE
LOW	Good condition with no evidence of damage
MILD	Minor weathering of surface
MODERATE	Major weathering exposing fine aggregate
HIGH	Abrasion damage exposing coarse aggregate
SEVERE	Abrasion damage with aggregate protuding
EXTREME	Abrasion damage with rounded edges and removal of aggregate

Table 4A3: Visual Appearance of Marine Concrete After Two Years

CONC. TYPE	GRADE (MPa)	CURING REGIME	LAB. CONTROLS		SIMONSTOWN		GRANGER BAY	
			28 DAYS	12 MON.	TIDAL	SPRAY	TIDAL	SPRAY
OPC	20	WET	18.8	24.0				
		MOIST	23.1	25.8	34.6	32.1		
		DRY	19.7	25.6	28.9	24.8		
	40	WET	42.4	50.6				
		MOIST	39.5	48.7	52.3	49.8	56.0	55.4
		DRY	41.0	50.4	46.6	42.3	42.8	44.7
	60	WET	56.2	75.3				
		MOIST	55.0	71.3			81.2	67.9
		DRY	55.7	75.2			75.3	70.7
FLY ASH	20	WET	20.8	38.1				
		MOIST	20.2	34.9	35.0	42.1		
		DRY	18.8	30.6	31.5	29.2		
	40	WET	35.0	64.9				
		MOIST	35.2	55.6	74.9	63.8	64.0	68.3
		DRY	30.5	57.0	53.3	44.2	53.8	58.6
	60	WET	51.6	89.3				
		MOIST	50.1	73.6			81.1	80.8
		DRY	42.5	67.1			73.9	72.6
SLAG	20	WET	21.0	31.0				
		MOIST	22.7	37.7	36.1	35.3		
		DRY	18.2	27.3	26.9	29.9		
	40	WET	40.5	52.3				
		MOIST	35.7	51.7	59.5	48.3	60.6	53.8
		DRY	34.4	46.4	51.1	42.7	42.4	43.7
	60	WET	57.7	87.2				
		MOIST	53.9	80.1			66.2	63.6
		DRY	49.1	62.9			63.3	62.4

Table 4A4: Core Strength Results After Twelve Months (MPa)

CONC. TYPE	GRADE (MPa)	CURING REGIME	LAB. CONTROLS		SIMONSTOWN		GRANGER BAY	
			28 DAYS	12 MON.	TIDAL	SPRAY	TIDAL	SPRAY
OPC								
	20	WET	8.95	9.56				
		MOIST	9.07	9.42	9.61	9.06		
		DRY	8.77	9.21	9.05	8.64		
	40	WET	10.06	10.47				
		MOIST	9.73	9.86	9.94	9.64	9.90	9.50
		DRY	9.37	9.76	10.29	9.15	9.08	9.54
	60	WET	10.20	10.93				
		MOIST	10.05	10.29			10.33	10.09
		DRY	9.59	9.81			9.91	10.00
FLY ASH								
	20	WET	8.92	9.89				
		MOIST	8.60	9.19	9.27	8.50		
		DRY	8.37	9.01	8.99	8.23		
	40	WET	9.84	11.21				
		MOIST	9.83	9.81	10.39	9.52	10.17	10.13
		DRY	9.22	9.81	10.27	8.92	9.87	9.82
	60	WET	10.31	11.70				
		MOIST	10.24	10.15			10.36	9.68
		DRY	9.78	10.39			10.27	10.29
SLAG								
	20	WET	8.88	9.99				
		MOIST	8.94	9.34	8.89	7.93		
		DRY	8.32	9.14	9.10	7.85		
	40	WET	9.90	10.13				
		MOIST	9.73	9.65	9.49	9.08	10.04	9.88
		DRY	8.83	9.41	9.80	9.06	8.87	9.01
	60	WET	10.39	11.07				
		MOIST	10.19	10.08			9.53	9.49
		DRY	9.94	9.78			9.65	9.60

Table 4A5: Oxygen Permeability Index Results After Twelve Months

CONC. TYPE	GRADE (MPa)	CURING REGIME	LAB. CONTROLS		SIMONSTOWN		GRANGER BAY	
			28 DAYS	12 MON.	TIDAL	SPRAY	TIDAL	SPRAY
OPC	20	WET	13.98	8.82				
		MOIST	13.98	9.42	8.25	9.29		
		DRY	14.38	9.21	6.00	10.03		
	40	WET	12.37	6.16				
		MOIST	13.58	6.03	4.55	9.82	6.75	6.38
		DRY	16.13	4.66	4.33	6.89	7.01	8.50
	60	WET	10.86	7.89				
		MOIST	9.20	7.70			6.48	7.33
		DRY	12.83	5.14			7.45	7.24
FLY ASH	20	WET	16.88	6.24				
		MOIST	20.35	6.69	5.92	7.14		
		DRY	27.27	7.02	8.95	6.58		
	40	WET	10.85	4.11				
		MOIST	12.13	4.44	2.70	5.43	5.64	5.35
		DRY	17.37	4.78	3.11	4.96	6.88	4.97
	60	WET	9.04	5.18				
		MOIST	9.66	3.57			5.19	5.73
		DRY	11.99	3.38			4.92	4.60
SLAG	20	WET	12.91	6.70				
		MOIST	13.64	7.37	5.38	4.43		
		DRY	22.22	5.85	8.25	3.97		
	40	WET	8.12	5.58				
		MOIST	9.15	3.22	2.64	4.23	4.61	3.99
		DRY	12.74	3.57	4.49	3.47	4.52	5.33
	60	WET	7.99	4.83				
		MOIST	7.22	3.52			4.83	5.41
		DRY	10.85	3.24			5.96	5.78

Table 4A6: Water Sorptivity Results After Twelve Months (mm/√hr)

CONC. TYPE	GRADE (MPa)	CURING REGIME	LAB. CONTROLS		SIMONSTOWN		GRANGER BAY	
			28 DAYS	12 MON.	TIDAL	SPRAY	TIDAL	SPRAY
OPC	20	WET	2.07	2.18				
		MOIST	3.68	3.12	1.82	2.97		
		DRY	4.07	2.89	1.58	3.39		
	40	WET	1.76	1.68				
		MOIST	2.14	1.93	0.88	1.66	1.47	1.41
		DRY	2.64	2.04	0.75	2.02	2.15	1.75
	60	WET	1.58	1.01				
		MOIST	1.70	1.13			1.00	1.13
		DRY	2.25	0.94			1.21	0.95
FLY ASH	20	WET	2.01	1.07				
		MOIST	2.51	1.59	1.09	1.28		
		DRY	4.69	1.94	1.52	1.97		
	40	WET	1.07	0.21				
		MOIST	1.59	0.70	0.44	0.53	0.59	0.59
		DRY	2.83	0.66	0.44	0.81	1.11	0.48
	60	WET	0.81	0.13				
		MOIST	1.05	0.34			0.13	0.28
		DRY	1.92	0.20			0.24	0.22
SLAG	20	WET	1.93	0.95				
		MOIST	2.23	1.36	1.20	1.33		
		DRY	4.43	1.88	1.84	1.98		
	40	WET	0.63	0.48				
		MOIST	0.96	0.32	0.46	0.57	0.52	0.47
		DRY	2.25	0.60	0.58	0.56	0.72	0.85
	60	WET	0.35	0.14				
		MOIST	0.52	0.18			0.30	0.33
		DRY	1.76	0.24			0.36	0.33

Table 4A7: Chloride Conductivity Results After Twelve Months (mS/cm)

CONC. TYPE	GRADE (MPa)	CURING REGIME	LAB. CONTROLS		SIMONSTOWN		GRANGER BAY	
			28 DAYS	12 MON.	TIDAL	SPRAY	TIDAL	SPRAY
OPC								
	20	WET	0.511	0.492				
		MOIST	0.504	0.319	0.445	0.543		
		DRY	0.684	0.327	0.366	0.489		
	40	WET	0.322	0.270				
		MOIST	0.374	0.209	0.212	0.240	0.222	0.200
		DRY	0.506	0.215	0.190	0.273	0.236	0.222
	60	WET	0.267	0.122				
		MOIST	0.284	0.110			0.180	0.145
		DRY	0.375	0.092			0.159	0.130
FLY ASH								
	20	WET	0.524	0.437				
		MOIST	0.644	0.440	0.432	0.457		
		DRY	0.799	0.473	0.411	0.587		
	40	WET	0.300	0.231				
		MOIST	0.416	0.199	0.223	0.248	0.103	0.171
		DRY	0.646	0.218	0.188	0.278	0.132	0.149
	60	WET	0.239	0.143				
		MOIST	0.312	0.133			0.126	0.125
		DRY	0.443	0.109			0.092	0.099
SLAG								
	20	WET	0.623	0.449				
		MOIST	0.758	0.476	0.410	0.591		
		DRY	1.043	0.559	0.402	0.636		
	40	WET	0.368	0.239				
		MOIST	0.554	0.222	0.277	0.352	0.183	0.160
		DRY	0.666	0.341	0.238	0.395	0.105	0.191
	60	WET	0.268	0.147				
		MOIST	0.344	0.115			0.103	0.151
		DRY	0.483	0.132			0.114	0.141

Table 4A8: Abrasion Resistance Coefficient After Twelve Months

CONC. TYPE	GRADE (MPa)	CURING REGIME	LAB. CONTROLS		SIMONSTOWN		GRANGER BAY	
			28 DAYS	12 MON.	TIDAL	SPRAY	TIDAL	SPRAY
OPC								
	20	WET	0.0	0.5				
		MOIST	0.5	7.5	1.5	7.0		
		DRY	1.0	8.5	2.5	9.5		
	40	WET	0.0	0.5				
		MOIST	0.5	4.5	1.5	3.0	0.0	1.0
		DRY	0.5	5.5	2.0	3.5	1.0	2.0
	60	WET	0.0	0.5				
		MOIST	0.5	1.0			0.5	1.0
		DRY	0.5	2.0			1.5	1.0
FLY ASH								
	20	WET	0.0	0.5				
		MOIST	0.5	10.5	1.5	8.5		
		DRY	1.0	11.5	2.5	11.5		
	40	WET	0.0	0.5				
		MOIST	0.5	4.5	2.0	4.5	1.5	2.0
		DRY	0.5	5.5	2.0	4.0	1.5	3.0
	60	WET	0.0	0.5				
		MOIST	0.5	1.5			1.0	1.0
		DRY	0.5	2.5			1.0	1.5
SLAG								
	20	WET	0.0	1.0				
		MOIST	1.0	11.5	2.0	10.0		
		DRY	1.0	14.0	3.0	12.5		
	40	WET	0.0	1.0				
		MOIST	0.5	4.0	1.5	5.5	1.0	2.0
		DRY	1.0	9.0	2.0	10.5	1.5	2.0
	60	WET	0.0	1.0				
		MOIST	0.5	2.0			1.0	1.0
		DRY	0.5	3.0			1.0	1.0

Table 4A9: Carbonation Depth After Twelve Months (mm)

CONCRETE TYPE	AGE (months)	% CHLORIDE AT DEPTH (mm)				Cs (%)	Dc (cm^2/s)
		5 - 25	25 - 50	50 - 75	75 - 100		
GRADE 20 OPC MOIST	12	2.49	0.83	0.40	0.21	3.79	1.47E-07
	18	1.31	0.69	0.30	0.22	1.75	2.95E-07
	24	2.11	0.84	0.44	0.16	3.02	9.88E-08
GRADE 20 OPC DRY	12	2.37	1.21	0.73	0.41	3.01	3.74E-07
	18	1.59	0.78	0.60	0.24	1.98	3.99E-07
	24	2.68	1.25	0.68	0.53	3.45	1.63E-07
GRADE 40 OPC MOIST	12	1.02	0.23	0.08	0.07	1.93	7.07E-08
	18	1.03	0.21	0.06	0.05	2.05	6.23E-08
	24	1.89	0.68	0.12	0.04	3.08	6.06E-08
GRADE 40 OPC DRY	12	1.00	0.32	0.18	0.16	1.54	1.36E-07
	18	0.92	0.24	0.05	0.04	1.70	7.69E-08
	24	1.33	0.41	0.14	0.04	2.22	5.35E-08
GRADE 20 FLY ASH MOIST	12	1.98	0.25	0.15	0.08	4.45	4.79E-08
	18	1.98	0.29	0.07	0.04	4.28	5.15E-08
	24	2.38	0.42	0.06	0.05	4.82	3.06E-08
GRADE 20 FLY ASH DRY	12	1.53	0.30	0.09	0.08	3.02	6.46E-08
	18	1.49	0.33	0.06	0.05	2.76	7.52E-08
	24	2.56	0.38	0.12	0.05	5.51	2.68E-08
GRADE 40 FLY ASH MOIST	12	1.11	0.05	0.04	0.04	3.50	2.58E-08
	18	0.89	0.04	0.04	0.03	3.13	2.20E-08
	24	1.29	0.07	0.04	0.03	3.82	1.43E-08
GRADE 40 FLY ASH DRY	12	1.05	0.17	0.03	0.04	2.27	5.15E-08
	18	1.06	0.05	0.03	0.03	3.52	2.41E-08
	24	0.96	0.05	0.03	0.03	3.10	1.24E-08
GRADE 20 SLAG MOIST	12	2.53	0.16	0.10	0.04	6.72	3.47E-08
	18	2.65	0.26	0.09	0.08	6.24	4.33E-08
	24	3.18	0.17	0.12	0.09	6.25	2.81E-08
GRADE 20 SLAG DRY	12	1.97	0.22	0.09	0.06	4.61	4.39E-08
	18	1.67	0.21	0.11	0.05	3.80	4.65E-08
	24	2.37	0.18	0.10	0.06	6.05	1.85E-08
GRADE 40 SLAG MOIST	12	0.76	0.06	0.02	0.01	2.30	2.70E-08
	18	1.10	0.06	0.04	0.04	3.55	2.49E-08
	24	0.77	0.11	0.09	0.08	1.76	2.22E-08
GRADE 40 SLAG DRY	12	0.85	0.04	0.03	0.02	3.01	2.19E-08
	18	1.08	0.05	0.04	0.03	3.53	2.45E-08
	24	1.29	0.12	0.06	0.05	3.25	1.87E-08

Table 4A10a: Chloride Content by Weight of Cement - Simonstown Tidal Zone (%)

CONCRETE TYPE	AGE (months)	% CHLORIDE AT DEPTH (mm)				Cs (%)	Dc (cm^2/s)
		5 - 25	25 - 50	50 - 75	75 - 100		
GRADE 20	12	0.31	0.09	0.06	0.05	0.57	6.83E-08
OPC	18						
MOIST	24	0.20	0.06	0.02	0.03	0.40	2.47E-08
GRADE 20	12	0.31	0.09	0.06	0.05	0.57	6.83E-08
OPC	18						
DRY	24	0.21	0.07	0.03	0.04	0.39	3.06E-08
GRADE 40	12	0.13	0.04	0.04	0.04	0.50	1.52E-08
OPC	18						
MOIST	24	0.12	0.04	0.03	0.03	0.48	7.07E-09
GRADE 40	12	0.19	0.04	0.04	0.03	0.59	2.12E-08
OPC	18						
DRY	24	0.09	0.03	0.02	0.02	0.42	6.83E-09
GRADE 20	12	0.36	0.08	0.04	0.04	0.75	5.05E-08
FLY ASH	18						
MOIST	24	0.41	0.04	0.03	0.04	1.15	1.47E-08
GRADE 20	12	0.46	0.08	0.04	0.04	1.02	4.52E-08
FLY ASH	18						
DRY	24	0.29	0.09	0.04	0.03	0.51	4.04E-08
GRADE 40	12	0.12	0.05	0.04	0.04	0.20	6.06E-08
FLY ASH	18						
MOIST	24	0.07	0.03	0.03	0.03	0.19	9.15E-09
GRADE 40	12	0.16	0.05	0.04	0.04	0.34	3.98E-08
FLY ASH	18						
DRY	24	0.11	0.03	0.03	0.03	0.52	6.55E-09
GRADE 20	12	0.35	0.12	0.05	0.04	0.59	9.09E-08
SLAG	18						
MOIST	24	0.18	0.09	0.09	0.06	0.22	1.60E-07
GRADE 20	12	0.66	0.14	0.05	0.04	1.31	6.15E-08
SLAG	18						
DRY	24	0.34	0.09	0.08	0.06	0.63	3.33E-08
GRADE 40	12	0.19	0.05	0.04	0.04	0.55	2.42E-08
SLAG	18						
MOIST	24	0.12	0.08	0.07	0.05	0.15	1.72E-07
GRADE 40	12	0.27	0.10	0.04	0.05	0.46	8.59E-08
SLAG	18						
DRY	24	0.22	0.08	0.05	0.04	0.36	4.33E-08

Table 4A10b: Chloride Content by Weight of Cement - Simonstown Spray Zone (%)

CONCRETE TYPE	AGE (months)	% CHLORIDE AT DEPTH (mm)				Cs (%)	Dc (cm^2/s)
		5 - 25	25 - 50	50 - 75	75 - 100		
GRADE 40 OPC MOIST	12	2.09	0.42	0.23	0.07	3.98	7.13E-08
	18	2.39	0.78	0.18	0.18	3.92	7.75E-08
	24	2.59	0.93	0.25	0.06	4.11	6.63E-08
GRADE 40 OPC DRY	12	1.79	0.69	0.26	0.12	2.69	1.60E-07
	18	2.49	1.76	1.18	0.49	3.08	4.35E-07
	24	2.30	1.39	0.48	0.25	3.16	1.49E-07
GRADE 60 OPC MOIST	12	1.68	0.17	0.11	0.09	4.04	4.12E-08
	18	1.12	0.38	0.16	0.12	1.74	8.66E-08
	24	1.60	0.26	0.07	0.04	3.66	2.78E-08
GRADE 60 OPC DRY	12	1.36	0.15	0.08	0.05	3.23	4.22E-08
	18	0.94	0.39	0.26	0.11	1.25	1.64E-07
	24	1.47	0.17	0.06	0.06	3.42	2.19E-08
GRADE 40 FLY ASH MOIST	12	0.90	0.19	0.08	0.05	1.76	6.46E-08
	18	2.29	0.30	0.07	0.06	4.75	3.71E-08
	24	1.94	0.09	0.04	0.04	5.65	1.48E-08
GRADE 40 FLY ASH DRY	12	1.66	0.11	0.05	0.05	4.45	3.37E-08
	18	1.14	0.22	0.07	0.06	2.31	4.02E-08
	24	2.60	0.16	0.16	0.08	5.95	2.20E-08
GRADE 60 FLY ASH MOIST	12	1.35	0.11	0.11	0.08	3.52	3.54E-08
	18	0.78	0.11	0.05	0.04	1.80	2.85E-08
	24	1.08	0.05	0.04	0.04	3.40	1.29E-08
GRADE 60 FLY ASH DRY	12	1.49	0.06	0.07	0.07	4.75	2.55E-08
	18	0.66	0.12	0.06	0.04	1.38	3.54E-08
	24	0.99	0.04	0.03	0.03	3.20	1.25E-08
GRADE 40 SLAG MOIST	12	0.98	0.18	0.06	0.05	2.02	5.71E-08
	18	1.29	0.31	0.13	0.12	2.35	5.35E-08
	24	1.28	0.10	0.04	0.03	3.29	1.82E-08
GRADE 40 SLAG DRY	12	0.66	0.14	0.04	0.04	1.30	6.18E-08
	18	0.88	0.13	0.10	0.08	1.98	3.10E-08
	24	1.58	0.12	0.09	0.05	4.05	1.85E-08
GRADE 60 SLAG MOIST	12	0.75	0.06	0.05	0.05	2.12	2.99E-08
	18	0.65	0.11	0.03	0.04	1.35	3.54E-08
	24	0.81	0.04	0.05	0.04	2.48	1.35E-08
GRADE 60 SLAG DRY	12	0.84	0.06	0.05	0.05	2.40	2.95E-08
	18	0.96	0.17	0.08	0.04	1.98	3.76E-08
	24	0.99	0.04	0.03	0.03	3.20	1.25E-08

Table 4A10c: Chloride Content by Weight of Cement - Granger Bay Tidal Zone (%)

CONCRETE TYPE	AGE (months)	% CHLORIDE AT DEPTH (mm)				Cs (%)	Dc (cm²/s)
		5 - 25	25 - 50	50 - 75	75 - 100		
GRADE 40 OPC MOIST	12	2.38	0.43	0.18	0.10	4.78	6.26E-08
	18						
	24	1.78	0.69	0.24	0.10	2.69	7.95E-08
GRADE 40 OPC DRY	12	1.36	0.45	0.13	0.09	2.22	1.16E-07
	18						
	24	1.47	0.58	0.13	0.08	2.24	7.45E-08
GRADE 60 OPC MOIST	12	1.08	0.08	0.05	0.04	2.95	3.23E-08
	18						
	24	0.94	0.19	0.03	0.03	1.85	3.21E-08
GRADE 60 OPC DRY	12	0.66	0.10	0.05	0.04	1.49	4.49E-08
	18						
	24	1.00	0.21	0.03	0.03	1.96	3.30E-08
GRADE 40 FLY ASH MOIST	12	1.86	0.10	0.05	0.04	5.26	3.09E-08
	18						
	24	2.72	0.04	0.03	0.03	6.55	1.99E-08
GRADE 40 FLY ASH DRY	12	1.71	0.16	0.06	0.04	4.18	4.02E-08
	18						
	24	2.66	0.18	0.05	0.03	6.25	2.14E-08
GRADE 60 FLY ASH MOIST	12	0.55	0.04	0.04	0.04	1.75	2.45E-08
	18						
	24	1.07	0.03	0.04	0.03	3.27	1.37E-08
GRADE 60 FLY ASH DRY	12	0.48	0.04	0.04	0.04	1.55	2.37E-08
	18						
	24	0.88	0.03	0.03	0.03	2.65	1.38E-08
GRADE 40 SLAG MOIST	12	1.18	0.11	0.04	0.04	2.96	3.77E-08
	18						
	24	1.48	0.07	0.06	0.06	4.28	1.49E-08
GRADE 40 SLAG DRY	12	1.66	0.09	0.04	0.04	4.61	3.19E-08
	18						
	24	1.68	0.13	0.05	0.03	4.25	1.89E-08
GRADE 60 SLAG MOIST	12	0.43	0.04	0.04	0.04	1.45	2.22E-08
	18						
	24	0.75	0.04	0.05	0.03	2.48	1.21E-08
GRADE 60 SLAG DRY	12	0.61	0.04	0.04	0.04	1.98	2.40E-08
	18						
	24	0.63	0.05	0.06	0.04	1.78	1.52E-08

Table 4A10d: Chloride Content by Weight of Cement - Granger Bay Spray Zone (%)

SAMPLE No.	LOCATION OF SAMPLE	EXPOSURE DIRECTION	HEIGHT ABOVE MSL	DISTANCE FROM SEA	EXPOSURE CATEGORY
P1	Muizenberg	South	5 m	50 m	Severe
P2	Muizenberg	South	3 m	10 m	Very severe
P3	Clovelly	South East	5 m	40 m	Severe
P4	Fish Hoek	East	4 m	15 m	Very severe
P5	Fish Hoek	East	10 m	25 m	Severe
P6	Simonstown	North West	3 m	10 m	Moderate
P7	Simonstown	North	3 m	5 m	Moderate
P8	Hout Bay	South East	5 m	60 m	Moderate
P9	Oudekraal	West	3 m	3 m	Very severe
P10	Oudekraal	West	5 m	15 m	Severe
P11	Granger Bay	North West	5 m	5 m	Very severe
P12	Granger Bay	North	3 m	10 m	Very severe
P13	Foreshore	North West	10 m	300 m	Moderate
P14	UCT	North	120 m	6000 m	Mild
P15	UCT	South	120 m	18000 m	Mild
P16	Control	Indoors	-	-	Mild

Table 4A11: Location of Marine Spray Zone Panels

SAMPLE NUMBER	DEPTH INTO CONCRETE (mm)					DIFFUSION COEFF.	SURF. Cl Cs
	0 - 5	5 - 10	10 - 15	15 - 20	20 - 25		
P1	3.64	2.62	2.33	1.43	0.66	4.92E-08	4.10
P2	4.14	3.80	3.17	1.92	1.09	7.04E-08	4.90
P3	3.02	2.55	1.47	0.83	0.32	3.11E-08	3.70
P4	3.10	3.93	2.51	1.57	0.80	7.27E-08	4.10
P5	-	-	-	-	-	-	-
P6	0.18	0.10	0.08	0.07	0.07	1.06E-08	0.24
P7	0.08	0.08	0.05	0.04	0.04	1.49E-08	0.10
P8	0.74	1.00	0.77	0.67	0.51	2.53E-07	1.00
P9	2.84	3.71	2.92	2.65	2.25	5.96E-07	3.50
P10	2.25	1.53	1.24	0.65	0.44	3.71E-08	2.55
P11	2.98	2.63	1.60	1.24	1.19	7.09E-08	3.30
P12	2.62	2.41	2.09	1.85	1.37	2.06E-07	2.85
P13	0.20	0.19	0.09	0.08	0.07	2.65E-08	0.25
P14	0.51	0.43	0.26	0.14	0.08	2.58E-08	0.65
P15	0.17	0.08	0.07	0.05	0.04	8.67E-09	0.20
P16	0.05	0.04	0.04	0.04	0.04	-	-

Table 4A12: Chloride content by Weight of Cement - Marine Spray Zone Panels

	MATHEMATICAL ERROR FUNCTION (erf x)									
x	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0113	0.0226	0.0338	0.4510	0.0564	0.0676	0.0789	0.0901	0.1013
0.1	0.1125	0.1236	0.1348	0.1459	0.1569	0.1680	0.1790	0.1900	0.2009	0.2118
0.2	0.2227	0.2335	0.2443	0.2550	0.2657	0.2763	0.2869	0.2974	0.3079	0.3183
0.3	0.3286	0.3389	0.3491	0.3593	0.3694	0.3794	0.3893	0.3992	0.4090	0.4187
0.4	0.4284	0.4380	0.4475	0.4569	0.4662	0.4755	0.4847	0.4937	0.5027	0.5117
0.5	0.5205	0.5292	0.5379	0.5465	0.5549	0.5633	0.5716	0.5798	0.5879	0.5959
0.6	0.6039	0.6117	0.6194	0.6270	0.6346	0.6420	0.6494	0.6566	0.6638	0.6708
0.7	0.6778	0.6846	0.6914	0.6981	0.7047	0.7112	0.7175	0.7282	0.7300	0.7361
0.8	0.7421	0.7480	0.7538	0.7595	0.7651	0.7707	0.7761	0.7814	0.7867	0.7918
0.9	0.7969	0.8019	0.8068	0.8116	0.8163	0.8209	0.8254	0.8299	0.8342	0.8385
1.0	0.8427	0.8468	0.8508	0.8548	0.8586	0.8624	0.8661	0.8698	0.8733	0.8768
1.1	0.8802	0.8835	0.8868	0.8899	0.8931	0.8961	0.8991	0.9020	0.9048	0.9076
1.2	0.9103	0.9130	0.9155	0.9181	0.9205	0.9229	0.9252	0.9275	0.9297	0.9319
1.3	0.9340	0.9361	0.9381	0.9400	0.9419	0.9438	0.9456	0.9473	0.9490	0.9507
1.4	0.9523	0.9539	0.9554	0.9569	0.9583	0.9597	0.9611	0.9624	0.9637	0.9649
1.5	0.9661	0.9673	0.9684	0.9695	0.9706	0.9716	0.9726	0.9736	0.9745	0.9755
1.6	0.9763	0.9770	0.9780	0.9788	0.9796	0.9804	0.9811	0.9818	0.9825	0.9832
1.7	0.9838	0.9844	0.9850	0.9856	0.9861	0.9867	0.9872	0.9877	0.9882	0.9886
1.8	0.9891	0.9895	0.9899	0.9903	0.9907	0.9911	0.9915	0.9918	0.9922	0.9925
1.9	0.9928	0.9931	0.9934	0.9937	0.9939	0.9942	0.9944	0.9947	0.9949	0.9951
2.0	0.9953	0.9955	0.9957	0.9959	0.9961	0.9963	0.9965	0.9967	0.9969	0.9970
2.2	0.9981	0.9982	0.9983	0.9984	0.9985	0.9986	0.9986	0.9987	0.9987	0.9988
2.4	0.9993	0.9993	0.9994	0.9994	0.9994	0.9995	0.9995	0.9995	0.9996	0.9996
2.6	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9999	0.9998
2.8	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999
3.0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

Table 4A13: Mathematical Error Function (erf x)

CHAPTER FIVE

CASE STUDIES OF MARINE CONCRETE STRUCTURES

5.1 Literature Survey

Concrete structures have been used for marine applications for a considerable length of time with varying degrees of effectiveness and durability. Marine structures have a wide range of properties resulting from construction and material effects and are exposed to different environmental and service conditions. Variations in materials and environment are responsible for the wide range of durability performance. The rapid deterioration of many reinforced concrete structures in the marine environment created an awareness of the severity of marine exposure and case studies of failures were undertaken to establish the reasons for the deterioration and provide solutions to the problem. Simple solutions have not been forthcoming from these investigations but a better understanding has been achieved about the complex interactions between materials, environment and loadings which cause damage.

Durability studies of concrete structures must ultimately relate to the behaviour of structures in service yet most research does not consider site conditions at all. There are several reasons for favouring a more analytical laboratory approach; site concrete is exposed to a multitude of influences which are difficult to quantify, design and construction data is often unavailable or unreliable and assessing the durability performance of a structure is relatively subjective. Despite these practical limitations, useful information can be obtained about the long-term behaviour of concrete under service conditions. Long-term information regarding durability performance is essential for confirming trends established from early-age laboratory and medium-term field exposure tests. The current emphasis of early-age durability testing must be tempered with long-term studies of concrete to ensure that assumptions made in extrapolations of potential durability performance are valid.

Interpretation of findings from case-studies of marine concrete structures must be done in the light of the limitations inherent to these investigations, particularly with respect to the reliability of construction data, changing exposure conditions and definition of durability performance. Original construction properties may be inferred from diagnostic techniques but factors such as curing, surface defects and contaminants may not be adequately quantified. Measuring the severity of exposure may be done using local records or site monitoring but conditions may change due to local effects from surrounding structures or altered service conditions. The durability performance of concrete may be determined relatively easily at serviceability failure but at earlier ages the durability performance must be inferred from deterioration mechanisms such as chloride ingress or carbonation. Effective case studies must ensure that sufficient data is available with regard to these factors since older structures may provide valuable information about deterioration but may not have sufficient construction data while the opposite may be true of modern structures.

Early researchers such as Vicat in the nineteenth century investigated the chemical attack of seawater on concrete as this was the most prevalent form of deterioration of marine concrete structures at the time [1]. The mechanism of seawater attack was a combination of sulphate and magnesium ions reacting with the surface layers of concrete causing gradual softening and dissolution. With the advent of reinforced concrete, steel corrosion rapidly became the most common form of deterioration, particularly in warm climates and slender structures with low cover to reinforcement [2]. Steel corrosion has become much more prevalent with the increased development in extreme environments such as the Middle East and the growing use of deicing salts on roads in cold regions. In colder climates other deterioration processes such as freeze thaw action, wave action and chemical attack were also major factors in lack of durability of marine concrete structures.

An extensive survey in the 1980's of marine concrete structures in the North Sea showed moderate to low levels of deterioration after more than twenty years exposure of predominantly slag concrete structures [3]. A comprehensive survey of reinforced concrete structures in Norwegian harbours revealed that the overall condition of structures was good even after sixty years service despite widespread evidence of steel corrosion [4]. In contrast an inspection of marine concrete structures in the Middle East showed that major corrosion damage was often evident within three to ten years of construction [5]. Whilst poor concrete and construction practices may have been a factor in these shorter service lives, the main reason was probably the severity of the environment with extremely hot and dry conditions. Warmer ambient temperatures may cause a coarser concrete pore structure and increase the rate of chemical processes such as chloride diffusion and steel corrosion causing rapid deterioration. The poor durability of reinforced concrete structures in the Middle East was convincing proof that marine environments are not necessarily similar and special precautions are required for durable construction in some environments. Table 5.1 shows the exposure conditions prevalent in four selected climatic areas and the consequences on durability of these environments.

Table 5.1: Climatic Conditions and Concrete Durability [6,7]

REGION (AREA)	AMBIENT CLIMATIC CONDITIONS			EXPOSURE CHARACTERISTICS	
	Average Temperature (°C)	Average Humidity (%)	Average Rain (mm)	Relative Rate of Chloride Ingress	Relative Reinforcement Corrosion Rate
North Sea (Aberdeen)	8	79	895	Slow with limited wetting and drying action	Low due to cold conditions
Middle East (Jordan)	25	41	23	High with severe wetting and drying action	High due to hot conditions
S.E. Asia (Indonesia)	28	73	1638	Fairly high with limited drying due to rainfall	High due to hot conditions
South Africa (Cape Town)	17	75	650	High with severe summer drying	Moderate due to mild conditions

Recent research by Sandberg and Tang in Sweden showed that chloride penetration into concrete was highest in the submerged zone [8]. Field testing was done on a four year old concrete column on the West coast of Sweden located in the tidal zone. Predictions of chloride contents from one year old exposure samples were far higher than those found in practice on the four year old column which was probably due to chloride binding reducing the diffusivity of the concrete with time. Details of the exposure samples and the method used to predict chloride levels were not given but it is assumed that a simple extrapolation of Fick's Law of diffusion was done using a constant diffusion coefficient. Hendriksen and Stoltzner reported in 1993 that bridge columns in the marine environment in Denmark suffered corrosion of reinforcement after less than twenty five years of service [9]. Gjorv reported corrosion initiation times of as little as five years for Norwegian harbour structures [10]. Chlorides penetrated fastest in the lower tidal zone while corrosion damage was most prevalent above the high tide level on beam and slab edges and along soffits. Freeze-thaw action can cause major damage to marine concrete in the tidal zone and Moskvin rated this as the most severe form of deterioration of marine concrete structures in Russia [11].

In the United States several marine concrete structures have suffered severe deterioration which in some cases has led to failure [12]. Mehta, in a review of the durability performance of marine concrete structures, stated that the control of cracking is as important as the control of permeability to ensure durability of marine concrete structures [13]. The most common form of deterioration of concrete is chloride-induced corrosion from deicing salts on road bridges while marine concrete structures have received far less attention. The emphasis on the detrimental effects of deicing salts is not surprising considering that the estimated cost of repairing corrosion damaged highway bridges in the United States and Canada is tens of billions of dollars.

In hot climates the type and rate of deterioration mechanisms are quite different from those found in temperate regions. Liam et al investigated a jetty in Singapore which had widespread corrosion damage after twenty four years of service [14]. Chloride contents in the upper tidal and splash zones were found to be considerably higher than those in the submerged zone. Chloride contents by mass of cement at the steel were between 0.7 and 1.2 % and potential mapping indicated a high probability of corrosion. Several columns exhibited evidence of corrosion in the form of rust staining and cracking. The relatively poor durability performance of the structure was surprising considering that a grade 45 OPC concrete with cover to reinforcement of 70 mm was used.

Several case studies of marine concrete structures in the Middle East illustrate just how severe conditions are with poor durability performance being common [15,16,17]. Deterioration from reinforcement corrosion sometimes occured after less than ten years of exposure as was found in a desalination plant in Saudi Arabia [18]. The plant was built in 1982 using a grade 25 concrete with 50 mm cover to reinforcement and exposed to severe marine conditions. After ten years there was widespread evidence of chloride-induced corrosion damage due to poor cover control and inadequate concrete quality. Well advanced corrosion damage was also observed on concrete wharfs built in Qatar between 1975 and 1978 when investigated in 1993 [19]. A grade 35 concrete was used but resistance to chloride penetration was severely compromised by poor compaction and corrosion was exacerbated by low cover to reinforcement in places.

Marine concrete has shown poor durability performance in semi-tropical environments such as Sydney, Australia where concrete wharfs were found to have severe corrosion damage after less than twenty five years service [20]. Corrosion potentials below -500 mV at anodes were recorded with chloride content at the steel of between 0.9 and 5.8 % by mass of cement. Poor compaction and honeycombing of concrete were to blame for the rapid ingress of chlorides. Carbonation depths were relatively low at between 5 and 15 mm.

There are few documented case studies of the durability performance of marine concrete structures in South Africa. A limited amount of work has been published about the deterioration of road bridges in the marine environment. The road bridge across the Mpambinyoni River estuary in Scottburgh exhibited widespread corrosion damage after only sixteen years of service [21]. After forty five years service, investigations found high chloride contents at the reinforcement and moderate levels of carbonation which contributed to the corrosion of reinforcement. A number of road bridges next to the sea in Port Elizabeth had major corrosion damage after ten years service [22]. Superficial patch repairs were then done and when inspected nine years later were found to have failed with extensive cracking and spalling of columns and deck soffits. High chloride levels were found at the steel of between 0.70 and 1.50 % by mass of cement, carbonation depths were moderate with an average value of 14 mm. Low cover depths of as little as 15 mm were thought to be the cause of the poor durability performance although chloride profiles indicated that reinforcement at a depth of 50 mm would have been at risk after nineteen years.

The severe effect of the marine environment on reinforced concrete structures has been known for some time in South Africa. Halstead and Woodworth investigated the deterioration of reinforced concrete under coastal conditions in 1955 while Lewis proposed a chloride threshold level for corrosion of reinforced concrete in 1962 [23,24]. Several conferences on concrete deterioration in the last ten years served to heighten an awareness of the problem but little perceptible improvement in concrete durability has been observed [25,26]. A recent survey of marine concrete structures in the Western Cape revealed that many structures will require major repairs to achieve their original design lives [27]. Most structures showed an unacceptable level of deterioration which had compromised the serviceability of the structures after less than thirty years.

Only in the 1980's did engineers realise that special precautions were necessary to ensure durability of marine concrete structures, particularly in hot environments [17]. These severe conditions called for specialised concrete practices to prevent early age damage such as plastic cracking and to ensure concrete durability. Unfortunately a large number of structures were built before these new technologies were adopted by designers and contractors and many marine concrete structures are still being designed and built with little regard to durability. This lack of knowledge about concrete durability and continued indifference to improve the situation has resulted in a legacy of poor quality structures which will require vast amounts of money for essential maintenance and repairs in the future.

The current approach toward durability investigations appears to be based on measuring the extent of the deterioration by conducting a damage survey. The alternative approach of measuring the residual resistance of the concrete to environmental and service conditions does not receive much attention in the literature. The complexity of material, loading and environmental interactions makes reliable measurements of residual durability difficult which could explain the popularity of damage surveys. Measuring the residual resistance of concrete is also complicated by the lack of appropriate tests which can accurately measure the material resistance to deterioration processes such as chloride ingress. Interpretation of data from damage surveys and predictions of future durability must be done with caution and requires considerable experience.

5.2 Marine Concrete Structures

Twelve marine concrete structures were investigated in this work, ranging in age from three to seventy five years. All structures were located in or immediately adjacent to the sea to try and ensure that concrete was exposed to similar marine conditions and all structures were still in service when tested. The structures were mostly civil engineering structures such as bridges, walls, breakwaters and jetties which were not protected from direct marine exposure by claddings or surface coatings which would complicate analysis of results. To assist with comparisons between structures four categories of marine exposure were defined according to the location of concrete in the marine environment:

Submerged - Concrete permanently submerged (below low tide level)
Tidal - Concrete exposed to tidal action between low and high tide
Splash - Concrete above high tide level exposed to periodic splashing
Spray - Concrete exposed to wind-driven salt spray only

The condition of the structures investigated varied considerably with most of the older structures being in poor condition while modern structures were generally in fair to good condition with a few exceptions. Two breakwaters were investigated which were essentially mass concrete but were studied as it was believed that useful information could be gained about the behaviour of concrete in marine conditions, particularly with regard to ingress of chlorides. Details of the marine structures investigated in this work are given in Table 5.2.

Table 5.2: Marine Concrete Structures Investigated

SITE No.	NAME	AGE (years)	EXPOSURE CONDITION *	GENERAL CONDITION	CONCRETE GRADE
1	Kogel Bay Tidal Pool	3	Extreme - tidal exposure	Good with some minor abrasion	55 MPa
2	Table Bay Breakwater	6	Extreme - tidal exposure	Fair with some abrasion damage	50 MPa
3	Koeberg Power Station	12	Severe - spray exposure	Poor condition with corrosion damage	30 MPa
4	Strandfontein Tidal pool	13	Extreme - tidal exposure	Good with some abrasion damage	40 MPa
5	Camps Bay Pump Station	18	Severe - spray exposure	Good with minor surface weathering	45 MPa
6	Oudekraal Retaining Wall	19	Very severe - tidal exposure	Poor with localised corrosion damage	35 MPa
7	East London Breakwater	28	Extreme - tidal exposure	Poor with severe abrasion damage	25 MPa
8	Muizenberg Bridges	38	Very severe - spray exposure	Poor with widespread corrosion damage	30 MPa
9	Wilderness Bridges	46	Very severe - spray exposure	Poor with localised corrosion damage	30 MPa
10	Sea Point Toilet Block	48	Very severe splash exposure	Severe with extensive corrosion damage	25 MPa
11	Sea Point Aquarium	55	Very severe splash exposure	Extreme with major corrosion damage	25 MPa
12	Simonstown Jetty	75	Very severe - tidal exposure	Extreme with major corrosion damage	25 MPa

*** Exposure ratings for each site were determined from guidelines given in section 4.2.**

The location of the structures around the Cape Peninsula and along the South African coastline is shown in Figure 5.1. Environments and exposure conditions varied widely depending on the location of the structure. Sea temperatures on the Atlantic side of the Cape Peninsula were on average 5°C lower than temperatures in False Bay while sea temperatures at East London were on average several degrees warmer than those in False Bay. Climatically, the Cape Peninsula is similar to Mediterranean conditions with winter rainfall and hot, dry summers with strong South Easterly winds. The Southern Cape coast is exposed to mild conditions with rainfall throughout the year while East London generally has summer rainfall with mild and drier winters.

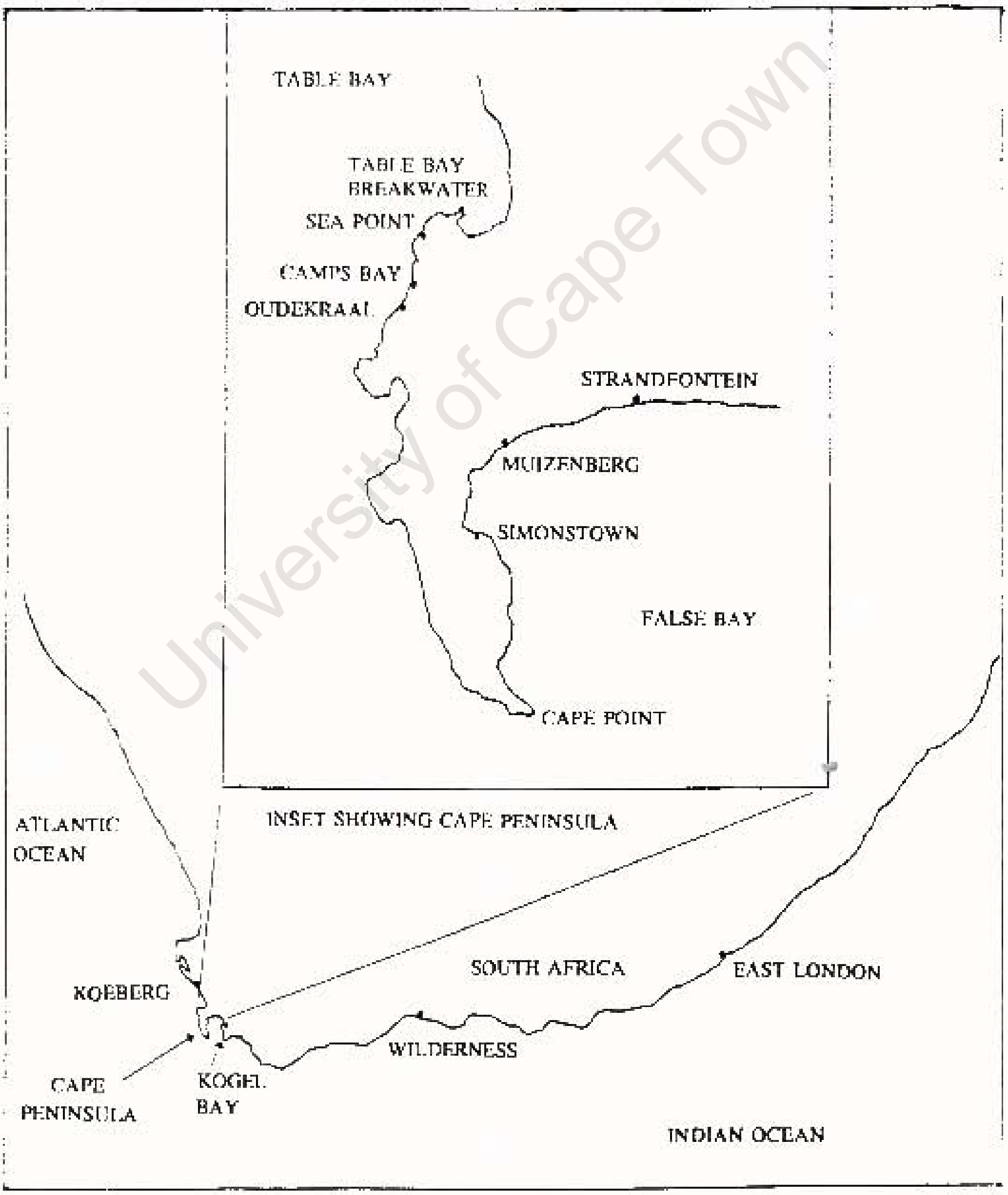

Figure 5.1: Location of Marine Structures Investigated

5.2.1 Kogel Bay Tidal Pool

The Kogel Bay tidal pool was built in 1990 on the East side of False Bay 6 km North of Rooiels. The area is exposed to severe wave action with little protection from the open sea. Construction of the tidal pool wall was done using precast concrete elements with an infill section being cast in situ. The concrete was in good condition with only minor abrasion damage after three years of exposure. The structure was tested in January 1994 at three locations along the tidal pool wall. Details of the structure are given below :-

Cement type - Ordinary portland cement
Concrete grade - Design 55 MPa, Core 72 MPa
Water / cement ratio - 0.40
Type of reinforcement - Galvanised high yield welded mesh
Cover to steel - Design 100 mm, Recorded 90 - 110 mm
Coarse aggregate - 19 mm greywacke stone (Malmesbury shale - hornfels)
Fine aggregate - Klipheuwel pit sand (Siliceous, well graded)
Construction details - Precast and in situ construction with no moist curing

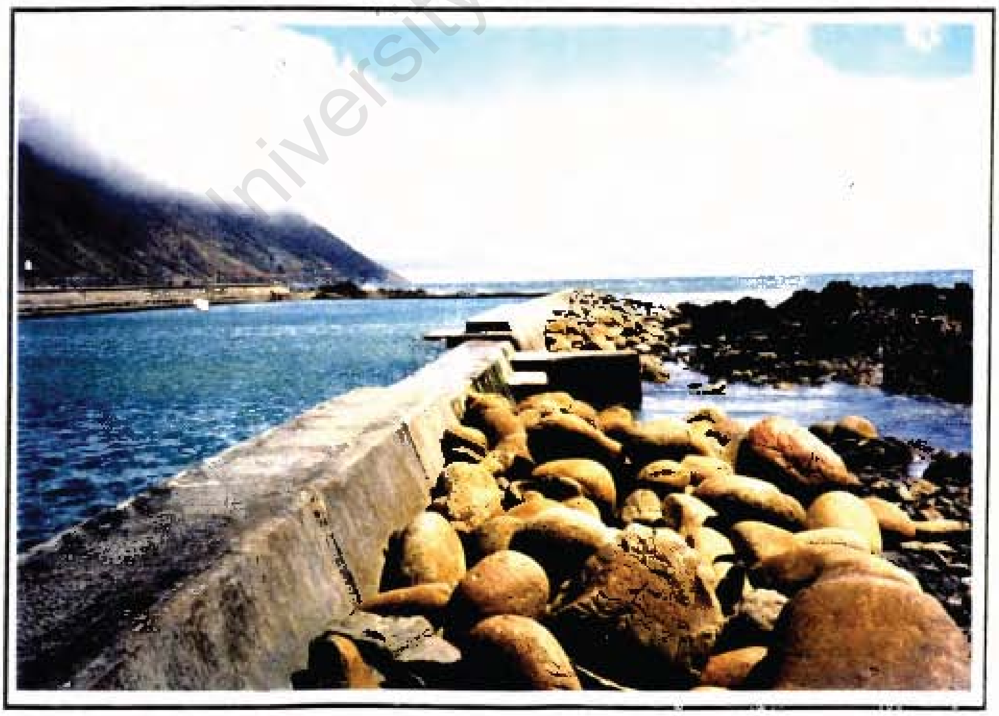

Figure 5.2: Kogel Bay Tidal Pool

5.2.2 Table Bay Breakwater

The Table Bay breakwater was reinforced with new concrete dolosse between 1986 and 1988 using 24 tonne units. The breakwater is subjected to extremely rough weather conditions in winter with swells at the breakwater of over seven metres being recorded. The concrete was generally in good condition but many units were inadequately compacted which resulted in honeycombing and voids. Several dolosse were broken after only six years service on the breakwater due to structural failure. Sixteen dolosse were tested along the the breakwater between September and November 1993. Details of the dolosse are given below :-

Cement type -	Low alkali sulphate resisting cement
Concrete grade -	Design 50 MPa, Core 62 MPa
Water / cement ratio -	0.465
Type of steel -	Railway track for extra weight in some cases
Cover to steel -	300 mm
Coarse aggregate -	19 mm greywacke stone
Fine aggregate -	Klipheuwel pit sand
Construction details -	Precast with mist spraying for seven days

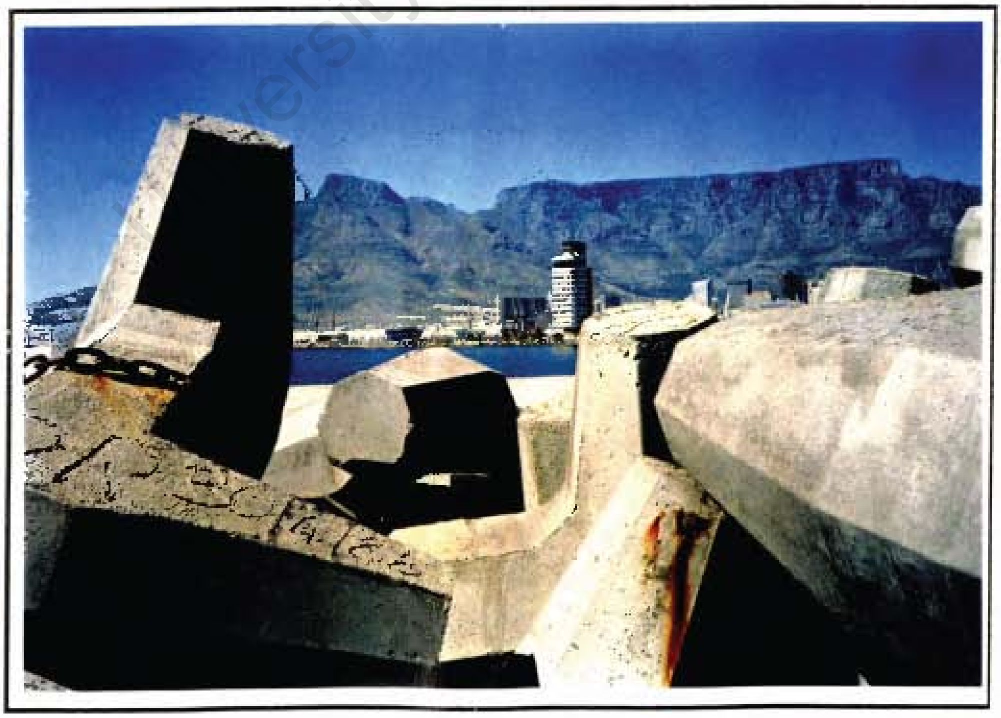

Figure 5.3: Table Bay Breakwater

5.2.3 Koeberg Power Station Pumphouse

The pumphouse at Koeberg Power Station was built in 1982 to provide cooling water to the main reactor building. The structure is adjacent to the sea and is exposed to severe marine conditions with heavy salt spray deposits from wave action and from the primary screens next to the pumphouse. The roof slab exhibited severe damage from steel corrosion in the form of rust staining and cracking. Concrete samples were extracted from seven locations and tested in May 1994 while in situ testing was done over most of the roof slab. Details of the pumphouse are given below :-

Cement type -	Ordinary portland cement
Concrete grade -	Design 30 MPa, Core 45 MPa
Water / cement ratio -	0.67
Type of steel -	High yield ribbed bars
Cover to steel -	Design 25 mm, Recorded 17 - 69 mm
Coarse aggregate -	19 mm granite stone (Cape granite)
Fine aggregate -	Klipheuwel pit sand
Construction details -	Poor cover control with some plastic settlement cracks

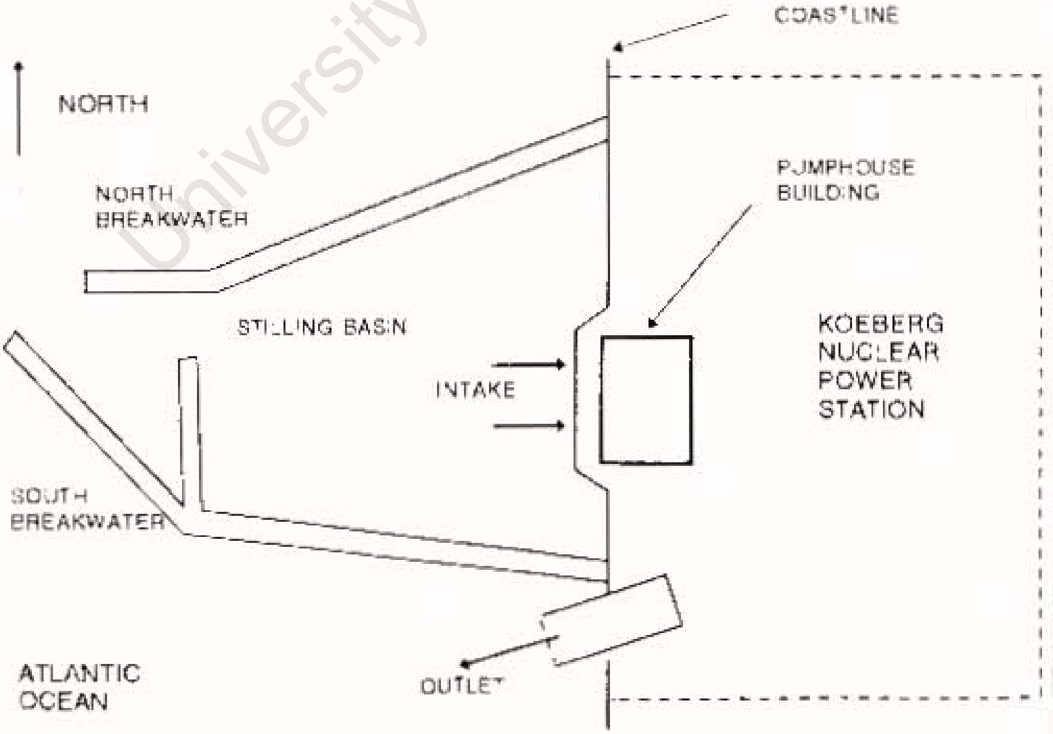

Figure 5.4: Koeberg Power Station Roof Slab

5.2.4 Strandfontein Tidal Pool

The Strandfontein tidal pool was built in 1980 on the Northern end of False Bay. The structure is exposed to moderate wave action and strong winds during summer. The concrete appears to be in fair condition except for abrasion damage on the front section of the wall due to attrition from loose rubble piled against the wall. Testing was done at eight positions along the wall during November 1993. Details of the structure are given below :-

Cement type -	85 % Ordinary portland cement and 15 % fly ash
Concrete grade -	Design 35 MPa, Core 36 MPa
Water / cement ratio -	0.61
Type of steel -	High yield ribbed bar
Cover to steel -	Design 100 mm, Recorded 100 - 150 mm
Coarse aggregate -	19 mm greywacke stone
Fine aggregate -	Cape Flats dune sand (windborne deposits - gap graded)
Construction details -	Cast using a travelling shutter with good compaction

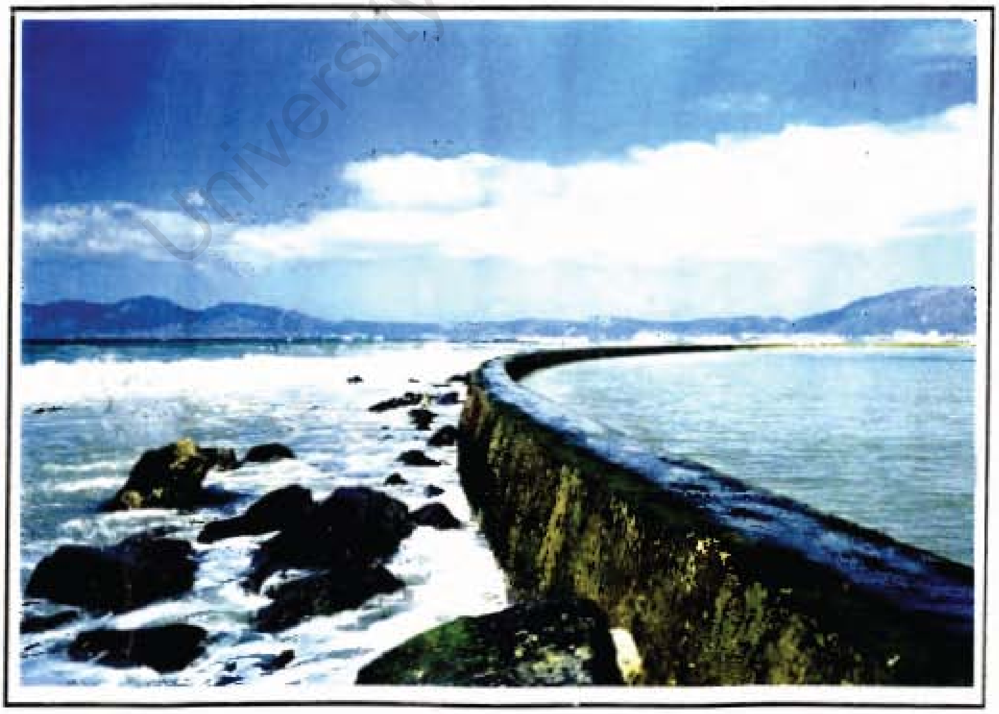

Figure 5.5: Strandfontein Tidal Pool

5.2.5 Camps Bay Pumpstation

The Camps Bay pumpstation was built in 1976 at the top of Camps Bay beach. The area is subject to moderate marine exposure with some wave action during winter storms. The concrete was in good condition apart from some minor weathering of the surface. Three locations were used for the testing in February 1994. Details of the pumpstation are given below :-

Cement type - Ordinary portland cement
Concrete grade - Design 45 MPa, Core 64 MPa
Water / cement ratio - 0.45
Type of steel - High yield ribbed bars
Cover to steel - Design 75 mm, Recorded 80 - 100 mm
Coarse aggregate - 19 mm greywacke stone
Fine aggregate - Cape Flats dune sand
Construction details - Standard in situ construction

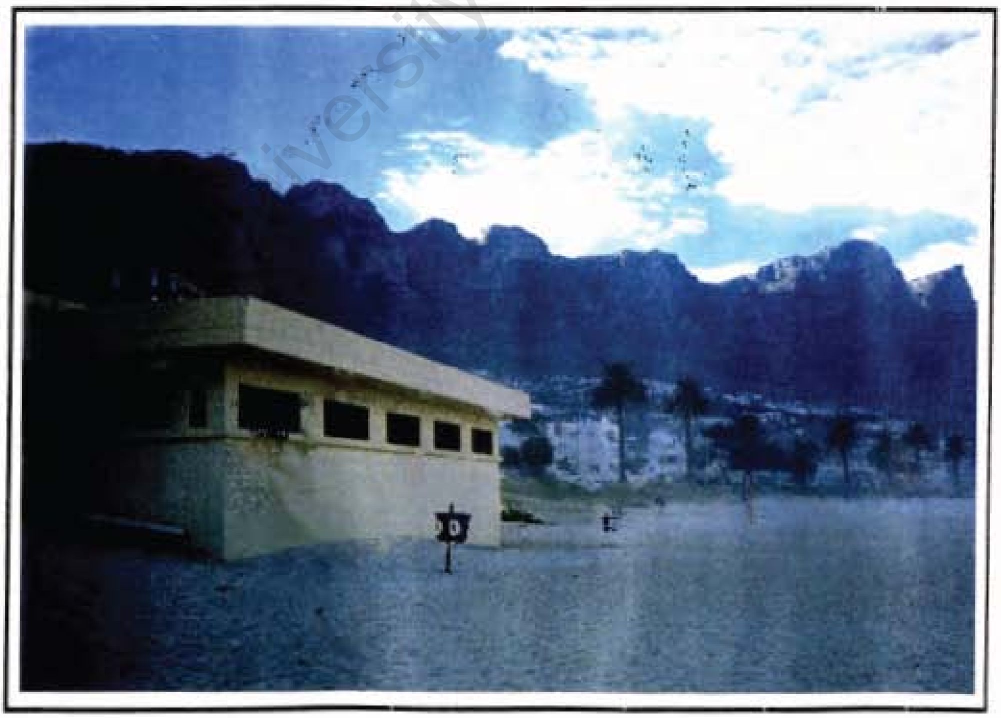

Figure 5.6: Camps Bay Pumpstation

5.2.6 Oudekraal Retaining Wall

The Oudekraal retaining wall was built in 1975 to support Victoria Drive as the road follows the coastline south of Camps Bay. The wall is subjected to regular wetting from wave action and is exposed to severe drying during summer. The structure exhibits several areas of corrosion damage and high levels of abrasion along the bottom of the wall. Six positions were tested on the wall in March 1994. Details of the structure are shown below :-

Cement type -	Ordinary portland cement
Concrete grade -	Design 35 MPa, Core 52 MPa
Water / cement ratio -	0.56
Type of steel -	High yield ribbed bar
Cover to steel -	Design 75 mm, Recorded 35 - 75 mm
Coarse aggregate -	40 mm greywacke stone
Fine aggregate -	Cape Flats dune sand
Construction details -	Standard in situ construction

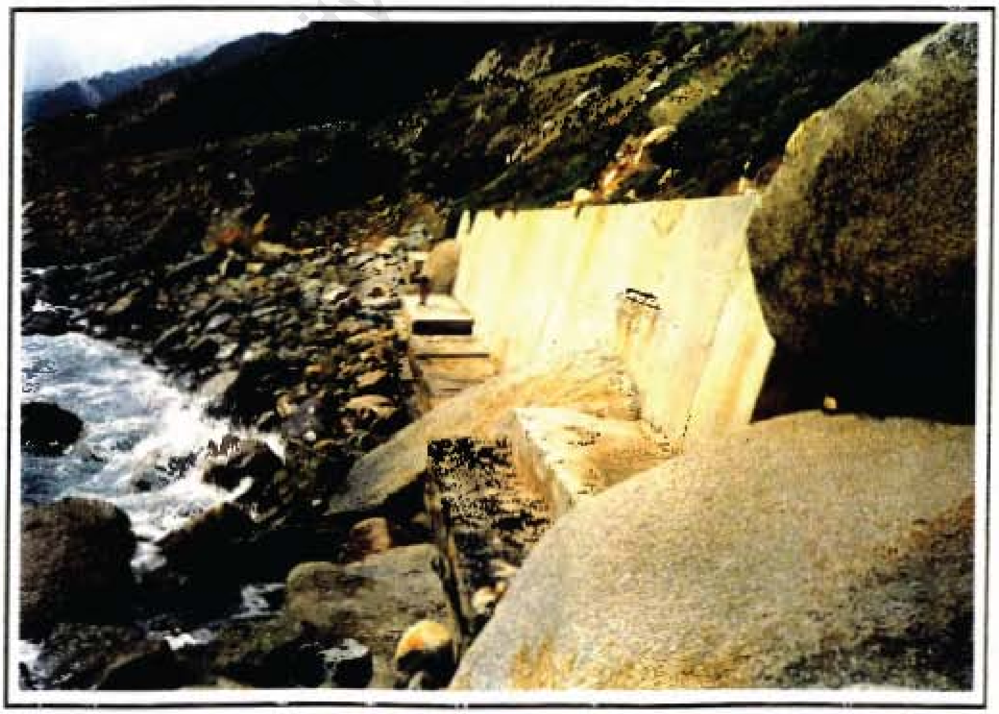

Figure 5.7: Oudekraal Retaining Wall

5.2.7 East London Breakwater

Concrete dolosse were first used on the East London breakwater in 1966 to resist wave action from the prevailing Westerly swells. The breakwater is subject to severe wave action and the area is prone to consistently high wind speeds. The dolosse were generally in poor condition with a high percentage of breakages after 28 years. Nine dolosse were tested along the breakwater during August and November 1994. Details of the dolosse are given below :-

Cement type -	OPC or 50 % OPC and 50 % slag
Concrete grade -	Design 30 MPa (OPC) and 25 MPa (Slag). Core 37 MPa (OPC) and 36 MPa (Slag)
Water / cement ratio -	0.68 (OPC) and 0.70 (Slag)
Type of steel -	No reinforcement
Cover to steel -	N/A
Coarse aggregate -	Graded metamorphosed sandstone
Fine aggregate -	Washed sea sand
Construction details -	Precast on site with minimal curing

Figure 5.8: East London Breakwater

5.2.8 Muizenberg Bridges

The Royal Road and Zeekoevlei bridges were built in 1955 along the Northern edge of False Bay. The bridges are exposed to sea water at high tide and are subjected to severe salt spray deposition from onshore winds. Both bridges exhibited severe corrosion damage on beam and slab soffits which required major rehabilitation work. The bridges were tested at 23 positions primarily for chloride content and carbonation during April 1993. Details of the bridges are shown below :-

Cement type -	Ordinary portland cement
Concrete grade -	Design 30 MPa, Core 37 MPa
Water / cement ratio -	0.67
Type of steel -	Mild steel round bar
Cover to steel -	Design 60 mm, Recorded 40 - 70 mm
Coarse aggregate -	19 mm greywacke stone
Fine aggregate -	Cape Flats dune sand
Construction details -	Standard in situ construction

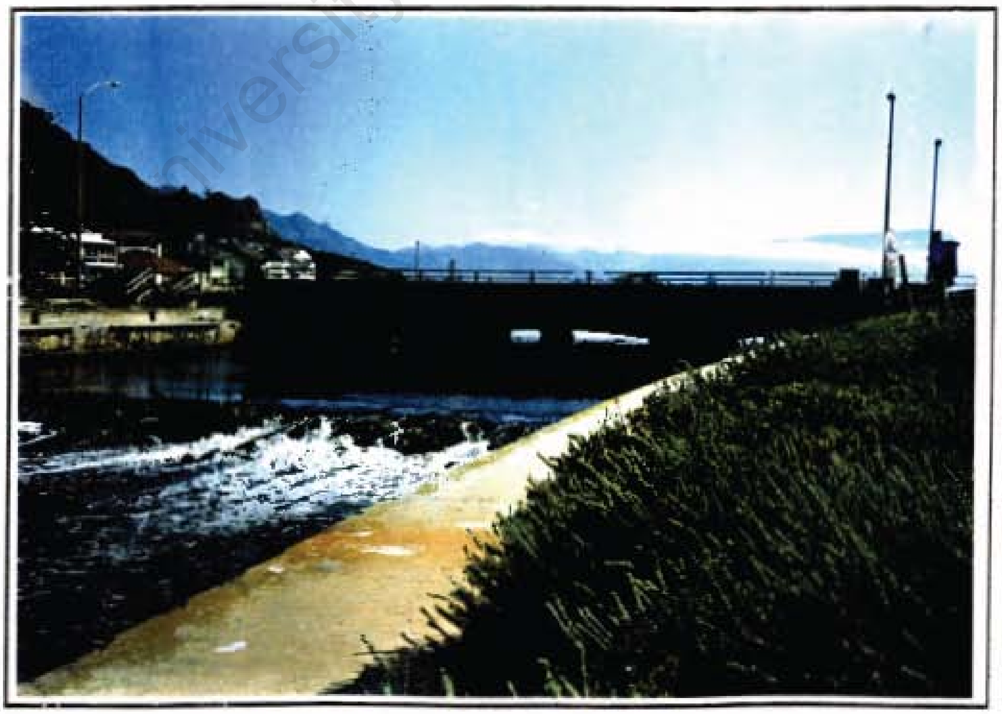

Figure 5.9: Muizenberg Royal Road Bridge

5.2.9 Wilderness Bridges

The Leentjiesklip and Touw River bridges were built in 1948 along the national road N2 on the Southern Cape shoreline at Wilderness. The bridges are exposed to limited direct seawater attack but onshore winds bring in salt spray. Both bridges exhibited minor damage from steel corrosion on the soffits of beams where chloride concentrations were highest. Seventeen locations were used to test the two bridges during August 1994. Details of the structures are given below :-

Cement type -	Ordinary portland cement
Concrete grade -	Design 30 MPa
Water / cement ratio -	0.75
Type of steel -	Mild steel round bar
Cover to steel -	Design 40 mm, Recorded 30 - 85 mm
Coarse aggregate -	19 mm quartzite stone
Fine aggregate -	Local river sand
Construction details -	Standard in situ construction

Figure 5.10: Touw River Bridge, Wilderness

5.2.10 Sea Point Toilet Block

The Sea Point toilet block was built in 1946 immediately adjacent to the swimming pool at the high tide mark. The building is exposed to a moderate marine environment with some wave action and abrasion during winter storms. The structure exhibits widespread corrosion damage in the form of cracking and spalling of the covercrete. Seven sample locations were used for the investigation done in July 1994. Details of the building are shown below :-

Cement type -	Ordinary portland cement
Concrete grade -	Design 25 MPa
Water / cement ratio -	0.75
Type of steel -	Mild steel round bar
Cover to steel -	Design 50 mm, Recorded 24 - 59 mm
Coarse aggregate -	19 mm granite stone
Fine aggregate -	Cape Flats dune sand
Construction details -	Standard in situ construction

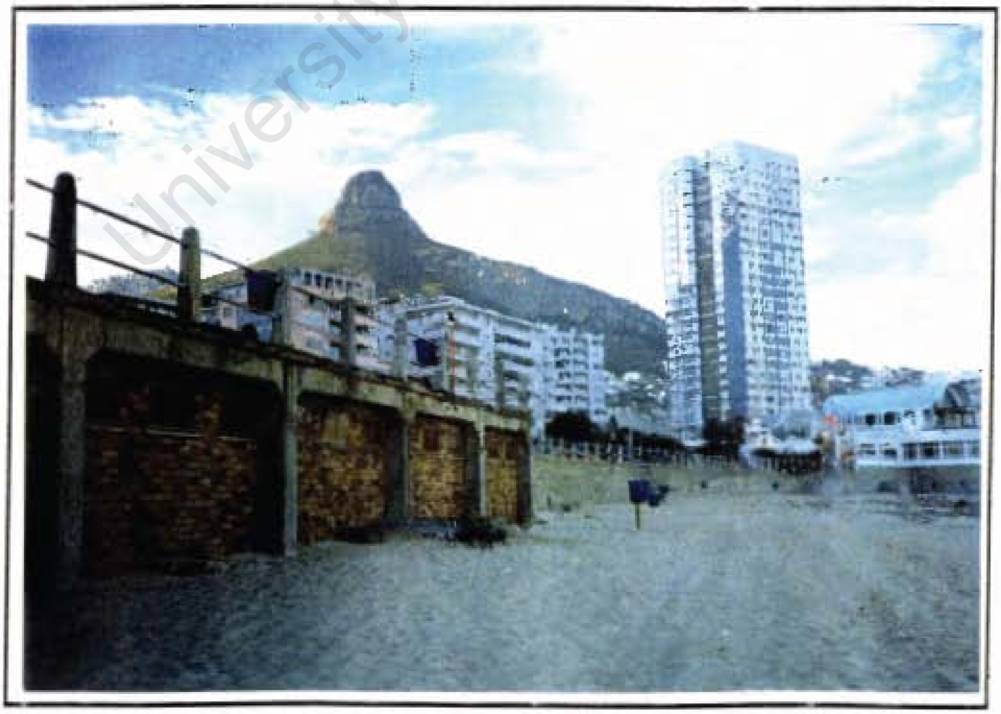

Figure 5.11: Sea Point Toilet Block

5.2.11 Sea Point Aquarium

The Sea Point Aquarium was built in 1939 at the end of the Sea Point promenade near Bantry Bay. The aquarium is set back from the sea but is exposed continuously to seawater from holding tanks in the basement of the building. Widespread corrosion damage was evident in the basement due to chloride ingress, particularly on beam and slab soffits. Eight positions were used to obtain samples along the retaining walls during April 1994. Details of the aquarium are shown below :-

Cement type - Ordinary portland cement
Concrete grade - Design 25 MPa, Core 27 MPa
Water / cement ratio - 0.75
Type of steel - Mild steel round bar
Cover to steel - Design 50 mm, Recorded 10 - 55 mm
Coarse aggregate - 13 mm granite stone
Fine aggregate - Dune sand
Construction details - Standard in situ construction, poor compaction

Figure 5.12: Sea Point Aquarium

5.2.12 Simonstown Jetty

The Simonstown Jetty was built in 1918 to the West of the naval dockyard in Simonstown. The jetty is exposed to mild marine conditions being in a sheltered bay with little wave action. Damage from steel corrosion has reached an advanced level with widespread cracking and spalling of the covercrete surrounding the reinforcement. Corrosion damage has reduced the load carrying capacity of the structure and resulted in major repairs being required to the worst affected members. The residual durability of the structure was assessed in January 1993 by testing at ten locations around the jetty. Details of the structure are given below :-

Cement type -	Ordinary portland cement
Concrete grade -	Design 25 MPa, Core 19 MPa
Water / cement ratio -	0.75
Type of steel -	Mild steel round bar
Cover to steel -	Design 50 mm, Recorded 30 - 90 mm
Coarse aggregate -	13 mm greywacke stone (poor quality)
Fine aggregate -	Dune sand
Construction details -	Standard in situ construction

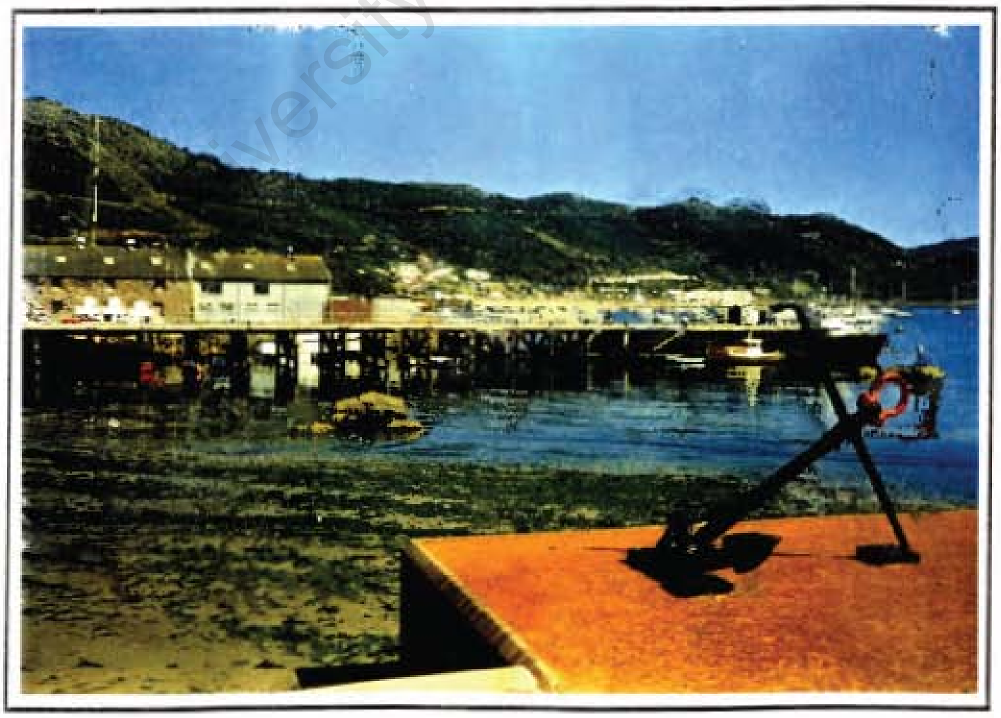

Figure 5.13: Simonstown Jetty

5.3 Experimental Procedure

Durability investigations of marine concrete structures were done using a combination of visual observations, in situ testing and laboratory tests on cored and drilled samples. Core and powder samples were taken from several locations on each structure to produce a representative sample of material conditions. The decision of where to sample was based on an initial visual and in situ survey of the structure to determine the range and extent of deterioration.

Each structure was examined visually to document evidence of damage such as cracking, rust staining, weathering and spalling of concrete. The visual examination was done to identify and define areas of distress but the survey was limited and general interpretations were made as opposed to full documentation of the entire structure. Each reinforced concrete structure was evaluated in terms of the overall extent of reinforcement corrosion and a subjective rating system was proposed.

Corrosion Rating	Description of Condition of Structure
0	As built condition with no damage
1	No evidence of corrosion but exposure damage
2	Minor signs of corrosion (rebar potentials < -350 mV)
3	Rust stains and minor distress
4	Localised areas of spalling
5	Widespread concrete spalling, exposed rebar
6	Severe corrosion affecting structural safety

In situ testing of the structure was done using a combination of resistivity, UPV, reinforcement cover depth and half-cell potential tests. The insitu testing performed on each structure was determined by the age, type and condition of the concrete being investigated.

Chloride-induced corrosion of steel in concrete is associated with anodic and cathodic areas along the reinforcement with consequent changes in electropotential of the steel. Potential mapping of reinforcement is a relatively simple in situ technique which is related to the thermodynamics of steel corrosion and may determine the probability of corrosion occurring in a particular environment but cannot evaluate the kinetics of the reaction. Rebar potentials were determined in accordance with ASTM C876-87 using a copper/copper sulphate half-cell connected to a handheld multimeter [28]. Care was taken to ensure adequate electrical connection to the reinforcement and allowance was made for carbonation and delaminations which may cause misleading results. Interpretation of rebar potentials for pitting corrosion of sound uncarbonated concrete using a copper/copper sulphate half-cell are given below.

Potential (mV)	Probability of Corrosion
> -200	< 5 %
-200 to -350	50 %
< -350	> 95 %

Concrete resistivity is believed to control the rate at which steel corrodes in reinforced concrete once favourable conditions for corrosion have been established [29]. Resistivity is dependent on the moisture condition, permeability and pore water concentration of concrete. Low resistivity values may allow high corrosion rates with the formation of expansive corrosion products causing cracking and spalling. Resistivity was measured using a Wenner probe apparatus connected to a portable resistivity meter. Surface effects such as carbonation, wetting profiles and salt deposition may cause erroneous readings and surface preparation was often required before reliable measurements could be taken [30]. Resistivity readings may provide a rough estimate of the possible corrosion rate as follows [31].

Resistivity (kOhmcm)	Corrosion Rate
< 5	Very High
5 - 12	Moderate to High
>12	Low

Cover depths were determined using a portable covermeter so that the chloride content and alkalinity of the concrete at the steel could be estimated from other tests. The cover depth also affects the rate of steel corrosion and the risk of cracking and spalling of cover concrete encasing corroding reinforcement.

Three concrete cores of 68 mm diameter were extracted at each test position on a structure. These were sealed in plastic bags and returned to the laboratory for testing. The cores were visually examined in the laboratory and the depth of carbonation determined using phenolphthalein indicator solution on the cut surfaces. The cores were then cut using a diamond saw and the outer 25 mm slice used to measure oxygen permeability, water sorptivity, chloride conductivity and abrasion resistance while the inner 60 mm slice was capped and the core strength determined. Details of these durability index tests are given in Chapter Three.

Concrete powder was extracted from structures using a plastic tube and sample packet held around the 20 mm drill bit of a rotary hammer drill. The outer 5 mm of each drilling was discarded due to local and seasonal fluctations of surface concentrations of chlorides. Concrete was extracted at 25 mm depth increments to a depth of 100 mm with powder from three 20 mm holes being combined to form a single sample. The concrete powder was analysed for chlorides in accordance with BS 1881 Part 124 but using a potentiometric titration [32]. Three titrations were done on each sample and outliers discarded using the method given in ASTM E178-80 [33]. Diffusion coefficients and surface concentrations were determined from the chloride profiles in accordance with details given in Chapter Four.

The presence of sufficient chloride at the surface of embedded reinforcement may depassivate steel and allow corrosion to occur under favourable conditions. Chloride contents of above 0.40 % at the steel (corrosion threshold) are generally sufficient to depassivate reinforcement while higher levels are required for high corrosion rates associated with cracking and spalling damage. Factors causing rapid corrosion of reinforced concrete are complex but chloride contents at the steel may be classified in accordance with a qualitative risk of corrosion as follows [34].

Chloride Content By Mass of Cement (%)	Qualitative Risk of Corrosion
< 0.4	Negligible
0.4 - 1.0	Possible
1.0 - 2.0	Probable
> 2.0	Certain

5.4 Experimental Results

Results from the investigation of twelve marine concrete structures produced a wide range of trends which are discussed in terms of observations from each structure and general observations.

5.4.1 Observations From Structures

a) Kogel Bay Tidal Pool

A grade 60 OPC concrete was used for precast units which made up the outer walls of the tidal pool wall and the concrete appeared to have resisted the severe abrasive conditions satisfactorily. Concrete cores extracted from the wall showed some evidence of poor compaction but durability index results indicated a dense, high quality concrete with good potential durability while core strengths were between 70 and 82 MPa.

Chloride ingress was surprisingly high for a grade 60 concrete which may have been partly due to the lack of active curing and inadequate compaction. The average diffusion coefficient of concrete was 5.8E-8 cm^2/s after three years exposure which was much higher than the diffusion coefficiente of 2.5E-8 cm^2/s for grade 60 OPC concrete blocks after two years exposure at Granger Bay (Chapter Four). The concrete should however provide adequate protection to the reinforcement as the steel is galvanised and at covers in excess of 100 mm. No repair or preventative maintenance was therefore needed for the structure.

b) Table Bay Breakwater

The quality of concrete observed on the dolosse varied considerably with many units exhibiting honeycombing, cracks and poorly cast cold-joints at pod joints. The 24 tonne concrete armour units were exposed to heavy wave action particularly during winter storms and several dolosse had been damaged. Defects at pod joints probably contributed to many of the failures observed at the front of the breakwater. While fluid transport mechanisms have limited bearing on the durability performance of mass concrete structures such as armour units, useful information was obtained regarding the chloride resistance of concrete under extreme marine conditions. Figure 5.14a shows the general condition of the dolosse.

Figure 5.14a: General Condition of Concrete on Dolosse

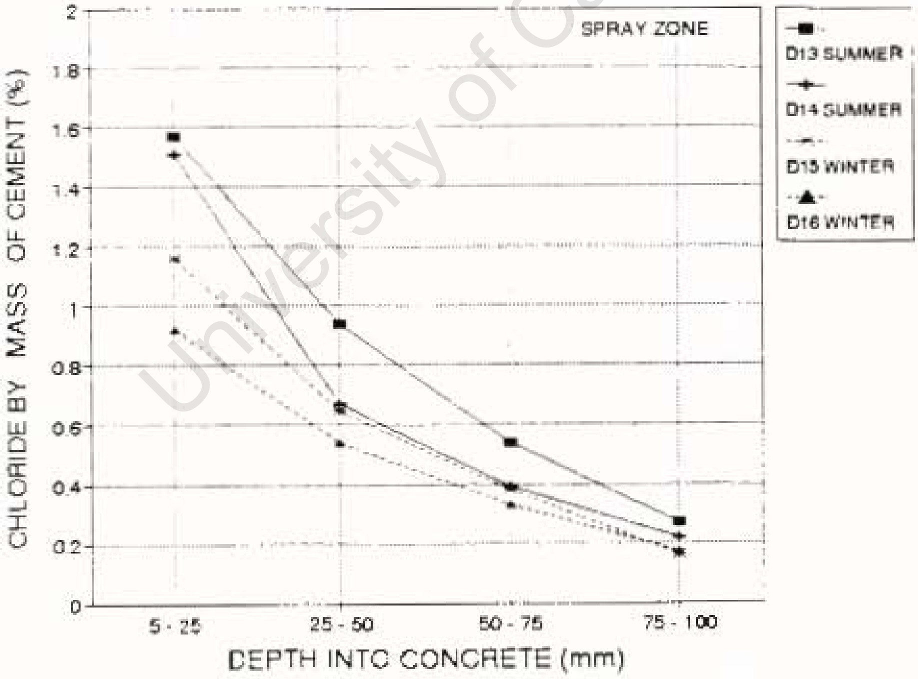

Figure 5.14b: Chloride Profiles in the Spray Zone

Chloride ingress was uniformly high in the tidal, splash and spray zones with an average diffusion coefficient of 5.0E-8 cm²/s after six years. Concrete cast in summer had slightly higher chloride levels than concrete cast in winter. The difference in chloride levels may be attributed to the drying of concrete during hot and dry summer conditions and limited active curing. In winter the cooler, moist conditions allowed extended moist curing from the environment resulting in a dense concrete with better chloride resistance. Chloride profiles for summer and winter cast dolosse in the spray zone are shown in Figure 5.14b. No repairs were recommended for the breakwater but improved construction control was suggested for new dolosse cast in the future.

c) Koeberg Pumphouse Roof

Severe corrosion damage was observed in several areas on top of the roof slab which were generally associated with low cover depths. Chloride contents at the reinforcement were uniformly high and had penetrated in significant quantities to a depth of 75 mm. In situ tests confirmed visual evidence of corrosion but interpretation was difficult in areas where delamination of the cover concrete had occurred. Table 5.3 gives details of cover depths, resistivity, half-cell potentials and chloride contents at the reinforcement. Corroded areas had low cover depths, low resistivity and high negative potentials while sound areas generally had deeper covers, high resistivity and medium to low negative rebar potentials.

Table 5.3: Experimental Results From Koeberg Pumphouse Roof

SAMPLE NUMBER	COVER DEPTH (mm)	RESIST-IVITY (kOhmcm)	Ecorr (Cu/CuSO4) (-mV)	Cl @ STEEL (%)	GENERAL CONDITION
K1	60	24	-	0.25	Good with minor abrasion damage
K2	23	7	570	0.59	Cracking and major spalling
K3	39	21	146	0.41	Fair with some weathering
K4	50	6	524	0.48	Cracking and delamination
K5	61	33	176	0.49	Fair with some weathering
K6	65	23	181	0.41	Weathering and minor cracks
K7	40	15	385	0.44	Rust stains and cracking

The use of a grade 30 OPC concrete with a specified cover of 25 mm in a severe marine environment made premature durability failure inevitable. Patch repairs were not recommended given the extent of the damage and previous attempts at localised repairs on the roof had proven to be ineffective. Electrochemical desalination was also not recommended as chlorides had penetrated beyond the steel and would not be removed from the concrete during desalination and might diffuse back to the steel with time. Mechanical removal of chloride contaminated concrete was therefore recommended and repairs coated with a suitable waterproof screed to prevent further chloride ingress.

d) Strandfontein Tidal Pool

The Strandfontein tidal pool wall was in good condition after 13 years service apart from some abrasion damage on the front of the wall, shown in Figure 5.15a. Examination of concrete cores revealed a dense material with no large pores and few visible voids. Durability index tests indicated a high quality concrete with low permeability but with below average abrasion resistance but a core strength of 36 MPa was measured for the grade 35 concrete.

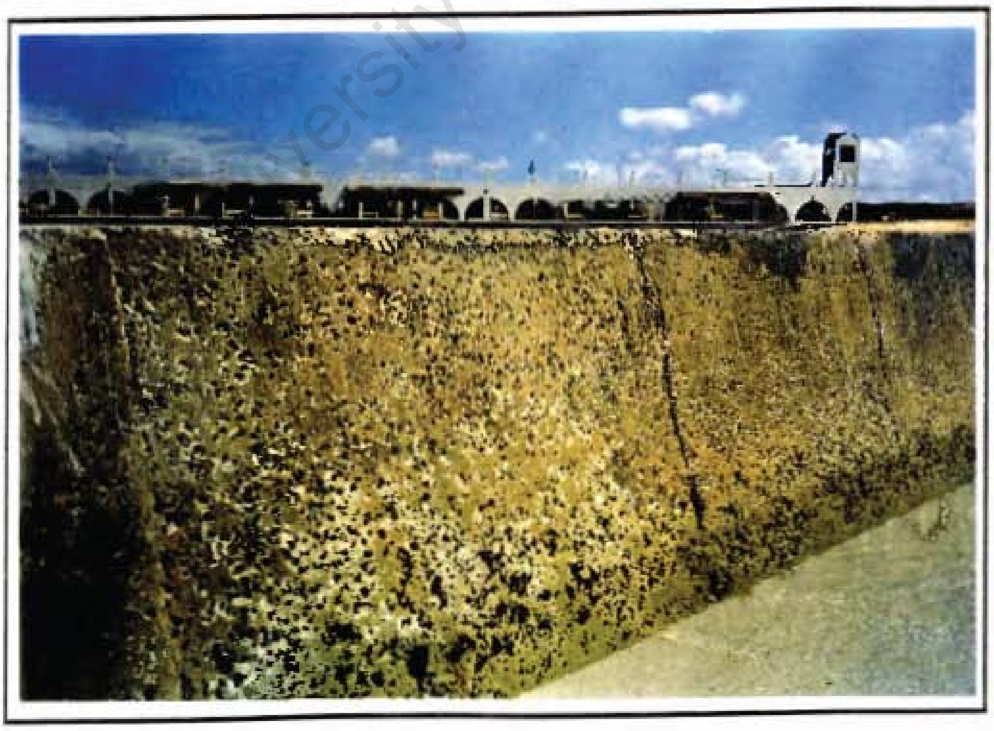

Figure 5.15a: Abrasion Damage on Front of Wall

The rate of chloride ingress was low with an average diffusion coefficient of 1.9E-8 cm²/s after thirteen years. Concrete exposed to abrasion on the front of the wall had higher chloride contents than similar concrete on the sides of the pool sheltered from abrasion. Abrasion was believed to have destroyed the protective layer of insoluble salts (brucite and aragonite) which deposit on or near the concrete surface of marine structures and restrict the access of seawater into the concrete [35]. Submerged concrete on the inside of the wall had lower chloride contents than tidal concrete due to the protected nature of submerged exposure and the lack of wetting and drying associated with tidal action. Figure 5.15b shows the chloride profiles for the three different exposure environments.

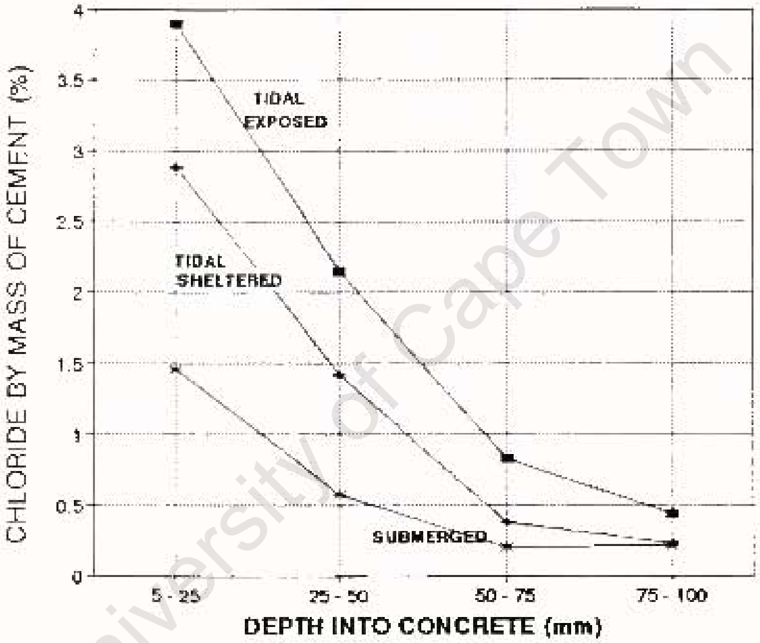

Figure 5.15b: Chloride Profiles For Strandfontein Tidal Pool Wall

The tidal pool wall was in good condition but continued abrasion of the concrete could adversely affect the durability of the structure. Recommendations were made to secure loose rubble on the seaward face of the wall and thereby reduce further abrasion damage.

c) Camps Bay Pumpstation

The pumpstation was exposed to mild marine conditions being exposed directly to seawater only at spring high tides. The concrete was in good condition with only minor weathering while cover depths were greater than 80 mm. Core strengths of over 60 MPa were recorded but durability index results were average except for permeability which was high most probably due to poor compaction of the concrete evident from the presence of large voids.

The rate of chloride penetration was relatively slow with an average diffusion coefficient of 1.9E-8 cm^2/s and a low surface concentration of 1.2 %. The relatively low rate of diffusion was probably due more to the mildness of exposure rather than the inherent material resistance of the concrete which was expected to be particularly good given the durability index test results. Chloride contents at the steel ranged between 0.08 and 0.15 % which were well below the corrosion threshold level. No repairs or preventative maintenance was thought necessary for the foreseeable future but further monitoring should be undertaken.

f) Oudekraal Retaining Wall

The retaining wall was in poor condition after nineteen years service with localized areas of rust staining, concrete spalling, salt crystallization and abrasion damage. Corrosion damage occured in areas of low cover and poorly compacted concrete. At the South end of the wall, cover depths of 35 mm contributed to the severe corrosion and spalling of the cover concrete. Other areas had cover depths greater than 70 mm and rebar potentials were generally in the passive range (> -350 mV). Figure 5.16a shows the extent of corrosion damage on the wall.

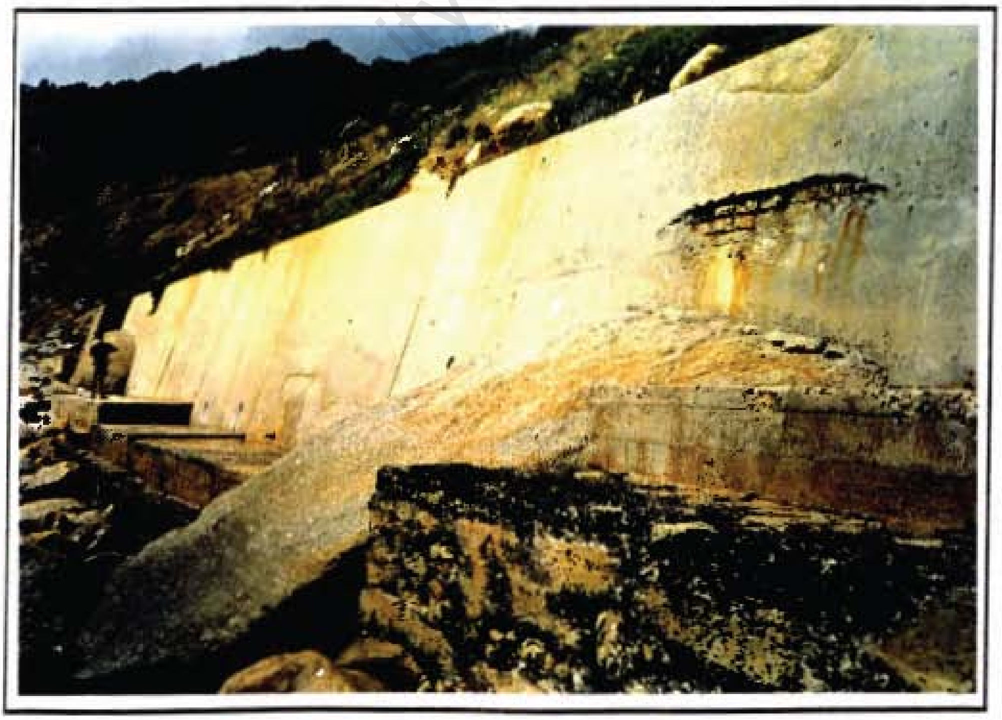

Figure 5.16a: Evidence of Corrosion Damage on Retaining Wall

Concrete in the splash zone had fairly high chloride levels with an average diffusion coefficient of 1.4E-8 cm²/s indicating a moderate rate of chloride ingress. Defective areas such as cracks and honeycombing would allow more rapid chloride ingress than that measured in sound concrete. Chloride contents at the steel were between 0.12 and 0.35 % for cover depths of 70 mm and were therefore lower than the corrosion threshold level. Predictions of future chloride levels based on a constant diffusion coefficient of 1.4E-8 cm²/s and a surface concentration of 3.0 % are shown in Figure 5.16b. The age referred to in the figure is the time of exposure of the structure from construction. Predicted chloride levels indicate that the corrosion threshold will soon be exceeded at cover depths of 70 mm causing more widespread steel corrosion.

Recommendations were made for immediate repairs of corrosion damage and defective areas such as honeycombing and cracking. Protection of the concrete using a barrier system was also suggested to prevent chloride reaching dangerous levels at the steel and causing widespread corrosion damage of the retaining wall.

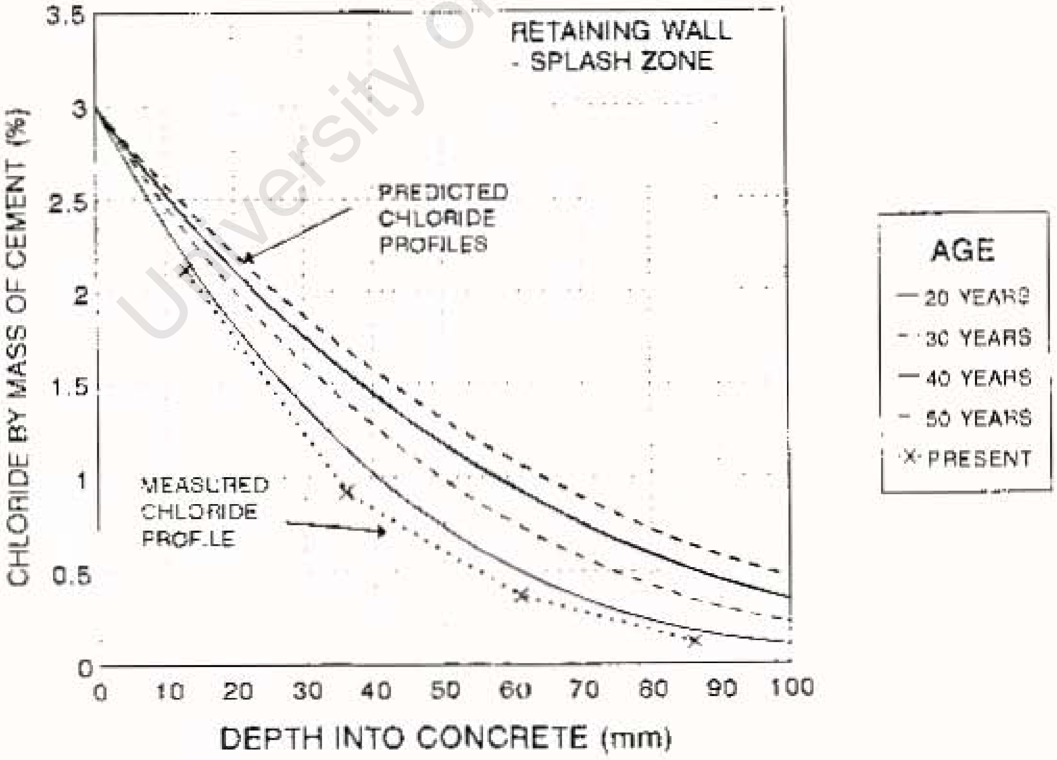

Figure 5.16b: Predicted Chloride Profiles for Oudekraal Retaining Wall

g) East London Breakwater

The inadequate grade of concrete used for dolosse (25, 30 MPa) on the East London Breakwater resulted in poor durability performance with many breakages and the need for regular repairs. The breakwater is exposed to extremely heavy wave action with large South Westerly swells being common and generate huge forces on the 20 tonne armour units. The excessive damage of dolosse has resulted in the harbour authorities having to replace damaged and lost dolosse with new grade 40 dolosse at regular intervals. Figure 5.17a shows typical damage experienced by armour units due to storm waves.

Figure 5.17a: Example of Physical Damage of Dolosse

Comparative information was obtained for OPC and slag concrete by testing dolosse cast from either type of concrete, both of which were used during initial construction of the breakwater. Before comparisons are made between OPC and slag concrete some allowance must be made for the lower grade of the slag (25 MPa) to that of the OPC concrete (30 MPa) and the lack of effective moist curing of the dolosse after casting. After 28 years the core strength of both concrete types was similar being on average 37 MPa. Visual examination of dolosse on site indicated that OPC concrete had less abrasion and salt crystallization damage than slag concrete and this trend was also found when cores were tested for abrasion resistance in the laboratory. This trend might not have been so apparent at higher grades such as 40 and 50 MPa which are currently used for dolosse production. Slag concrete was more impermeable, had lower chloride conductivity and lower chloride contents than OPC concrete indicating the advantage of using slag concrete for reinforced concrete structures in the marine environment. Average chloride profiles for OPC and slag concrete are shown in Figure 5.17b.

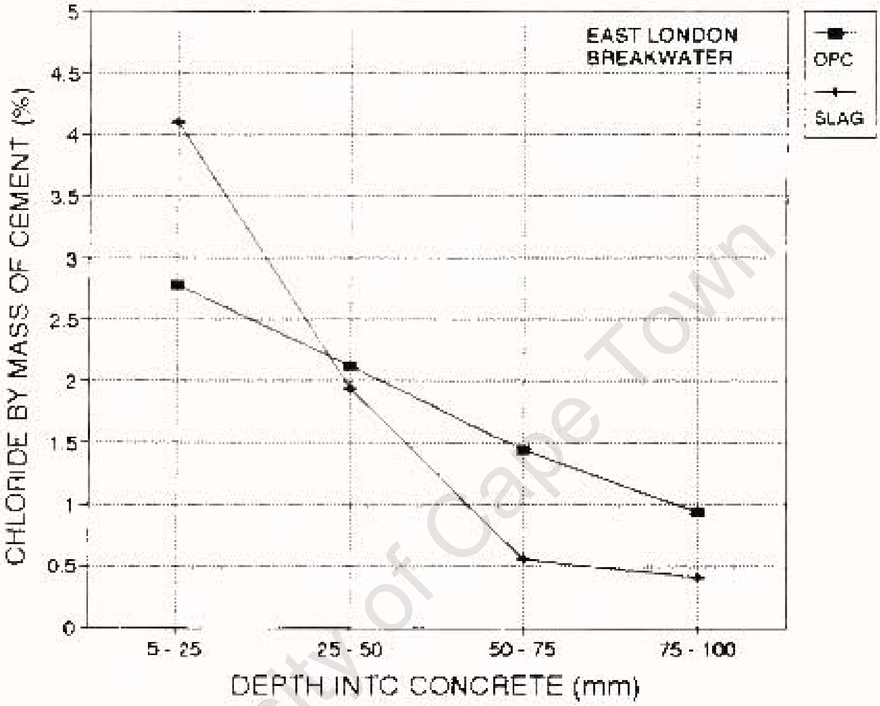

Figure 5.17b: Average Chloride Profiles for OPC and Slag Concrete

The investigation found that strength should be the main consideration when designing concrete armour units to ensure adequate performance under the severe environmental and loading conditions prevalent on a harbour breakwater. The refined pore structure and chloride resistance of slag concrete are less important on mass concrete structures than on reinforced concrete structures.

h) Muizenberg Bridges

Both the Royal Road and Zeekoevlei bridges exhibited corrosion damage on beam and slab soffits with chloride contents at the steel ranging between 0.17 and 2.26 %. The original grade of 30 MPa and cover depths of 50 mm were inadequate for the severe exposure conditions. Concrete in abutments and piers had high chloride contents but exhibited no signs of corrosion probably due to the saturated nature of the concrete, which retarded the rate of corrosion. The bridge decks were isolated from abutments and piers and corrosion occurred preferentially at the bottom of beams and slabs.

Due to the advanced nature of corrosion on the Zeekoevlei bridge deck, remedial work was not considered and complete removal of the bridge deck was recommended. Royal Road bridge had suffered less corrosion damage and chloride penetration was generally less than 100 mm. Localized remedial work was recommended to remove chloride-contaminated concrete and repair with a high quality material.

i) Wilderness Bridges

The Leentjiesklip and Touw River bridges both showed signs of corrosion which was generally limited to longitudinal cracks and minor spalling along beam soffits. Concrete of grade 25 and cover depths of as low as 30 mm were responsible for the deterioration observed after 46 years. Apart from superficial evidence of distress the bridges were in good condition and the structural integrity did not appear to be adversely affected. Figure 5.18 shows localized corrosion damage on a beam soffit of the Touw River Bridge.

Figure 5.18: Corrosion Damage of Touw River Bridge

The Leentjiesklip bridge is located 100 m from the sea on a hill overlooking Wilderness and was exposed to mild levels of sea spray carried by onshore winds. Chloride penetration had occurred to a depth of 75 mm on seaward faces and soffits of beams and minor corrosion damage was evident in the form of cracks and rust stains. The Touw River bridge spans across a tidal estuary and had moderate levels of corrosion damage with some spalling on beam and slab soffits. Chloride penetration had occurred to a significant extent at the top and bottom of the bridge deck and in piers and abutments. Chloride contents at the steel were above 1.0 % at both top and bottom steel and rebar potentials were highly negative in several areas (< - 400 mV on slab and beam soffits).

Suitable remedial work was recommended to prevent further damage from reinforcement corrosion and ensure continued serviceability of the two structures. Further monitoring might assist in determining if repairs were effective in ensuring the continued safety of the structures.

j) Sea Point Toilet Block

The investigation of the toilet block was of a very limited nature due to the extent of deterioration and unreliable construction and design records. The structure was in poor condition with widespread cracking and spalling from reinforcement and the concrete appeared to be weak and friable. Chloride contents were all above 1.0 % and reached levels of 2.8 % in places. Carbonation depths of 70 mm were recorded inside the building with exterior values of 15 mm. Carbonation is known to release bound chlorides which may then be washed out of the concrete by rainfall and water seepage and this appears to have occurred to both exterior and interior concrete tested on the structure. Figure 5.19 shows the chloride profiles and carbonation depths for interior and exterior concrete from the toilet block.

The concrete had reached an advanced level of deterioration and parts of the building were unsafe due to delaminations of the cover concrete on roof soffits. It was therefore recommended that the structures be demolished and rebuilt using a durable concrete, preferably containing slag or fly ash for chloride resistance and with minimum cover depths of 60 mm.

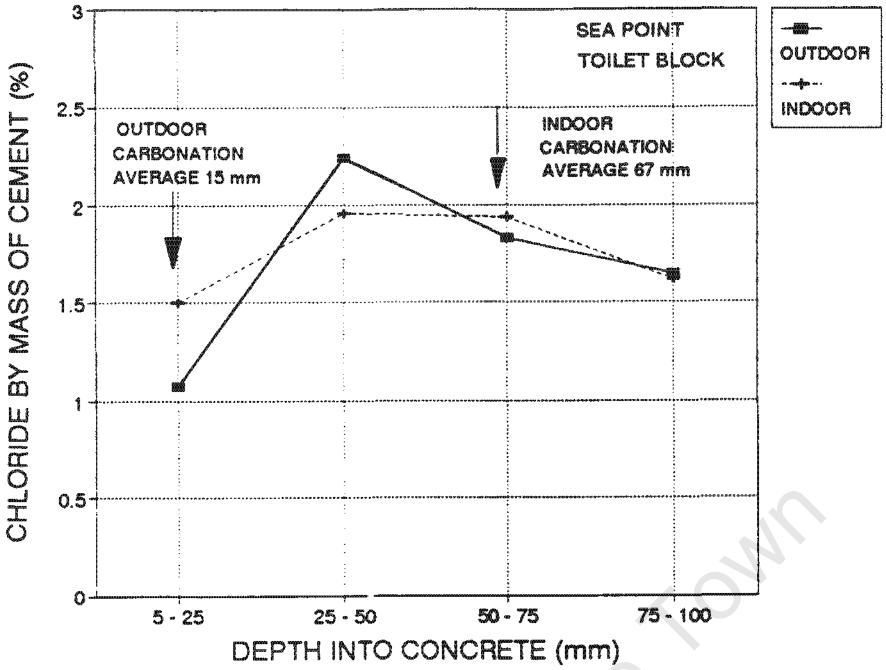

Figure 5.19: Chloride Profiles and Carbonation for Toilet Block

k) Sea Point Aquarium

Deterioration of the aquarium building had reached an advanced stage after 55 years where even the structural integrity had been compromised. Reasons for the damage include low grade concrete (25 MPa), poor design allowing cover depths of 30 mm and congested steel preventing full compaction of concrete and uncontrolled leaking of seawater from overhead tanks. Chloride contents and carbonation were high at all test positions and both deterioration processes had progressed sufficiently to activate corrosion of the reinforcement.

Interior beams and columns exhibited severe corrosion damage while boundary retaining walls appeared to be in fair condition apart from minor rust staining. Rebar potentials and examination of cores extracted from the retaining walls revealed that the reinforcement in the walls had corroded extensively. The lack of evidence of distress on the surface of the walls was due to cover depths of 50 mm or more and small bar diameters (12 mm) which resulted in negligible expansive forces being generated from the corrosion product at the bar. Interior beams and columns on the other hand had low cover depths (30 mm) and large bar diameters (25 mm) which generated large expansive forces resulting in cracking and spalling. Figure 5.20 shows the geometry of reinforcing in retaining walls and interior beam and columns.

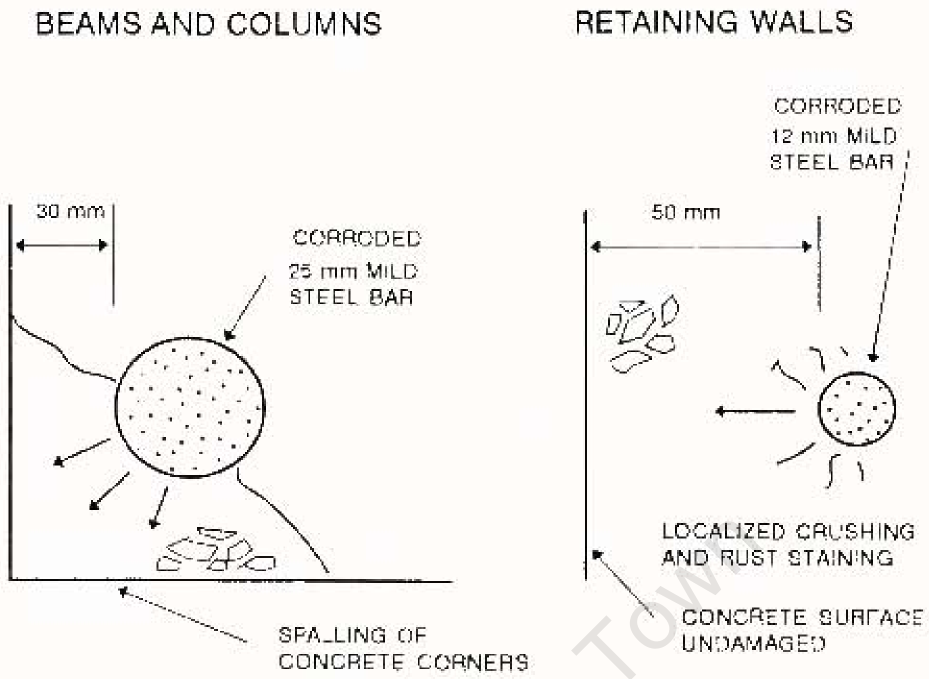

Figure 5.20: Corrosion of Steel in Beam, Columns and Walls

Given the extent of deterioration and the loss of structural integrity, the only suitable repair option was demolition of the structure. A grade 40 slag or fly ash concrete with minimum cover of 50 mm was recommended for the new aquarium building.

l) Simonstown Jetty

After 75 years service, the Simonstown Jetty had suffered extensive corrosion damage and there was concern about the load carrying capacity of the structure. The worst affected areas of corrosion damage were above the high tide level and on beam soffits. Corrosion damage was exacerbated by the slender configuration of piers, beams and diagonal braces supporting the deck slab. Several piers had been propped to prevent further structural damage as can be seen in Figure 5.21a which shows the extent of corrosion damage on the first span of the jetty. Extensive patch repairs of corrosion damaged area had been done in the past and repairs done in 1989 had failed when examined in 1993 and appeared to provide minimal protection to the structure.

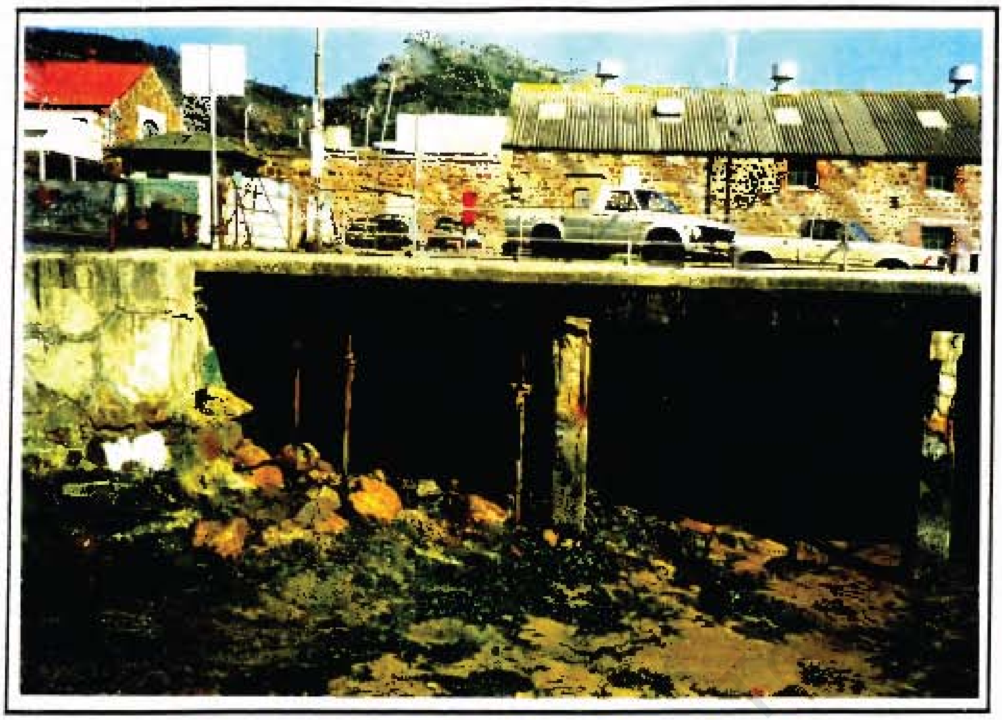

Figure 5.21a: Corrosion Damage of Simonstown Jetty

Carbonation depths were all below 7 mm but chloride levels were extremely high in the tidal zone. Chloride contents in the concrete were significantly higher in the tidal zone than in the splash zone due to the protected nature of the site with little wave action. Figure 5.21b shows the chloride profiles for tidal and splash zone concrete. Core strengths were low (17 - 22 MPa) which indicates some loss of strength for the original grade 25 MPa material. The concrete had poor durability index results and high permeability in some samples which indicated internal damage.

The continued use of patch repairs to rehabilitate the structure will not significantly prolong the service life of the structure and the money should rather be invested in long-term repair solutions. The structure has reached an advanced level of deterioration with high levels of chloride contamination which would prevent conventional repairs from being effective. Cathodic protection coupled with strengthening of the structure may provide the only practical solution and may be more cost effective than demolition and rebuilding of the jetty. The aesthetic attraction and historical value of the jetty may make cathodic protection the only suitable repair solution if demolition is not possible. This advice was accepted and major repairs using cathodic protection are planned for 1996.

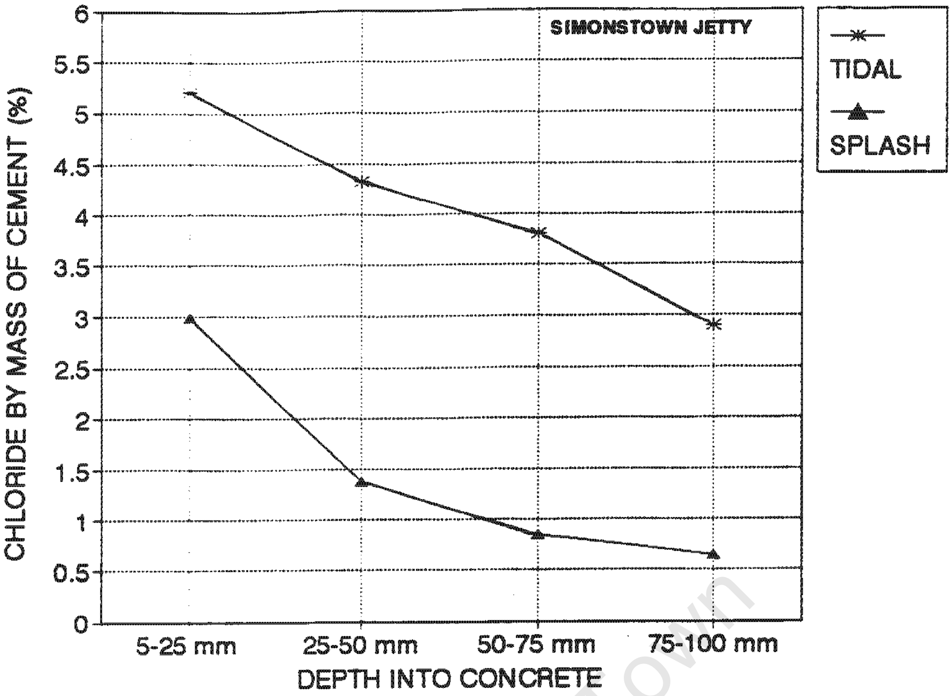

Figure 5.21b: Chloride Profiles in Simonstown Tidal and Splash Zones

5.4.2 General Observations

a) Core Strength

The use of strength as a durability criterion for concrete structures has been popular for many years even though there is often little correlation between the two properties. High strength concrete may have poor surface qualities due to inadequate curing and compaction allowing relatively easy ingress of aggressive agents. Core strength testing was done to assess the residual strength of the concrete and to estimate the original grade of the concrete. Core strength results from older structures were fairly low, for instance the Simonstown Jetty had a residual strength of less than 20 MPa. Modern structures had much higher core strengths reflecting the trend toward higher grade concrete for marine structures. This trend of increasing strength is shown in Figure 5.22a.

Most cores tested showed little evidence of deterioration from the marine environment although older concrete from the Sea Point Aquarium and Simonstown Jetty was friable and appeared to have suffered internal damage. Coarse aggregate from older structures was also of poor quality either in terms of size, grading or soundness. This poor quality of the aggregate could have contributed to the poor strengths achieved. Core strength results were generally higher than the original specified strength due to continued cement hydration and differences between characteristic and target strength. The percentage of core to design strength for different aged marine concrete structures is shown in Figure 5.22b.

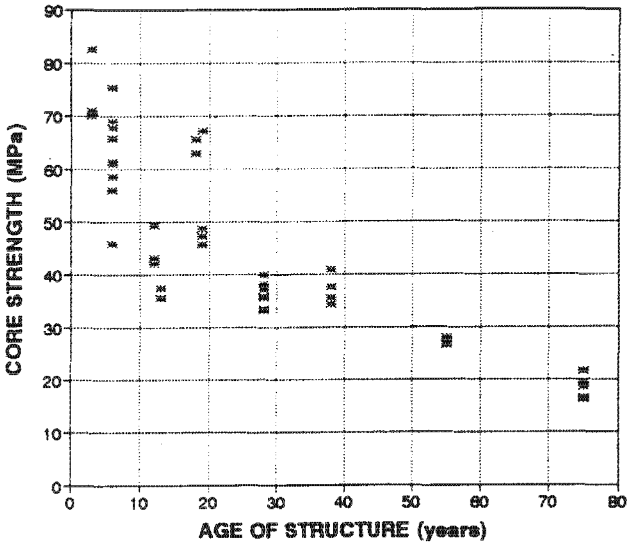

Figure 5.22a: Core Strength vs Age of Structure

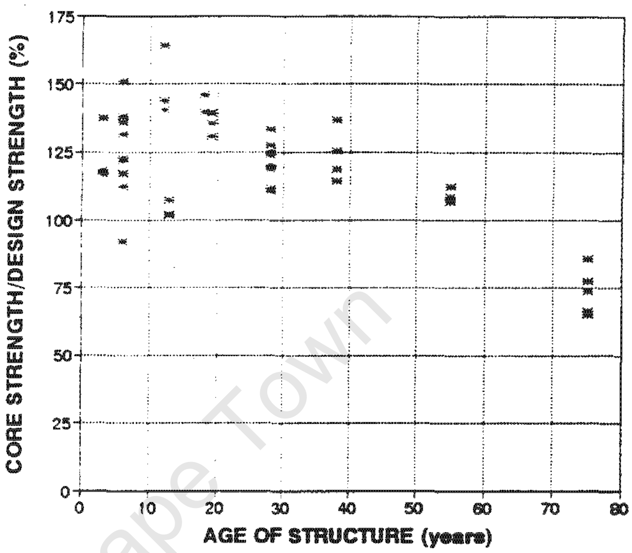

Figure 5.22b: Core/Design Strength vs Age of Structure

b) Oxygen Permeability

The oxygen permeability test uses a falling head permeameter to measure the permeability of concrete to oxygen under pressure. Permeability measurements and pore size distributions measured by Young indicate that fluid flow under pressure is strongly influenced by the macropore fraction within the cement paste [36]. The coarser pores allow virtually unrestricted passage for the bulk of permeating fluids and therefore control the permeation characteristics of the material. Permeability testing should therefore be more sensitive to macro-defects such as cracking and poor compaction than to minor pore structure changes caused by chloride binding and carbonation. The oxygen permeability test was expected to be useful for assessing the overall microstructural state of the cover concrete while being less sensitive to surface phenomena such as salt deposition and carbonation.

Oxygen permeability appeared to increase (i.e. index decreased) with increasing age of the structure as shown in Figure 5.23. Virtually all structures over twenty years old had oxygen permeability index values of less than 9.0 which is equivalent to a moist cured 20 MPa concrete tested at 28 days. Such poor permeability values may be ascribed to microstructural damage due to environmental and/or structural effects or older concretes having a coarser pore structure than modern concretes. Modern specifications generally call for at least a 40 MPa concrete with seven days moist curing or equivalent. Concretes made according to specifications would have resulted in an oxygen permeability index of about 9.80 after 28 days. Of the modern structures tested (less than 20 years) only Kogel Bay and Strandfontein tidal pools had higher values (10.5 and 9.9). This was probably due to the high grade of concrete used at Kogel Bay (60 MPa) and the use of fly ash and high frequency vibration at Strandfontein.

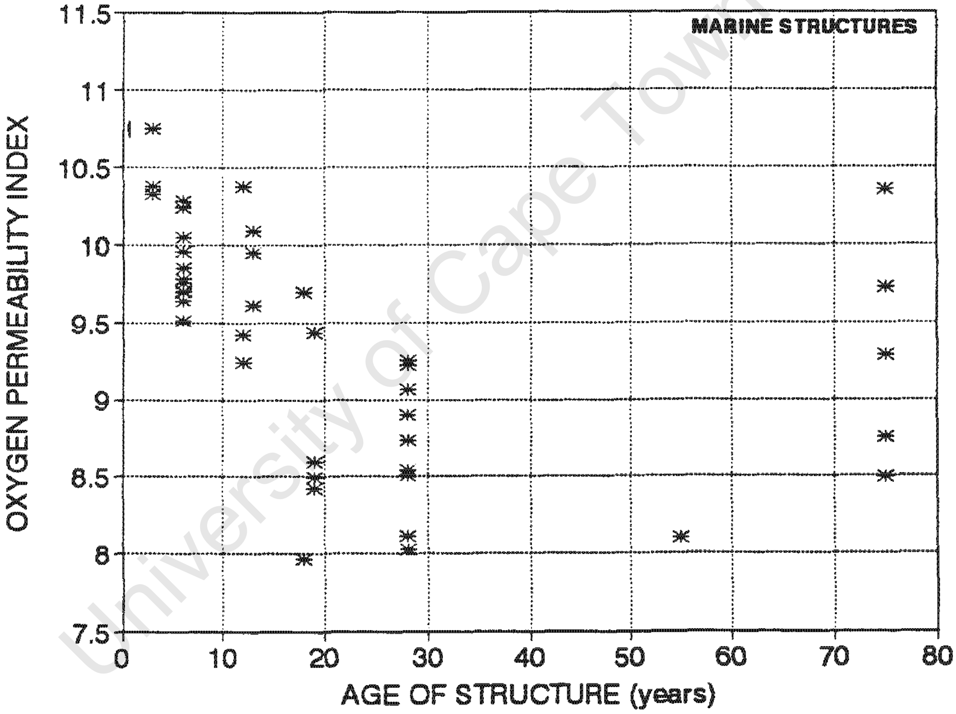

Figure 5.23: Oxygen Permeability Index vs Age of Structure

Older structures produced misleading oxygen permeability results particularly Simonstown Jetty where some concrete samples were almost saturated with salt which may have contributed to the low permeability measured. The observed results were therefore the combined effect of the material resistance together with the relative amount of salt ingress into the pore structure. Electrochemical desalination might be an effective method of removing salt from concrete to produce a more reliable indication of the inherent permeability of the material.

c) Water Sorptivity

Measurements of water sorptivity appeared to be significantly affected by surface effects such as salt deposition and carbonation. Sorptivity values fell within the range of 1.0 to 5.0 mm/hr$^{1/2}$ which would represent a very low rate of water absorption for a laboratory concrete. The low values of sorptivity recorded were thought to be the result of surface effects disrupting the water absorption process. Changes to the concrete surface result from local exposure conditions which makes comparisons of sorptivity values for different structures difficult.

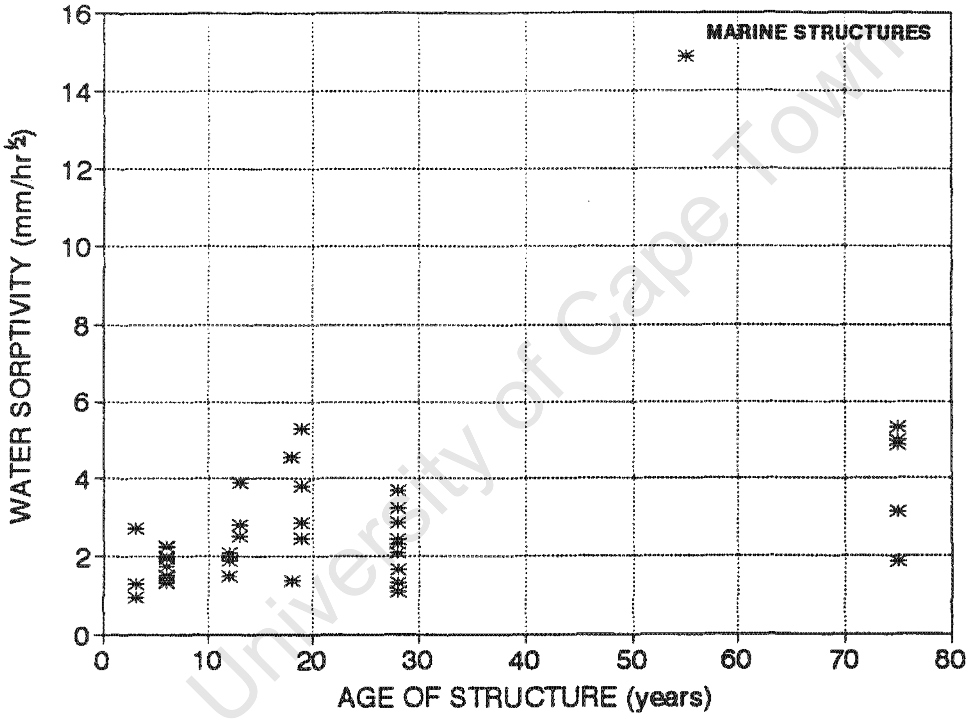

Figure 5.24a: Water Sorptivity vs Age of Structure

Water sorptivity results showed little difference with increasing age as can be seen in Figure 5.24a. Results from this investigation indicate that sorptivity testing of marine concrete was sensitive to environmental effects and testing should preferably be done before exposure to the marine environment. Once concrete is exposed to seawater, changes occur at the concrete surface and in the covercrete zone that significantly alter the absorption rate of the material. These changes are difficult to quantify and depend on local conditions making measurements of sorptivity arbitrary and unreliable for use in assessing residual durability.

Initially, sorptivity testing was done over a period of 64 minutes but significant interference occurred during this period so the test period was extended to six hours which produced a better correlation between weight of water absorbed and square root of time. Typical water absorption curves for marine concrete from the East London breakwater are shown in Figure 5.24b. It appears that after an initial unsteady phase, water absorption follows a similar pattern to that established for laboratory concrete. The slope of the curve was determined by a linear regression analysis of data between 1 and 360 minutes unless saturation was achieved before six hours. Regression analysis of the water absorption curves showed a poorer linear fit than that achieved with laboratory concrete.

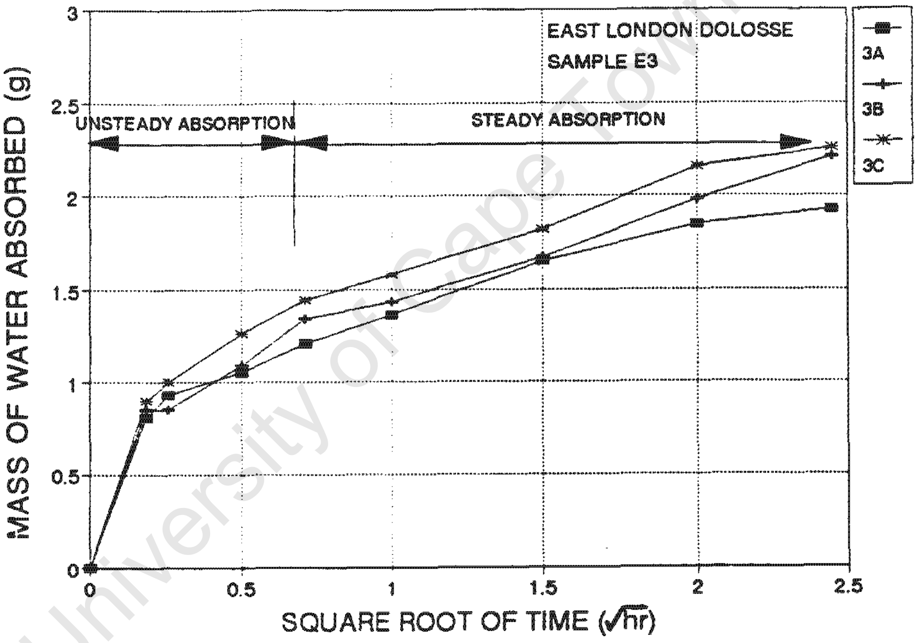

Figure 5.24b: Typical Water Absorption Curves for Marine Concrete

d) Chloride Conductivity

The chloride conductivity test was developed by Streicher midway through this investigation of marine concrete structures and resulted in limited information being available for comparative purposes. Chloride conductivity appeared to be affected by the presence of salt in the concrete pore structure which reduced the apparent diffusivity of the concrete due to chloride binding and pore blocking at the surface [35]. This resulted in virtually all marine samples having chloride conductivity values of less than 1.0 mS/cm which is equivalent to a grade 75 OPC concrete tested at 28 days. A relatively poor quality concrete with high levels of salt contamination resulted in a lower apparent diffusivity of the material. The presence of salt would increase the conductivity of the pore water in situ while restricting the concrete pore structure. Chloride conductivity measured in the laboratory standardizes the pore solution by saturating the sample with a 5M NaCl solution so the presence of a salt contamination would result in a lower chloride conductivity value reflecting only the pore structure changes. The chloride conductivity test was unable to differentiate between material and external influences contributing to the observed chloride conductivity.

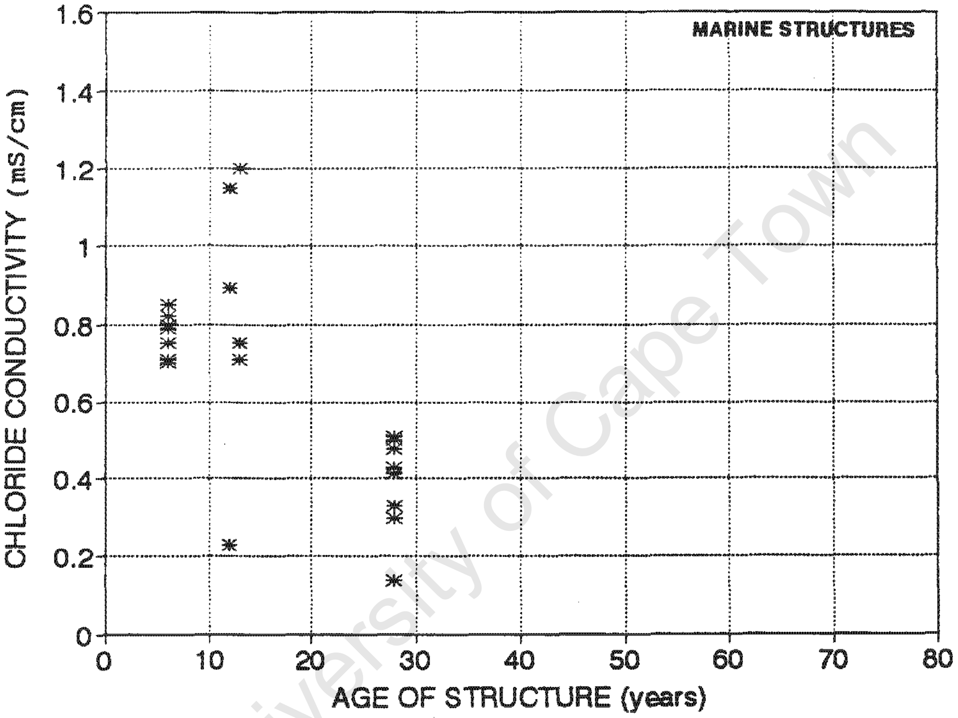

Figure 5.25: Chloride Conductivity vs Age of Structure

Chloride conductivity of concrete extracted from marine concrete structures was found to decrease with increasing age. The observed decrease in chloride conductivity was ascribed to chemical effects such as chloride binding, salt crystallization and pore blocking together with material effects such as continued hydration. The change of chloride conductivity with time is shown in Figure 5.25. If the chloride conductivity test is to be used on mature marine concrete, allowance must be made for chemical interactions between sea water and concrete or the sample must be desalinated before testing so that only the material properties are measured. Other researchers also found that deterioration processes can complicate the interpretation of durability test results taken from concrete in service [37]. Sandberg and Tang observed a reduction of apparent diffusivity of concrete with time of exposure over a four year period for a bridge column [8]. It is recommended that chloride conductivity testing of concrete be done before exposure to the marine environment to eliminate environmental effects.

e) Abrasion Resistance

Abrasion resistance testing of marine concrete structures provided useful data about the physical resistance of concrete. Results were affected by the amount of deterioration which had occurred particularly with regard to abrasion damage which effectively removed poor quality surface layers exposing internal concrete. Fly ash and slag concrete structures tended to have lower abrasion resistance than OPC concrete and observations from site confirmed that the abrasion performance of these concretes was poor. It should be noted that the recorded core strengths from Strandfontein and East London were low (36 and 37 MPa) which could have been a contributory factor. Abrasion resistance was found to be slightly dependent on the strength of concrete and Figure 5.26 shows the correlation between abrasion resistance and core strength.

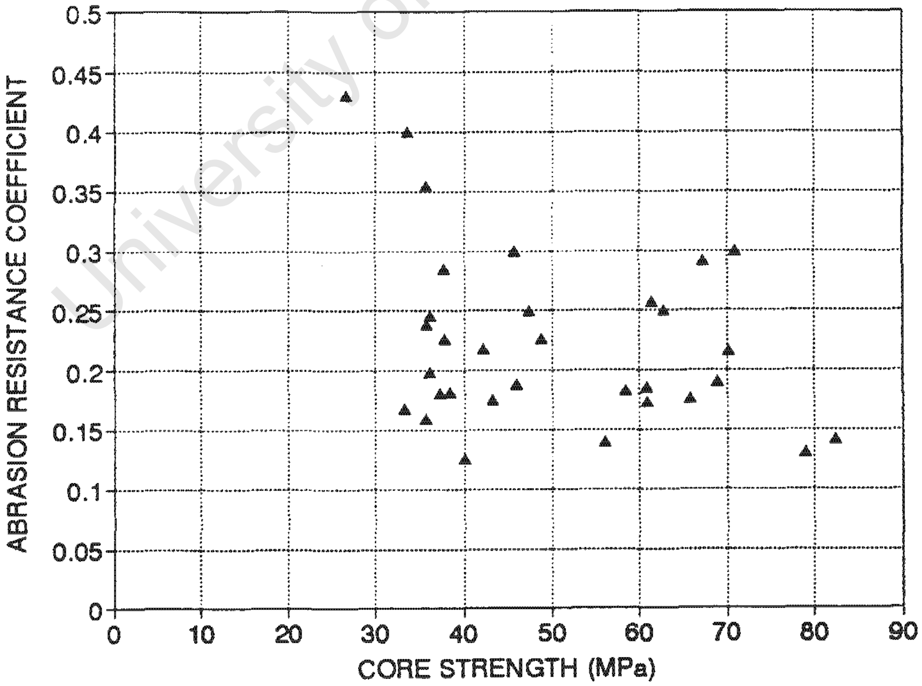

Figure 5.26: Abrasion Resistance Coefficient vs Core Strength

f) Carbonation Depth

Carbonation depths recorded for marine concrete were low due to the saturated nature of the concrete being exposed to regular tidal or splash action and with ambient humidity values seldom less than 80 % R.H. All structures had carbonation depths of less than 10 mm except for interior concrete at the Sea Point Aquarium (30 to 70 mm) and the Sea Point Toilet Block (24 to 59 mm) due to the less saturated nature of the concrete. It should be noted that the concrete used in the aquarium and toilet block was poor quality material of grade 25 MPa. Figure 5.27 shows the carbonation depths plotted against age of structure. Carbonation depths of less than 7 mm were recorded at Simonstown Jetty in a poor quality concrete exposed for 75 years. Such low depths indicate that carbonation is not a major deterioration mechanism for most marine structures exposed to humid or saturated conditions.

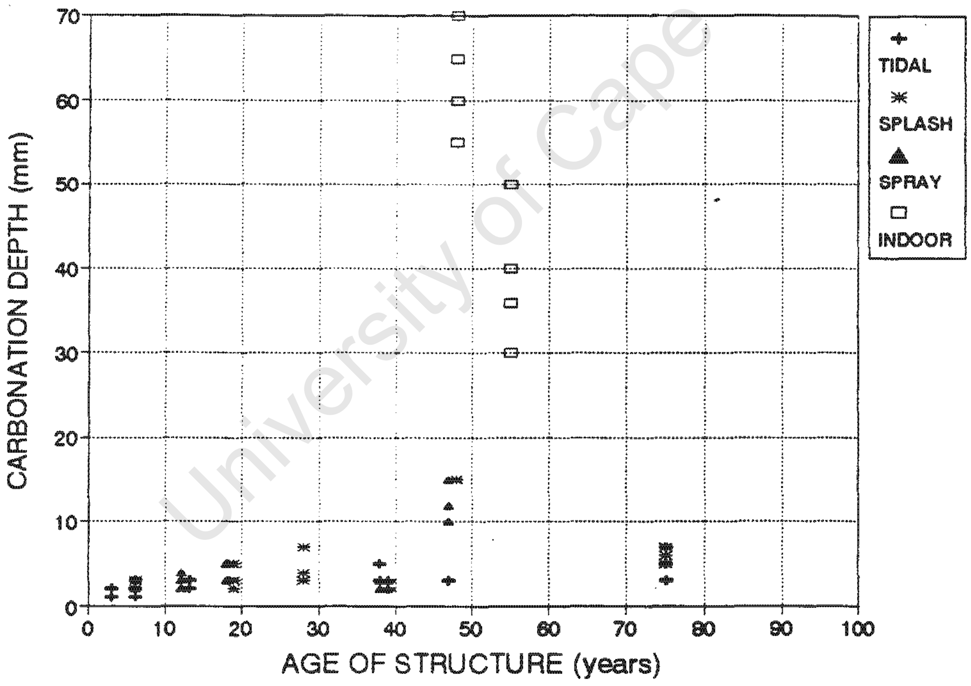

Figure 5.27: Carbonation Depth vs Age of Structure

g) Chloride Content

Chloride contents increased progressively with age and older structures had particularly high chloride levels which might have been partly due to corrosion damage allowing faster rates of chloride ingress through cracks in the concrete. Figure 5.28a shows the chloride contents at 60 mm (normal marine cover depth) for the structures tested. All structures over twenty years old were found to have dangerously high chloride levels at a depth of 60 mm and chloride contents were more variable which was probably due to the extent of damage and poor quality of the concrete. The discrepancy of chloride contents at Simonstown Jetty (75 years) was also due to the large difference in exposure conditions between tidal and splash zones.

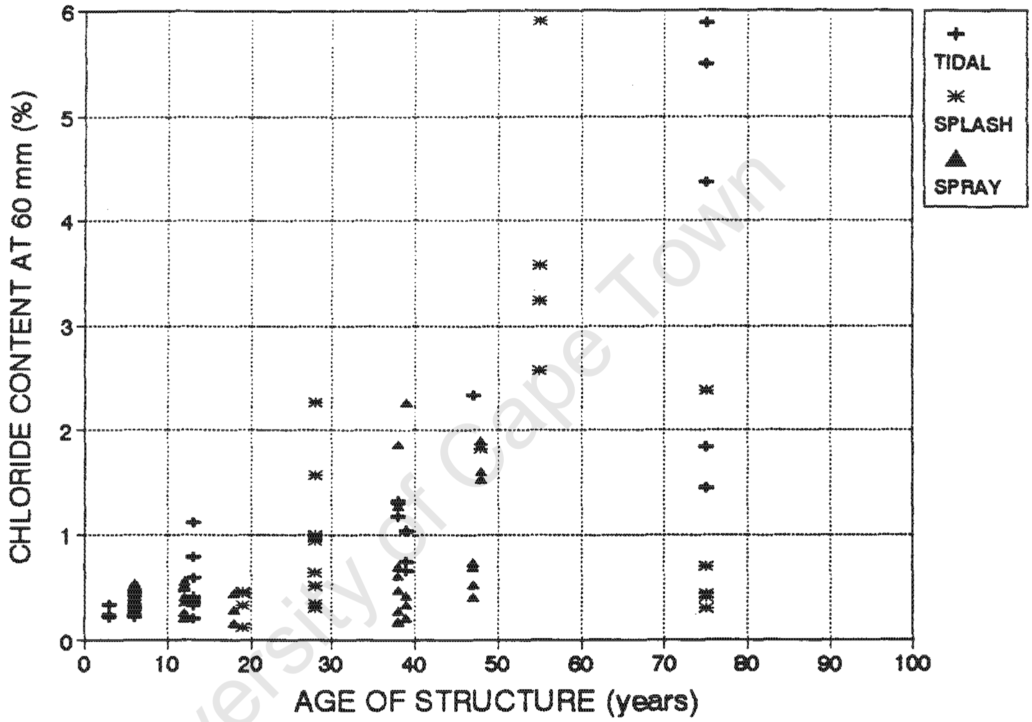

Figure 5.28a: Chloride Content at 60 mm versus Age of Structure

A trend was found between average chloride levels at the reinforcement and the amount of corrosion damage evident on each structure as determined by the corrosion rating (Figure 5.28b). Structures investigated with average chloride contents at the rebar of greater than 1.0 % by mass of cement had active corrosion with major damage from cracking and spalling while structures with chloride contents at the rebar of less than 0.4 % had no evidence of corrosion. This correlates well with data published by other researchers and conforms with the general qualitative risk of corrosion outlined in section 5.3 [38]. The results confirm the premise that chloride penetration rates may be used as an indicator of durability performance for reinforced concrete structures.

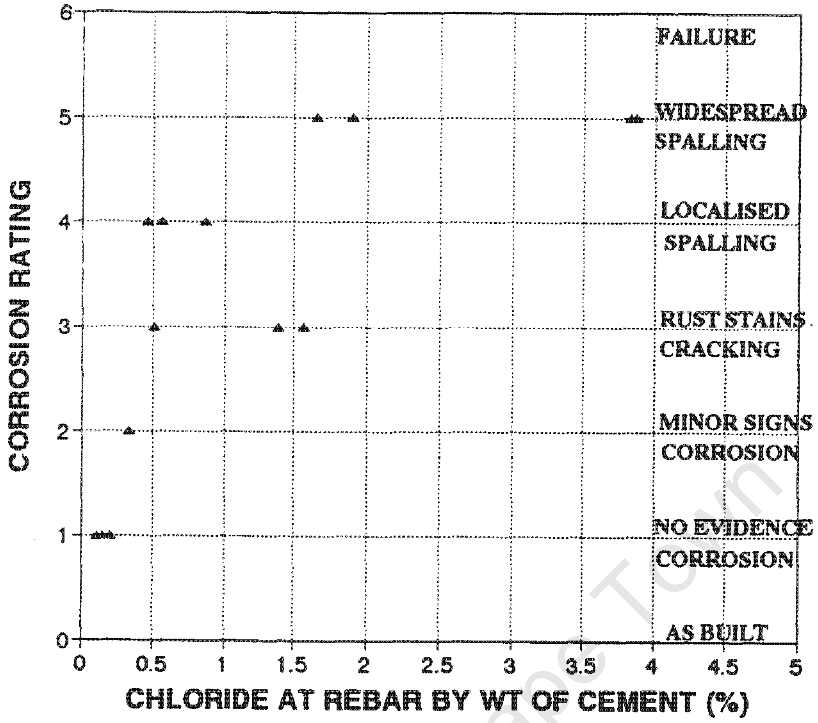

Figure 5.28b: Corrosion Rating versus Average Chloride at Reinforcement

Chloride diffusion coefficients appeared to decrease with the length of exposure in the marine environment as shown in Figure 5.28c. A general trend was observed in the reduction of diffusion coefficients with time despite the range of exposure conditions, materials and structures being compared. This has been observed by other researchers and is due to mechanisms such as chloride binding, pore blocking and continued cement hydration which constrict the pores in the concrete [39]. The trend of decreasing chloride diffusion coefficients with age was observed even though modern structures generally had higher specifications in terms of original concrete grade than older structures. The reduction in diffusion coefficient has implications on the correct application of Fick's Law for predicting long-term chloride contents in concrete. It would appear that chloride diffusion coefficients stabilised after ten to twenty years of marine exposure. Older structures often had misleading diffusion coefficients due to high chloride concentrations in the concrete. Chloride concentrations in some cases reached saturation levels near the surface, effectively blocking further chloride penetration and the suitability of using Fick's Law to describe the diffusion process under these conditions becomes questionable.

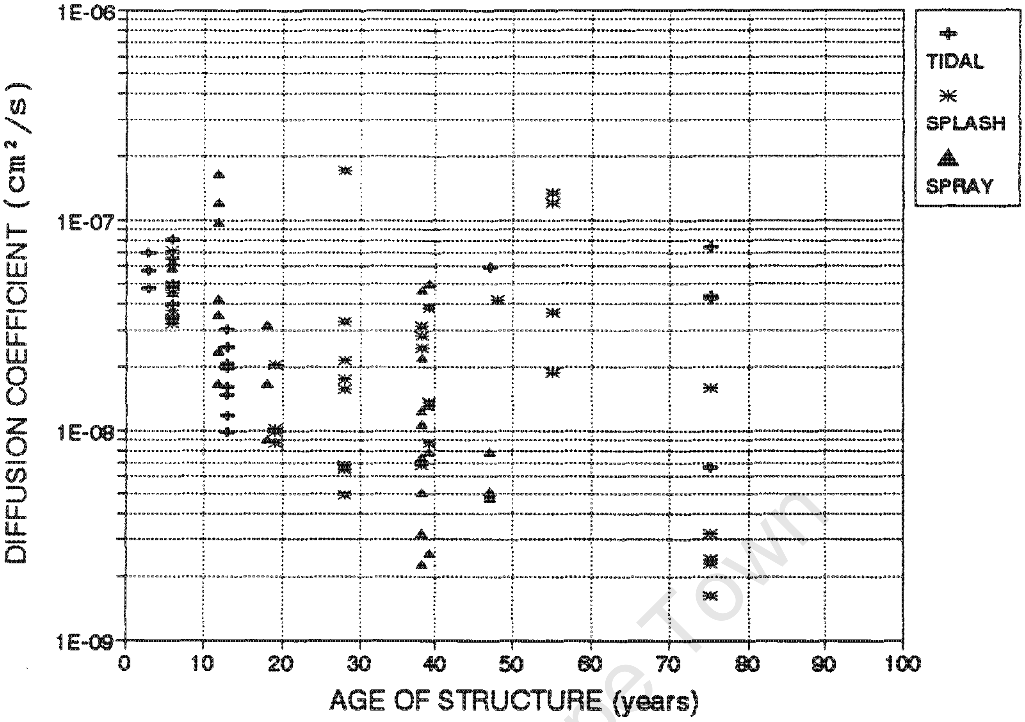

Figure 5.28c: Diffusion Coefficient vs Age of Structure

A reasonable correlation between diffusion coefficient and chloride conductivity was expected given the nature of the two variables, and the relationship is shown in Figure 5.28d. The rate of chloride penetration into concrete is theoretically related to the chloride conductivity and a trend was found between the two parameters. The poor correlation between the two diffusion parameters was due to several variables which affected the rate of chloride penetration such as different environmental and service conditions, extent of deterioration of the concrete and the different ages of structures. A comparison of diffusion coefficients with core strength produced a worse correlation which confirms that strength is a poor indicator of durability for marine concrete (Figure 5.28e).

Fly ash and slag concrete were found to be more resistant to chloride penetration than similar OPC concrete. The average diffusion coefficient for fly ash concrete from the Strandfontein tidal pool wall was 1.8E-8 cm²/s, slag concrete from dolosse on the East London breakwater was 9.4E-9 cm²/s while most OPC concrete structures had diffusion coefficients between 2.0E-8 and 7.5E-8 cm²/s. This confirms trends observed from short-term marine exposure results at Simonstown and Granger Bay presented in Chapter Four and correlates well with published research [40].

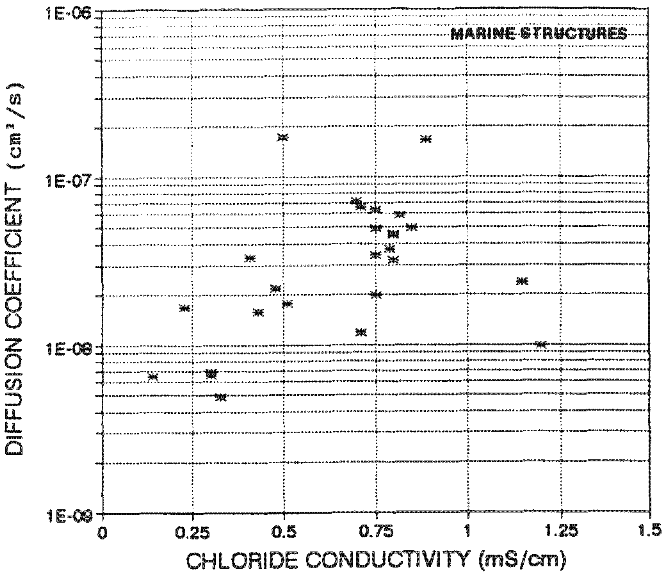

Figure 5.28d: Diffusion Coefficient vs Chloride Conductivity

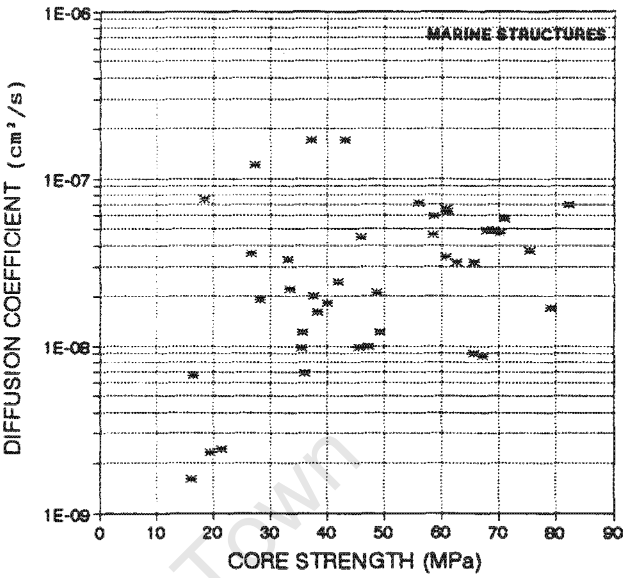

Figure 5.28e: Diffusion Coefficient vs Core Strength

5.5 Recommendations

Several practical recommendations were formulated from observations from the case studies which are discussed in more detail below :-

5.5.1 Durability Audits During Construction

The durability of several structures investigated was severely compromised by design and construction faults such as inadequate cover, plastic cracking and poor compaction with little or no attention to curing being likely. These faults allowed rapid deterioration to occur and led to premature failure or reduced service lives in some cases. All concrete structures should be inspected after construction and provision made in construction contracts to allow for immediate remedial work in areas of poor quality construction. Cover depths should be checked as a matter of course and appropriate action taken in areas of low cover.

Poor compaction of concrete and inadequate control of cover to reinforcement depths were found on most of the structures investigated. Figure 5.29a shows the minimum cover depths measured on site plotted against specified minimum cover depths. Concrete core samples were visually rated for compaction rating into either excellent, good, fair or poor categories depending on the amount of voids and large pores. Figure 5.29b shows the result of the compaction rating indicating that inadequate compaction of concrete is commonplace during construction. The results of poor compaction are often not detected by casual examination of the hardened concrete surface.

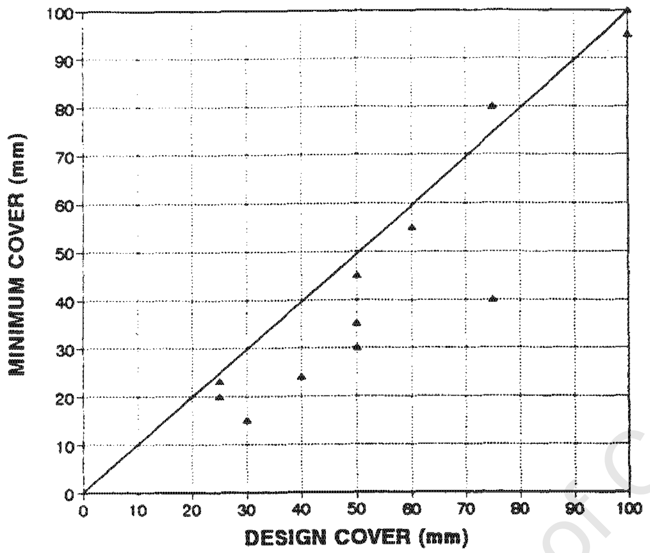

Figure 5.29a: Minimum vs Design Cover

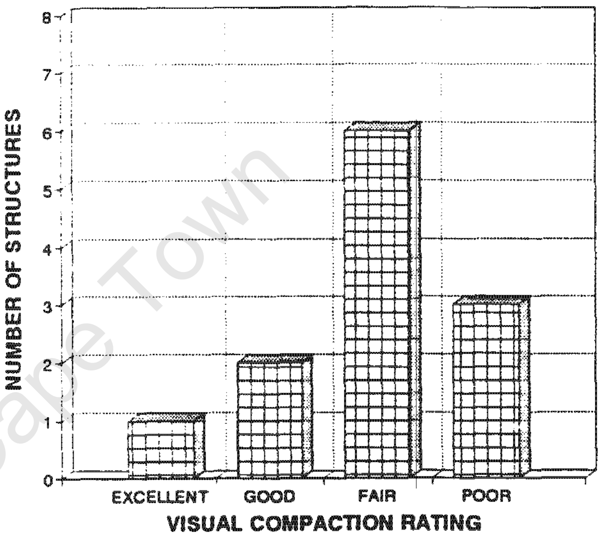

Figure 5.29b: Visual Compaction Rating

5.5.2 Assessing the Residual Durability

The residual durability of concrete structures needs to be assessed so that appropriate maintenance and future capital expenditure can be planned. The three durability index tests (oxygen permeability, water sorptivity and chloride conductivity) were not reliable indicators of residual durability of the covercrete zone. Measurements of the resistance of the covercrete to deterioration are complicated after marine exposure by chemical and physical interactions between the concrete and the environment. The durability index tests were generally unable to differentiate between material properties and pore structure changes caused by deterioration. These effects caused the resistance of the concrete to be over-estimated in most cases. Determining the residual durability is best done using standard methods which measure the level of deterioration such as chloride ingress, carbonation or reinforcement corrosion. Once the mechanisms, extent and implications of deterioration have been assessed a rational maintenance programme may be implemented.

Chloride ingress measurements appear to give a reliable estimation of the corrosion risk of the reinforcement and can be determined quickly and accurately using laboratory techniques. The rate of chloride ingress into marine concrete structures was found to be the only reliable indicator of concrete deterioration and suitable methods of predicting chloride levels will help estimate the service life of reinforced concrete structures.

5.5.3 Durability Specifications

Durability index tests have an important place in measuring potential durability of concrete structures but should preferably be used during initial planning and construction. More reliable information about the comparative durability performance of the structures investigated could have been obtained if the original concrete had been suitably characterized during construction. Records need to be maintained of new marine concrete structures so that the durability performance in service can be accurately assessed in the future.

5.6 Conclusions

The investigation of marine concrete structures was successful in terms of being able to quantify the rate and type of deterioration mechanisms which occur on site concrete. The study was not able however to determine the original durability properties of the concrete and therefore make predictions of durability based solely on case study data. Results obtained from this investigation will however be used to validate the predictive model for determining long-term chloride levels in marine concrete which is developed in Chapter Seven. Useful information was gained about the comparative performance of different concrete types in the marine environment and the effect of local variations of marine exposure and service conditions.

A comparative assessment of the durability performance of a marine structure can only be made once the severity of exposure and original concrete properties have been accurately assessed. Neither of these factors could be reliably assessed during investigations which made comparisons between different structures difficult. An estimation of the severity of exposure could be made from data gathered during short-term concrete exposure work detailed in Chapter Four but more research needs to be done in this important area. Information about the original construction was less reliable for structures built before 1960. Insufficient records of older structures limited the conclusions that could be drawn about long-term characteristics of the material.

Durable concrete structures generally had high cover depths with a dense well compacted concrete producing low diffusion coefficients. Several structures such as the Kogel Bay and Strandfontein tidal pools and the Camps Bay Pumpstation should last at least fifty years with out major repairs being required. Other structures such as Koeberg Pumphouse and Oudekraal Retaining Wall have already exhibited premature deterioration and will require extensive repairs to achieve their expected design lives. The poor durability performance of many structures could have been avoided with little additional expense using good design and construction practice. Cement extenders such as slag and fly ash are particularly effective in reducing chloride penetration and are recommended for use in reinforced concrete structures exposed to marine conditions.

5.7 References

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5.8 Appendices

SITE NAME	SAMPLE No.	UPV (m/s)	RESIST-IVITY (kOhmcm	COVER DEPTH (mm)	CARBON-ATION (mm)	LOCATION
KOGEL BAY TIDAL POOL (3 years)	G1	4780	60	100	2	Tidal
	G2	4890	37	100	2	Tidal
	G3	4590	45	110	1	Tidal
TABLE BAY BREAKWATER (6 years)	D1	4320	48	-	2	Splash
	D2	4430	30	-	2	Splash
	D3	4500	64	-	3	Splash
	D4	4555	22	-	2	Tidal
	D5	4635	29	-	2	Tidal
	D6	4625	27	-	3	Tidal
	D7	4470	30	-	3	Splash
	D8	4335	32	-	2	Splash
	D9	4500	22	-	2	Splash
	D10	4370	22	-	1	Tidal
	D11	4510	18	-	1	Tidal
	D12	4475	23	-	2	Tidal
	D13	4500	44	-	3	Spray
	D14	4315	46	-	2	Spray
	D15	4405	40	-	2	Spray
	D16	4345	35	-	2	Spray
KOEBERG PUMPHOUSE (12 years)	K1	-	24	60	3	Spray
	K2	-	7	23	3	Spray
	K3	-	21	39	-	Spray
	K4	-	6	50	2	Spray
	K5	-	33	61	4	Spray
	K6	-	23	65	-	Spray
	K7	-	15	40	-	Spray

Table 5A1: Insitu Results - Sheet 1

SITE NAME	SAMPLE No.	UPV (m/s)	RESIST-IVITY (kOhmcm	COVER DEPTH (mm)	CARBON-ATION (mm)	LOCATION
STRAND-FONTEIN TIDAL POOL (13 years)	S1	4380	25	100	2	Tidal
	S2	4390	28	150	2	Tidal
	S3	4425	20	120	3	Submerged
	S4	4590	20	115	3	Tidal
	S5	4570	17	125	2	Tidal
	S6	4500	24	110	2	Tidal
	S7	4455	28	135	3	Tidal
	S8	4475	15	120	2	Tidal
CAMPS BAY PUMPSTATION (18 years)	C1	-	-	100	3	Splash
	C2	-	-	90	3	Splash
	C3	-	-	85	5	Splash
OUDEKRAAL RETAINING WALL (19 years)	O1	-	58	65	2	Splash
	O2	-	57	70	3	Splash
	O3	-	28	75	5	Splash
	O4	-	19	75	3	Spray
EAST LONDON BREAKWATER (28 years) E1-E4 OPC E5-E9 SLAG	E1	-	-	-	3	Splash
	E2	-	-	-	3	Splash
	E3	-	-	-	4	Splash
	E4	-	-	-	4	Splash
	E5	-	-	-	7	Splash
	E6	-	-	-	7	Splash
	E7	-	-	-	3	Splash
	E8	-	-	-	3	Splash
	E9	-	-	-	3	Splash

Table 5A2: Insitu Results - Sheet 2

SITE NAME	SAMPLE No.	UPV (m/s)	RESISTIVITY (kOhmcm)	COVER DEPTH (mm)	CARBONATION (mm)	LOCATION
MUIZENBERG BRIDGES (38 years)	M3	-	-	60	3	Splash
	M6	-	-	55	3	Splash
	M9	-	-	65	5	Splash
	M10	-	-	60	2	Splash
	Z2	-	-	65	2	Splash
	Z3	-	-	70	3	Splash
	Z5	-	-	60	2	Splash
WILDERNESS BRIDGES (46 years)	W1	-	-	30	10	Spray
	W2	-	-	85	12	Spray
	W3	-	-	200	3	Splash
	L1	-	-	34	12	Spray
	L2	-	-	33	15	Spray
SEA POINT TOILET BLOCK (48 years)	T1	-	-	34	70	Spray
	T2	-	-	49	65	Spray
	T3	-	-	33	55	Spray
	T4	-	-	35	70	Spray
	T5	-	-	59	15	Splash
SEA POINT AQUARIUM (55 years)	Q1	-		50	40	Splash
	Q2	-		40	30	Splash
	Q3	-		50	50	Splash
SIMONSTOWN JETTY (75 years)	J1	4370	8	60	3	Tidal
	J2	3710	12	60	5	Tidal
	J3	3865	17	65	3	Tidal
	J6	3775	36	55	7	Splash
	J7	3990	27	60	5	Splash
	J8	2985	19	60	6	Splash

Table 5A3: Insitu Results - Sheet 3

SITE NAME	SAMPLE No.	CORE fcu (MPa)	OXYGEN PERM. INDEX	SORP-TIVITY (mm/ hr)	CHLORIDE CONDUCT (mS/cm)	ABRASION RESIST. COEFF.
KOGEL BAY TIDAL POOL (3 years)						
	G1	82.4	10.75	2.73		0.141
	G2	70.1	10.38	1.30		0.215
	G3	71.0	10.33	0.96		0.300
TABLE BAY BREAKWATER (6 years)						
	D1	69.0	9.96	1.53	0.75	0.190
	D2	65.8	9.85	1.32	0.80	-
	D3	75.5	9.78	2.02	0.79	-
	D4	-	-	-	-	-
	D5	-	-	-	-	-
	D6	-	-	-	-	-
	D7	67.9	10.05	1.35	0.85	-
	D8	46.0	9.75	1.49	0.80	0.187
	D9	56.1	10.24	1.76	0.70	0.139
	D10	60.9	10.28	1.95	0.71	0.172
	D11	-	-	-	-	-
	D12	-	-	-	-	-
	D13	61.3	9.51	2.24	0.75	0.257
	D14	60.9	9.70	2.25	0.75	0.185
	D15	58.5	9.64	2.24	0.80	0.182
	D16	58.6	9.69	1.35	0.82	-
KOEBERG PUMPHOUSE (12 years)						
	K1	79.1	10.38	2.10	0.23	0.130
	K2	49.3	-	-	-	-
	K3	-	-	-	-	-
	K4	43.2	9.41	1.90	0.89	0.175
	K5	42.1	9.24	1.50	1.15	0.217
	K6	-	-	-	-	-
	K7	-	-	-	-	-

Table 5A4: Durability Test Results - Sheet 1

SITE NAME	SAMPLE No.	CORE fcu (MPa)	OXYGEN PERM. INDEX	SORPTIVITY (mm/ hr)	CHLORIDE CONDUCT (mS/cm)	ABRASION RESIST. COEFF.
STRANDFONTEIN TIDAL POOL (13 years)	S1	37.6	9.95	2.55	0.75	0.285
	S2	35.6	10.09	3.91	1.20	0.354
	S3	35.7	9.61	2.83	0.71	0.238
	S4	-	-	-	-	-
	S5	-	-	-	-	-
	S6	-	-	-	-	-
	S7	-	-	-	-	-
	S8	-	-	-	-	-
CAMPS BAY PUMPSTATION (18 years)	C1	65.7	9.69	1.38	-	0.176
	C2	-	-	-	-	-
	C3	62.8	7.97	4.57	-	0.249
OUDEKRAAL RETAINING WALL (19 years)	O1	47.4	9.43	2.88	-	0.249
	O2	45.7	8.60	3.79	-	0.299
	O3	48.7	8.42	2.49	-	0.226
	O4	67.3	8.49	5.30	-	0.291
EAST LONDON BREAKWATER (28 years)	E1	40.0	8.51	1.33	0.51	0.125
	E2	37.2	8.03	2.31	0.50	0.180
	E3	38.3	9.22	1.10	0.43	0.181
	E4	33.2	8.12	2.42	0.41	0.167
	E5	33.5	8.54	3.25	0.48	0.400
	E6	36.0	8.74	2.89	0.30	0.245
	E7	35.6	9.07	3.69	0.33	0.158
	E8	37.7	8.90	2.09	0.30	0.226
	E9	36.0	9.25	1.67	0.14	0.198

Table 5A5: Durability Test Results - Sheet 2

SITE NAME	SAMPLE No.	CORE fcu (MPa)	OXYGEN PERM. INDEX	SORP-TIVITY (mm/ hr)	CHLORIDE CONDUCT. (mS/cm)	ABRASION RESIST. COEFF.
MUIZENBERG BRIDGES (38 years)	M3	41.0	-	-	-	-
	M6	37.7	-	-	-	-
	M9	-	-	-	-	-
	M10	-	-	-	-	-
	Z2	35.6	-	-	-	-
	Z3	34.3	-	-	-	-
	Z5	-	-	-	-	-
WILDERNESS BRIDGES (46 years)	W1	-	-	-	-	-
	W2	-	-	-	-	-
	W3	-	-	-	-	-
	L1	-	-	-	-	-
	L2	-	-	-	-	-
SEA POINT TOILET BLOCK (48 years)	T1	-	-	-	-	-
	T2	-	-	-	-	-
	T3	-	-	-	-	-
	T4	-	-	-	-	-
	T5	-	-	-	-	-
SEA POINT AQUARIUM (55 years)	Q1	26.7	8.10	14.90	-	0.430
	Q2	28.1	-	-	-	-
	Q3	27.2	-	-	-	-
SIMONSTOWN JETTY (75 years)	J1	16.6	8.75	4.98	-	-
	J2	18.5	8.49	5.31	-	-
	J3	-	-	-	-	-
	J6	19.4	9.72	3.15	-	-
	J7	16.3	9.28	1.86	-	-
	J8	21.5	10.36	4.87	-	-

Table 5A6: Durability Test Results - Sheet 3

NAME	SAMPLE	TOTAL CHLORIDE CONTENT WITH DEPTH				Dc	Cs
	No.	5 - 25	25 - 50	50 - 75	75 - 100	(cm ^ 2/s)	(% m/m)
KOGEL BAY (3 years)	K1	1.33	0.57	0.34	0.22	7.01E-08	1.83
	K2	1.66	0.70	0.21	0.08	4.72E-08	2.56
	K3	1.85	0.90	0.23	0.14	5.71E-08	2.76
	AVG					5.81E-08	2.38
TABLE BAY BREAKWATER (6 years)	D1	1.29	0.70	0.38	0.21	4.87E-08	1.70
	D2	1.11	0.60	0.24	0.15	3.23E-08	1.65
	D3	1.48	0.80	0.33	0.14	3.73E-08	2.10
	D4	1.45	0.71	0.44	0.23	4.96E-08	1.85
	D5	1.25	0.64	0.30	0.16	3.48E-08	1.80
	D6	1.23	0.84	0.47	0.32	8.13E-08	1.55
	D7	1.05	0.63	0.28	0.14	4.94E-08	1.35
	D8	0.98	0.54	0.27	0.11	4.52E-08	1.25
	D9	1.11	0.74	0.39	0.19	7.17E-08	1.35
	D10	1.03	0.71	0.42	0.16	6.63E-08	1.35
	D11	0.98	0.51	0.21	0.13	4.03E-08	1.25
	D13	1.57	0.94	0.54	0.27	6.40E-08	2.00
	D14	1.51	0.67	0.39	0.22	3.43E-08	2.15
	D15	1.16	0.65	0.38	0.16	4.55E-08	1.60
	D16	0.92	0.54	0.33	0.17	5.87E-08	1.15
	AVG					5.06E-08	1.61
KOEBERG PUMPHOUSE (12 years)	K1	1.28	0.56	0.20	0.02	1.68E-08	1.75
	K2	0.60	0.58	0.56	0.28	1.20E-07	0.85
	K3	0.53	0.31	0.25	0.26	4.22E-08	0.75
	K4	0.76	0.46	0.51	0.40	1.66E-07	0.80
	K5	1.53	0.64	0.49	0.33	2.37E-08	2.10
	K6	1.46	0.91	0.41	0.32	3.57E-08	1.85
	K7	0.53	0.46	0.36	0.19	9.73E-08	0.65
	AVG					7.17E-08	1.25
STRAND-FONTEIN TIDAL POOL (13 years)	S1	2.49	1.42	0.41	0.22	1.98E-08	3.50
	S2	2.82	0.93	0.32	0.24	9.84E-09	4.50
	S3	1.46	0.57	0.20	0.21	1.17E-08	2.25
	S4	4.33	1.98	0.78	0.28	1.63E-08	6.25
	S5	3.84	2.09	0.59	0.47	2.48E-08	4.75
	S6	3.15	1.88	0.38	0.25	2.08E-08	4.25
	S7	3.09	1.44	0.36	0.20	1.49E-08	4.50
	S8	3.54	2.39	1.12	0.56	3.04E-08	4.90
	AVG					1.86E-08	4.36

Table 5A7: Chloride Content Results - Sheet 1

NAME	SAMPLE No.	TOTAL CHLORIDE CONTENT WITH DEPTH				Dc	Cs
		5 - 25	25 - 50	50 - 75	75 - 100	(cm ^ 2/s)	(% m/m)
CAMPS BAY PUMPSTATION (18 years)	C1	0.70	0.24	0.15	0.09	9.04E-09	1.00
	C2	1.03	0.56	0.28	0.10	1.68E-08	1.35
	C3	1.02	0.90	0.44	0.15	3.21E-08	1.35
	AVG					1.93E-08	1.23
OUDEKRAAL RETAINING WALL (19 years)	O1	2.95	1.29	0.46	0.15	1.02E-08	4.25
	O2	2.39	0.80	0.45	0.06	9.84E-09	3.25
	O3	1.02	0.67	0.33	0.15	2.06E-08	1.35
	O4	0.61	0.21	0.12	0.10	8.72E-08	0.85
	AVG					3.20E-08	2.43
EAST LONDON BREAKWATER (28 years)	E1	2.43	1.78	0.98	0.45	1.77E-08	3.20
	E2	3.06	2.57	2.26	2.05	1.73E-07	3.25
	E3	2.99	1.99	0.94	0.50	1.59E-08	3.75
	E4	2.65	2.18	1.57	0.71	3.33E-08	3.25
	E5	2.62	1.76	1.00	0.83	2.16E-08	3.25
	E6	2.46	1.13	0.35	0.23	6.85E-09	3.50
	E7	3.52	1.37	0.3	0.25	4.89E-09	5.53
	E8	5.19	2.46	0.51	0.36	6.57E-09	7.65
	E9	6.78	2.98	0.64	0.35	6.53E-09	9.75
						3.18E-08	4.79
MUIZENBERG BRIDGES - ROYAL ROAD BRIDGE (38 years)	M1	2.74	1.75	0.69	0.67	1.07E-08	3.53
	M2	2.46	0.82	0.61	0.40	5.05E-09	3.42
	M3	2.85	2.15	1.30	1.07	2.46E-08	3.35
	M4	1.08	0.34	0.17	0.10	2.29E-09	1.88
	M5	0.78	0.41	0.27	0.33	1.23E-08	0.91
	M6	2.56	1.63	1.33	1.01	2.79E-08	2.85
	M7	1.80	0.94	0.47	0.31	7.50E-09	2.40
	M8	2.28	0.87	0.16	0.12	3.23E-09	3.65
	M9	2.40	1.92	1.31	0.98	3.15E-08	2.80
	M10	4.90	1.98	1.18	0.86	6.83E-09	6.52
	M11	3.88	3.23	1.87	1.12	2.21E-08	4.80
	M12	1.69	2.43	1.27	0.92	4.63E-08	2.50
	AVG					1.67E-08	3.22

Table 5A8: Chloride Content Results - Sheet 2

NAME	SAMPLE No.	TOTAL CHLORIDE CONTENT WITH DEPTH				Dc	Cs
		5 - 25	25 - 50	50 - 75	75 - 100	(cm ^ 2/s)	(% m/m)
MUIZENBERG BRIDGES - ZEEKOEVLEI BRIDGE (38 years)	Z1	3.74	2.17	2.26	1.74	4.97E-08	3.80
	Z2	1.54	1.09	1.04	0.57	3.82E-08	1.72
	Z3	2.03	1.26	0.74	0.54	1.36E-08	2.48
	Z4	1.57	0.50	0.20	0.13	2.59E-09	2.65
	Z5	2.35	1.34	0.66	0.34	8.69E-09	3.08
	Z8	0.78	0.64	0.42	0.49	4.97E-08	0.85
	Z9	0.64	0.40	0.33	0.33	1.31E-08	0.88
	Z10	3.49	1.55	1.06	0.49	7.92E-09	4.53
	AVG					2.29E-08	2.50
WILDERNESS BRIDGES (46 years)	W1	2.06	1.23	0.41	0.19	4.76E-09	3.10
	W2	2.43	1.73	0.73	0.49	7.88E-09	3.45
	W3	4.02	2.40	2.35	2.30	5.98E-08	3.90
	L1	1.52	1.20	0.69	0.25	1.35E-08	1.95
	L2	1.51	1.28	0.53	0.20	1.08E-08	2.04
	AVG					1.93E-08	2.89
SEA POINT TOILET BLOCK (48 years)	T1	1.27	1.54	1.52	1.46	-	-
	T2	1.32	1.80	1.85	1.23	-	-
	T3	1.70	2.05	1.90	1.16	-	-
	T4	1.47	1.54	1.60	1.49	-	-
	T5	3.07	2.24	1.83	1.65	4.20E-08	3.32
	AVG						
SEA POINT AQUARIUM (55 years)	Q1	4.21	3.26	2.57	2.28	3.63E-08	4.75
	Q2	6.25	6.23	3.57	1.65	1.89E-08	8.00
	Q3	3.35	3.84	3.24	3.13	1.20E-07	4.50
	AVG					5.84E-08	5.75
SIMONSTOWN JETTY (75 years)	J1	4.90	3.18	1.85	0.68	6.73E-09	6.00
	J2	5.41	5.95	5.88	5.15	7.50E-08	6.00
	J3	7.22	5.83	5.50	4.27	4.36E-08	8.00
	J4	4.35	2.40	1.45	0.43	4.24E-08	6.00
	J5	4.19	4.27	4.37	3.95	1.13E-06	4.50
	J6	2.45	1.02	0.30	0.27	2.33E-08	3.50
	J7	3.04	0.75	0.41	0.29	1.64E-09	4.50
	J8	2.69	1.22	0.44	0.31	2.44E-09	4.00
	J9	2.45	1.18	0.70	0.28	3.22E-09	3.50
	J10	4.30	2.71	2.39	2.10	1.61E-08	5.00
	AVG					1.34E-07	5.10

Table 5A9: Chloride Content Results - Sheet 3

CHAPTER SIX

LABORATORY BASED DURABILITY TESTING OF MARINE CONCRETE

6.1 Literature Survey

Comparative indications of durability may be produced using accelerated laboratory testing of concrete but the approach must be related to long-term characteristics of the material to be reliable. Methods of accelerating deterioration may change the material response fundamentally causing erroneous results which have little relevance to the behaviour of concrete in service. Accelerated techniques are useful however as rapid testing can be done allowing materials and design to be optimized for a particular environment which could take years to replicate under natural exposure conditions. These accelerated tests for concrete therefore have a place in providing useful information which can complement data from case studies and field exposure results.

A number of laboratory investigations were undertaken to study specific aspects of the durability performance of concrete under simulated marine conditions. Topics investigated included the protection of concrete using sealants and coatings, corrosion of reinforcement in concrete, electrochemical chloride extraction and surface effects of seawater attack on concrete. The information acquired from these tests was used to complement the findings of marine exposure of concrete and case studies of marine structures reported in Chapter Four and Five respectively.

a) Barrier protection of concrete

Protection of concrete using barrier systems is becoming increasingly popular due to the poor performance of many reinforced concrete structures especially under severe conditions. Several conferences and committees have dealt extensively with the practical utilisation of sealants and coatings to protect concrete from deterioration [1,2,3,4]. Barrier systems for concrete may be broadly classified as being either internal or external treatments and the most appropriate system will depend on the type of performance required and the condition of the concrete to be protected. Internal treatments of concrete include penetrants, water repellant pore liners and sealers, external treatments include coatings, waterproofers, renders, overlays and claddings [5].

Penetrants, such as polymers impregnated to considerable depth into the concrete, are low viscosity monomers which are polymerised in situ by heating or using retarded catalysts. The process ensures that the pore structure is effectively sealed thereby reducing the concrete permeability and increasing the tensile strength of the material. Polymer impregnation is reported to increase the corrosion and chemical resistance of concrete and reduces the permeability but the method is expensive, requires expert application and has low fire resistance [6].

Pore liners such as silanes and siloxanes are low viscosity liquids which penetrate up to 5 mm into the concrete and line the pores with a hydrophobic layer. The material does not block the pores of the concrete allowing water vapour transmission but the hydrophobic lining of the pores repels water molecules thereby reducing absorption. The sealant system therefore allows drying to occur from water vapour diffusion but reduces the uptake of water so that it should theoretically be possible to reduce the moisture content of concrete by using a pore lining sealant such as a silane. Blight was not able to confirm this premise for ASR damaged structures in a relatively dry highveld environment but Brown showed that for a marine structure silane coating reduced the risk of corrosion as measured with half-cell potentials [7,8]. Experimental findings appear to show that pore lining sealants reduce the ingress of aggressive agents but there is insufficient evidence to suggest that sealants reduce the reinforcement corrosion in chloride contaminated concrete by reducing the moisture content substantially [9,10].

Sealers are fairly viscous liquids which penetrate into the outer skin of concrete, blocking the pores by forming a film on the concrete surface but have shown to have limited effectiveness against environmental attack [11]. Coatings form a relatively thick film on the concrete surface (0.3 - 1.0 mm) and are more viscous than sealers resulting in virtually no penetration into the concrete. Coatings are generally fairly impermeable but are vulnerable to surface damage, inadequate adhesion, UV instability and damage from substrate cracking. Renders, overlays and claddings tend to be cement based materials usually thicker than 5 mm and have variable effectiveness in protecting concrete.

Barrier systems, according to Swamy and Tanikawa, must be resistant to chemical attack, diffusion and weathering and have flexibility, crack bridging ability and durability [12]. While these performance requirements appear obvious, testing of barrier systems for compliance with these requirements is extremely difficult. Successful application of sealants and coatings is dependent on the surface preparation of the substrate but no satisfactory standard exists for concrete surface preparation often resulting in poor in situ performance [5]. The performance of concrete barrier systems is also affected by ambient conditions during application and post application effects such as dynamic loads and cracking.

The protection provided by concrete barrier systems may be estimated by either empirical approaches using actual or simulated exposure conditions or by measuring more intrinsic properties such as the increase in the material resistance to transport mechanisms. Several empirical investigations have been done into the protection provided by different sealants and coatings for marine concrete [13,14]. The advantage of empirical methods is that the complex interactions of concrete, environment and barrier system can be monitored under simulated exposure conditions. Intrinsic testing of the permeation characteristics of concrete protected with barrier systems is often done under ideal conditions with no allowance for external effects which may alter the performance of the barrier system. Tests need to be developed which characterize the potential performance of sealants and coating under typical exposure conditions.

b) Corrosion of reinforcement

The corrosion of reinforcement in concrete is generally too slow to monitor within the time scale of most laboratory investigations, so various methods have been devised to accelerate the corrosion process. Chloride-initiated corrosion may be accelerated by adding chlorides during mixing or by increasing the rate of ingress of chlorides using rapid wetting and drying cycles. The rate of corrosion may be increased by using an external impressed current to the reinforcement or changing anode/cathode area ratios. These external and internal agencies impact on the corrosion performance of reinforced concrete and may create artificial conditions and side effects which do not occur in reality [15]. Chloride admixtures in fresh concrete can not be equated to similar chloride levels resulting from external sources as the reinforcement has been exposed to differing chloride levels during the intervening period and admixed chlorides alter the binding characteristics [16]. Admixed chlorides may prevent the passive gamma ferric oxide layer forming on the steel so that local areas of depassivation are not produced as occurs with external chlorides. Impressed current accelerates the corrosion process but side effects such as hydrogen evolution, alkali build up and other effects may change the structural properties of the concrete and the corrosion products formed. Studying steel corrosion in artificial pore water solutions allows greater control and flexibility but has also been shown to produce results of a dubious quality [17].

Useful information about corrosion initiation and chloride corrosion threshold levels has been produced using half-cell potential measurements. Several investigations have been done exposing concrete specimens to chloride solutions either by partial immersion or using spray chambers and the corrosion monitored using half-cell potentials [18,19]. The amount of corrosion may be accurately measured by gravimetric means after a suitable period of exposure which is typically between two and five years. Typical results of corrosion monitoring using rebar potentials are shown in Figure 6.1. Other more sophisticated and reliable corrosion monitoring methods such as linear polarization and coulostatic techniques are available but may be prohibitively expensive for practical testing [20]. Laboratory investigations of steel corrosion must be related to results from site where conditions are more variable and are complicated by a variety of external influences [21,22].

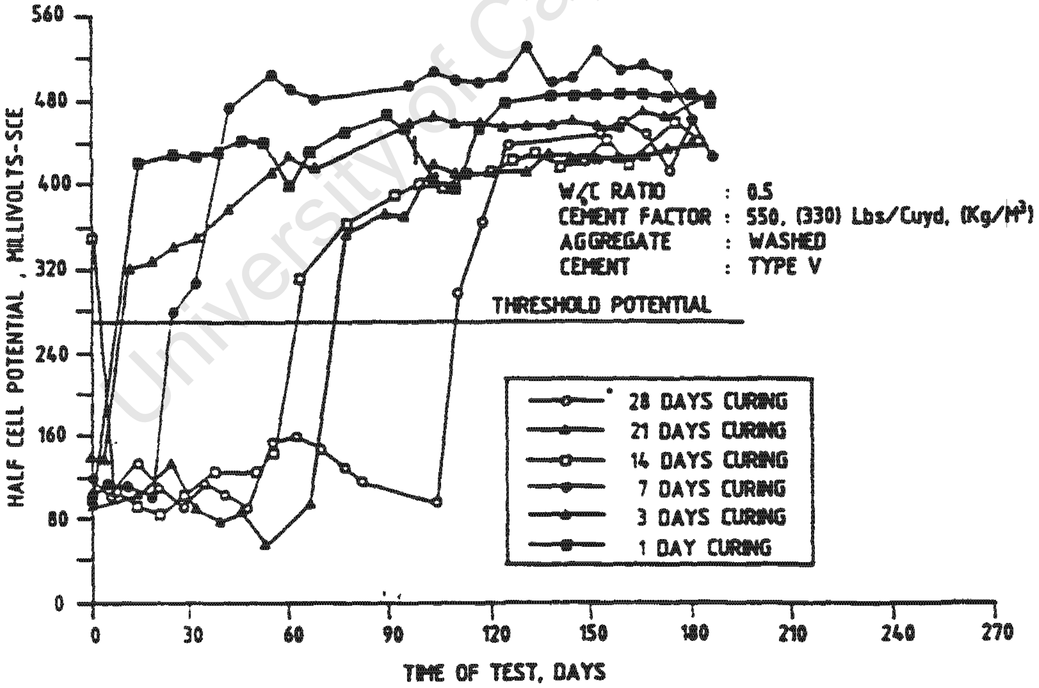

Figure 6.1: Rebar Potentials For Different Curing Methods [20]

Cement extenders such as fly ash, slag, and silica fume have been shown to increase the resistance of concrete to chloride penetration but concern has been expressed about the corrosion threshold levels and corrosion rates of reinforcement in these concretes [23].

Chemical changes occur in the concrete when cement extenders are used, affecting the ratio of bound to free chlorides and the hydroxyl ion concentration. Changes in the Cl^-/OH^- ratio in concrete could theoretically alter the corrosion threshold level of embedded reinforcement. The formation of a passive oxide layer on steel embedded in slag concrete is reportedly slower than for similar OPC concrete and the layer formed tends to be more porous [24]. This could have implications on the corrosion characteristics of slag concrete but no evidence of this possible shortcoming has been found on slag concrete structures in service [25].

Dhir et al showed that fly ash concrete had similar corrosion threshold levels to OPC concrete when exposed to a concentrated NaCl spray environment [26]. The rate of propagation of corrosion was found to be higher in OPC concrete than fly ash concrete and was found to increase with increasing chloride content. Bamforth states that the reduction of OH^- levels in fly ash and slag concrete is compensated by the increasing chloride binding capacity of these concretes [27]. Results from concrete exposure samples in Folkestone, England indicate that similar corrosion threshold levels exist for OPC, fly ash and slag concrete [28]. In a recent review on the subject Rasheeduzafar and Hussein concluded that "fly ash and slag do not change the Cl^-/OH^- ratio of the pore solution significantly' and should therefore have similar critical chloride levels to that of OPC concrete [29]. Silica fume concrete has been found to have low corrosion threshold levels due to the limited chloride binding capacity and the marked reduction of hydroxyl ions which effectively increases the Cl^-/OH^- ratio [27].

c) Electrochemical chloride extraction

The feasibility of using electrochemical techniques to remove chloride ions from concrete has been investigated in the past but the high current densities employed raised concerns about negative side-effects such as cracking from high temperatures, increasing permeability and debonding of reinforcement from hydrogen evolution [30]. Renewed interest in the process using lower current densities has become apparent in order to solve the growing number of case of reinforcement corrosion. An impressed direct current is applied using an external anode and electrolyte and the current is passed between the anode and the reinforcing steel which acts as the cathode. Temporary current densities of up to $1A/m^2$ of concrete surface area are used which are significantly higher than those used for cathodic protection ($10mA/m^2$) [31].

The impressed current creates an electric field in the concrete which induces negatively charged ions (Cl^- and OH^-) to migrate from the reinforcement (cathode) to the external electrode (anode). Hydroxyl ions are formed at the reinforcement while at the anode oxygen and chlorine gas are formed. Figure 6.2 shows the electrochemical process in schematic form. The electrochemical process is generally believed to decrease the potential of the reinforcement, increase the hydroxyl ion concentration and decrease the chloride ion concentration around the steel. Concern still remains that the technique might promote alkali aggregate reaction, hydrogen embrittlement of the steel and acid generation at the concrete surface. The method may not be practicable when repairing complex shaped structures, concrete with major defects such as cracks and voids or when electrical continuity can not be guaranteed [31].

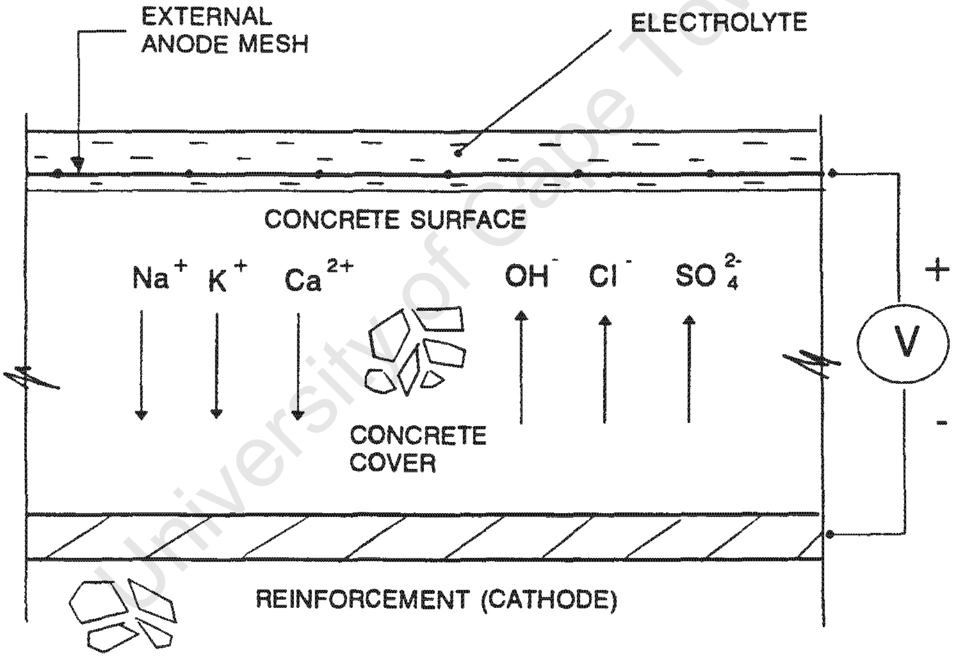

Figure 6.2: Electrochemical Extraction of Chlorides

The time required to extract chlorides varies from a few weeks to several months depending on chloride levels, concrete quality and reinforcement distribution. During the process, regular monitoring is essential to ensure suitable progress and safe rebar potentials. Monitoring may be done by testing the concrete for chloride content periodically or by testing the electrolyte. Assessing the risk of hydrogen embrittlement and debonding of reinforcement is difficult to achieve on site structures and it is therefore essential to ensure that driving voltages are sufficiently low to prevent hydrogen evolution at the steel.

Research has shown the electrochemical techniques are able to remove both free and bound chlorides from concrete within relatively short periods of time [32]. Bound chlorides are believed to dissolve into solution due to the effect of the electric field which allows "easy desorption of adsorbed chloride" [32]. The reduction of chloride ion levels has also been shown to reduce the corrosion risk as measured by rebar potentials [30]. Concern has been expressed about the effectiveness of the technique of removing chlorides which have diffused beyond the reinforcement particularly for structures containing multiple reinforcing mats [33]. Chloride which has not been removed from the concrete may diffuse back towards the reinforcement, after the impressed current has been stopped, and activate corrosion [34]. This back diffusion would require multiple applications of the desalination process.

d) Seawater effect on concrete surface

Research on the surface effects of seawater attack on concrete indicates that insoluble salts may be deposited on the concrete surface [35]. Buenfeld and Newman showed that deposition of brucite and aragonite was responsible for increasing the resistance of concrete since the deposition effectively blocks the pores [36]. Haynes claimed that the build-up of insoluble salts on the concrete surface made concrete spheres impervious to seawater when submerged at great depths below the sea [37]. Van der Wegen et al showed that the permeability of concrete decreased with time of exposure to seawater under pressure due to "precipitation reactions near the porewater/seawater interface" [38]. The protective layer is relatively thin and may be dislodged by abrasion making it only effective in mild environments or under submerged conditions. The formation of an insoluble layer on the surface of marine concrete has major implications especially with regard to laboratory studies where simulations of marine exposure often use NaCl solutions instead of seawater.

Little research has been documented comparing chloride diffusion through concrete using seawater and salt water. Extensive research has been done on the diffusion of chloride ions into concrete, producing useful fundamental data about the influence of chloride binding, cement types, concrete grade and initial curing. These influences are generally studied in isolation and external effects such as carbonation, environmental and dynamic loading were generally ignored. Durability investigations need to consider a multivariate approach where several influences may interact simultaneously as occurs with site concrete [39,40]. Research of this nature is more complex but models site conditions more accurately than laboratory simulations which separate the influences of environmental, material and service factors which affect concrete durability.

6.2 Experimental Investigations

Details of the experimental investigations done in the laboratory are given below.

6.2.1 Sealant Testing Programme

The effectiveness of sealants proposed for use in repairs to a coastal river bridge was assessed in an accelerated laboratory testing programme. The bridge is located on the Kwazulu-Natal coast of South Africa and after thirty five years service exhibited widespread reinforcement corrosion damage. Repairs consisted of breaking out areas of chloride contaminated concrete, patching with a repair concrete and coating the bridge with a sealant system. The sealant system was used to prevent further chloride ingress and to limit the availability of moisture and oxygen in the concrete. The laboratory investigation consisted of exposing concrete panels to fifty wetting and drying cycles with a salt spray or submersion in a salt solution and determination of the chloride content in the concrete after the six week exposure period.

A grade 30 OPC concrete with a slump of 25 mm was used to produce six 300x300x50 mm panels which were subjected to different curing and coating treatments. Details of the concrete mix design and curing and coatings options are summarized in Table 6.1 and 6.2. Ten days after casting, each panel was cut into quarters of size 150x150x50 mm and all sides except the 150x150 mm test face were coated with epoxy. Two of the panels were used for accelerated marine exposure (50 wet/dry cycles), one panel was submerged in a 5M NaCl solution and one panel was kept as a control at 23°C and 60% R.H. until 28 days. Two 68 mm diameter cores were extracted from each control panel at 28 days and the surface 25 mm section tested for oxygen permeability, water sorptivity and chloride conductivity in accordance with details given in Chapter Three.

Table 6.1: Concrete Mix Design (Grade 30 OPC Concrete)

MIX CONSTITUENT	TYPE	QUANTITY
Cement	Ordinary Portland Cement - PPC	303 kg/m^3
Sand	Cape Flats Dune Sand	801 kg/m^3
Stone	19 mm Greywacke	1050 kg/m^3
Water	Potable	202 kg/m^3
Slump	-	25 mm

Table 6.2 : Curing and Coating Systems Applied to Concrete

SAMPLE	TYPE OF CURING	SILANE SEALANT	CEMENTITIOUS RENDER
A	Moist cured at 90 % R.H. for 7 days	None	None
B	Curing Compound after 24 hours	None	None
C	Curing Compound after 24 hours	One Coat at 7 days	None
D	Curing Compound after 24 hours	One Coat at 7 days	Two coats at 8 days
E	Dry cured at 60 % R.H. for 7 days	None	None
F	Shutter cured for 7 days	None	None

Details of the accelerated marine exposure testing which was started after 14 days are given below :-

- Ten seconds exposure to a 5M NaCl spray
- Two hours at high humidity (20°C and 95% R.H.)
- Ten hours at high temperature (50°C and 15% R.H.)

The short period of exposure to salt spray allowed surface wetting but was not long enough to cause washing off of surface deposits. The concrete was then kept at high humidity in a closed container for two hours to allow time for absorption before drying commenced. The concrete panels were then transferred to an oven to ensure that the concrete surface dried out and allowed significant absorption during the following wetting cycle. After every ten wetting and drying cycles the panels were submerged in tap water for 48 hours to allow diffusion of chlorides into the interior of the concrete. This was done to ensure that a build-up of salts did not occur near the concrete surface and prevented further penetration into the concrete.

The concrete panels were tested at 70 days for chloride penetration after fifty wetting and drying cycles or after six weeks submersion in the 5M NaCl solution. The concrete was sampled at 5mm increments to a depth of 25 mm using a rotary hammer drill and powder from three 20 mm holes was combined to ensure a representative sample. Chlorides were analysed in accordance with BS 1881 Part 124 using a potentiometric titration [41].

6.2.2 Corrosion of Mild and High Tensile Steel

The corrosion behaviour of reinforced concrete samples partially immersed in different test media was investigated using half-cell potential monitoring. The partial immersion allowed drying to occur at the top surface and promoted capillary suction forces. The capillary action was responsible for drawing up the test solution towards the steel. Mild steel round bars and high tensile ribbed bars of 16 mm diameter were embedded in grade 20 and 40 OPC concrete prisms and tested in solutions of 5% NaCl, sea water and tap water for periods up to 200 days using half-cell potential mapping. The concrete mix designs used to cast the concrete prisms are given in section 3.4.1 while the test arrangement is shown in Figure 6.3.

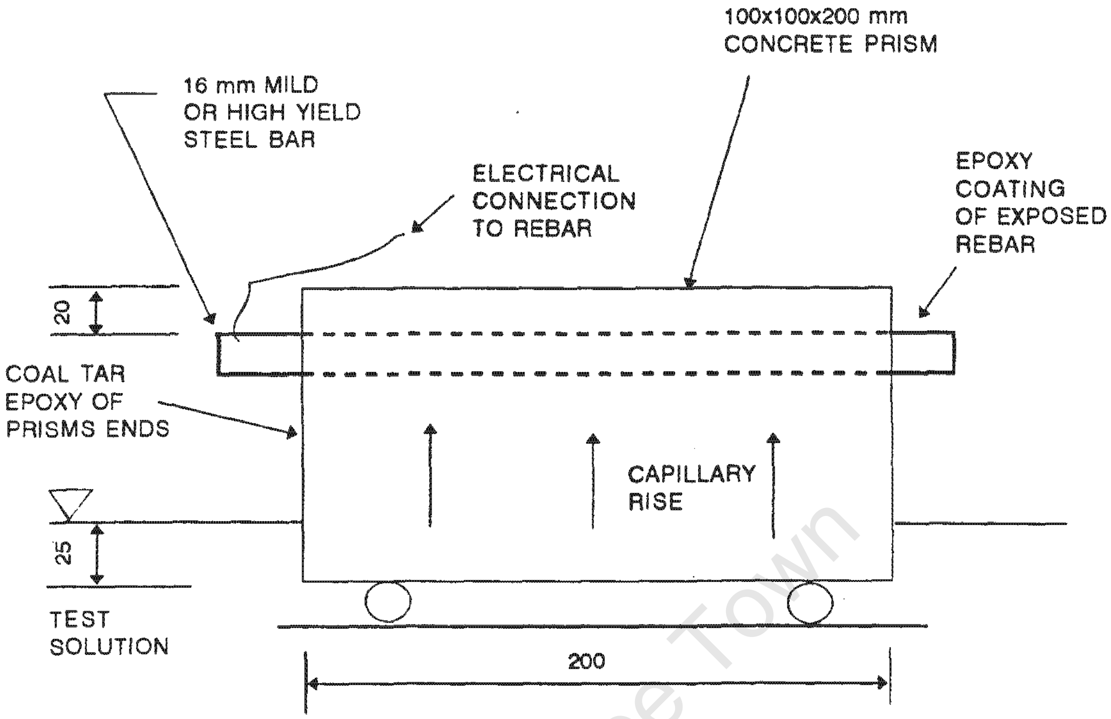

Figure 6.3: Arrangement of Concrete Prisms For Corrosion Monitoring

The reinforcing steel cast into the prisms was initially cleaned with an abrasive grinder to remove any mill scale and rust and each bar was weighed before being cast into the concrete prisms. The 200x100x100 mm concrete prisms were cast vertically with the steel bars being held in position with PVC plates at either end of the prism. After demoulding the prisms at 24 hours, the exposed ends of the reinforcement were coated with epoxy except for a 3 mm hole which was used to provide an external connection to the reinforcement. The 100x100 mm ends of the concrete prism were coated with a coal tar epoxy paint to prevent penetration of chlorides from the end faces. The concrete prisms were initially stored at 23°C and 60% R.H. for 28 days before being placed into each test solution.

Four specimens were used for each type of steel, concrete and test medium with a total of 48 concrete prisms being cast. Half-cell potential measurements were taken with a copper / copper sulphate electrode connected to a digital voltmeter in accordance with ASTM C876-91 [42]. Measurements were taken in the centre of the prism at regular intervals for 200 days and at the end of the test period prisms were split open and the amount of corrosion assessed visually.

6.2.3 Corrosion of Steel in Different Types of Concrete

The effect of OPC, fly ash and slag concrete on the chloride corrosion thresholds of high yield reinforcing steel was investigated using half-cell potentials and chloride contents. Concrete blocks of size 300x450x150 mm were used with a grid of 16 mm high yield ribbed bars placed at a minimum cover of 30 mm. Nine concrete blocks were used being either grade 20, 40 or 60 OPC, fly ash and slag concrete. Details of the concrete mixes are given in Chapter Three while the configuration of the concrete blocks is shown in Figure 6.4.

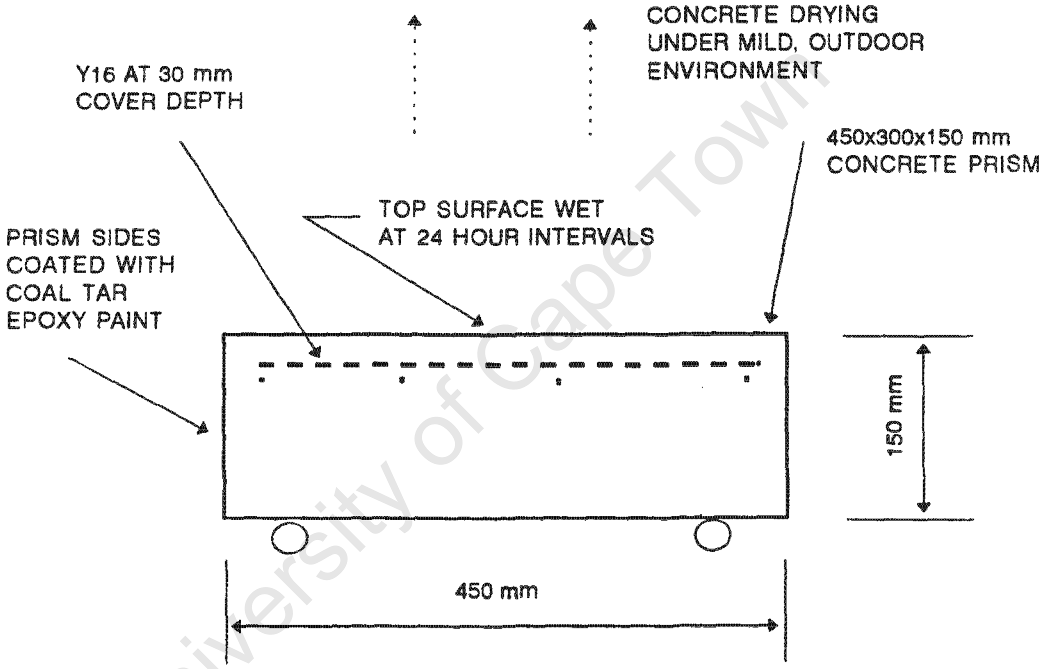

Figure 6.4: Arrangement of Concrete Blocks Used for Corrosion Investigation

The concrete blocks were cast in wooden moulds and stripped after 24 hours and thereafter stored at 23°C and 60 % R.H. in an environmentally controlled room until 28 days. The sides of the blocks were coated with two layers of coal tar epoxy paint to ensure uniaxial water penetration from the test surface. After 28 days the blocks were placed outside the laboratory exposed initially to summer conditions and were wet daily with a 5% NaCl solution. Wetting of the concrete surface was done by pouring 200 ml of salt solution over each sample which resulted in minor ponding on the horizontal surface. The wetting and drying cycles were expected to cause rapid penetration of chlorides into the concrete by a combination of absorption and diffusion so that corrosion would be activated within the six month test period.

The concrete blocks were tested at regular intervals using half-cell potential mapping and chloride content. Half-cell potentials were measured using a copper/copper sulphate electrode in accordance with ASTM C 876-91. Potential measurements were taken over the whole top surface of each block and the highest rebar potentials recorded. Chloride contents were measured from drilled samples extracted from two 20 mm holes at depths between 25 to 35 mm below the surface.

6.2.4 Electrochemical Chloride Extraction

OPC and fly ash concrete blocks used for corrosion investigations (6.2.3) were left out doors for three months exposed to winter weather before being subjected to elctrochemical chloride extraction. The extraction process was done using a stainless steel mesh as an external anode submerged in a 0.5M NaOH solution. An impressed direct current was applied between the external anode and the imbedded reinforcement (cathode) with a current density of 1A/m² of concrete surface area being used. Figure 6.5 shows the test setup with three concrete blocks connected in series. The current was maintained for 56 days and regular monitoring was done by analysing chlorides in concrete and rebar potentials every fourteen days.

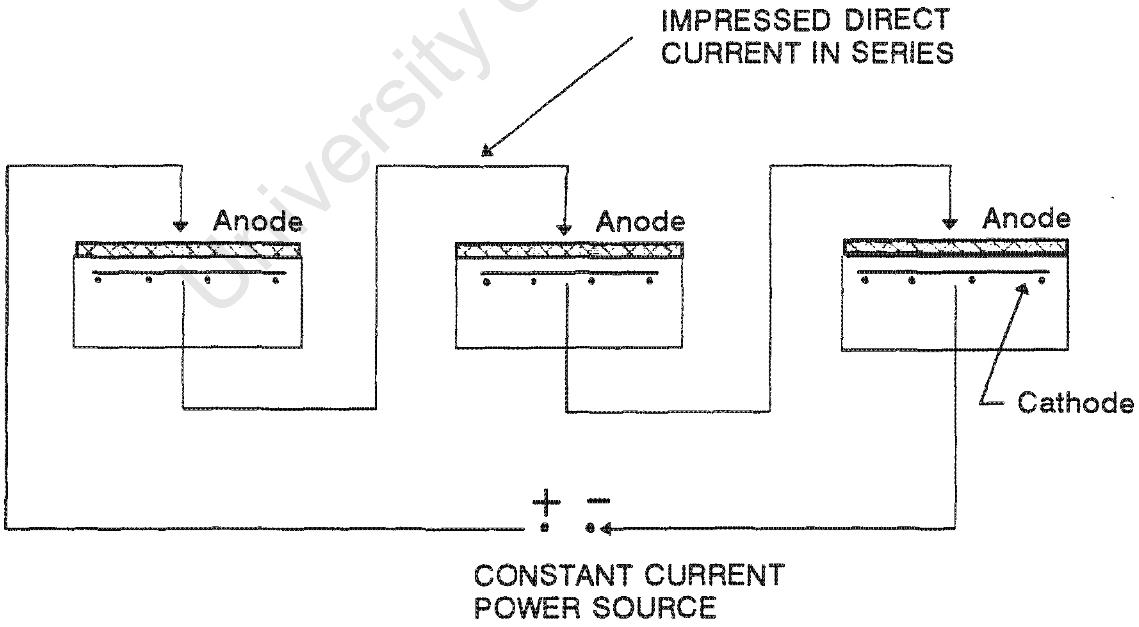

Figure 6.5: Elecrochemical Chloride Extraction Apparatus

After 56 days, the concrete blocks were placed outdoors to allow the reinforcement to depolarize and return to resting potentials. No assessment was made of the increased risk of alkali aggregate reaction from the electrochemical chloride extraction due to the increasing alkalinity around the reinforcement but the risk of hydrogen evolution was assessed using rebar potentials.

6.2.5 Effect of Sea Water on the Surface of Concrete

The chloride penetration into concrete exposed to seawater and salt water solutions was investigated by immersing OPC, fly ash and slag concrete in three different test media in the laboratory. Grade 40 concrete was cast into 100 mm concrete cubes moulds using concrete mixes given in section 3.4.1. The concrete cubes were demoulded after 24 hours and immediately exposed to either a 3% NaCl solution, sea water or tap water for a period of 180 days. Eighteen concrete cubes were placed in each 50 litre container with all vertical faces exposed directly to the test media. The solutions were replaced twice monthly to maintain reasonably stable conditions.

The concrete specimens were continuously submerged during the six months of exposure so that chloride ingress was by diffusion alone. Care was taken to minimise handling of the specimens during immersion to prevent damage of the concrete surface. Chloride profiles were determined after six months exposure by analysing drilled powder taken from two 20 mm holes at 10 mm increments from 0 - 40 mm. Compressive strength testing was also done on concrete cubes exposed to sea water, 3% NaCl solution and tap water after six months.

6.3 Experimental Results

Results from the investigations of concrete sealants, steel corrosion, electrochemical chloride extraction and seawater/concrete interaction are given below.

6.3.1 Sealant Testing Programme

Concrete characterization testing done at 28 days showed distinct differences, with dry and shutter cured concrete having the worst potential durability as measured by the durability index tests. Results from the concrete characterization testing are given in Table 6.3. Concrete treated with silane or silane and render had lower chloride conductivity than other concrete but similar oxygen permeability and sorptivity values. The curing compound appeared to reduce the sorptivity and chloride conductivity of concrete compared to dry and shutter cured concrete but the compressive strength was similar after 28 days. Remnants of the wax emulsion used in the curing compound were still visible on the concrete surface during testing and this may have interfered with the test processes (particularly sorptivity).

Table 6.3: Concrete Characterization Results

SAMPLE No.	CURING / COATING	COMPRESSIVE STRENGTH (MPa)	OXYGEN PERMEABILITY INDEX	WATER SORPTIVITY (mm/ hr)	CHLORIDE CONDUCTIVITY (mS/cm)
A	Moist curing for 7 days	29.5	9.02	8.7	2.23
B	Curing comp.	24.3	9.42	9.8	2.06
C	Curing comp. and silane	(24.3)	8.97	10.2	1.74
D	Curing comp. silane, render	(24.3)	9.56	8.8	1.24
E	Dry curing for 7 days	22.0	9.13	13.1	2.66
F	Shutter cured for 7 days	27.0	9.35	10.5	2.14

After fifty cycles of wetting and drying with a 5M NaCl solution several panels showed evidence of salt crystallization damage, particularly the dry and shutter cured concrete. Concrete submerged continuously had lower chloride levels than concrete subjected to wet-dry cycles which indicates the severity of the wet-dry cycles. Concrete treated with silane or silane and render had significantly lower chloride levels in both environments than uncoated concrete while dry and shutter cured concrete had higher chloride levels. Chloride profiles for concrete exposed to the spray and submerged environments are shown in Figure 6.6a and 6.6b while chloride contents are given in Table 6A1.

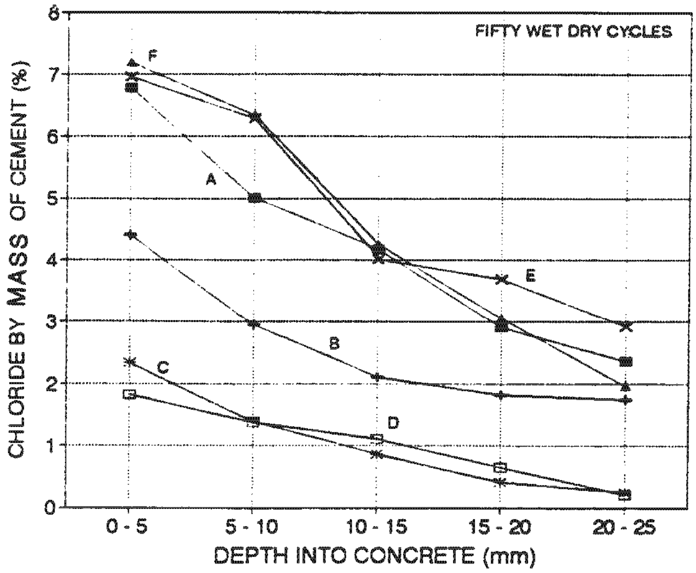

Figure 6.6a: Chloride Profiles After Fifty Wet Dry Cycles

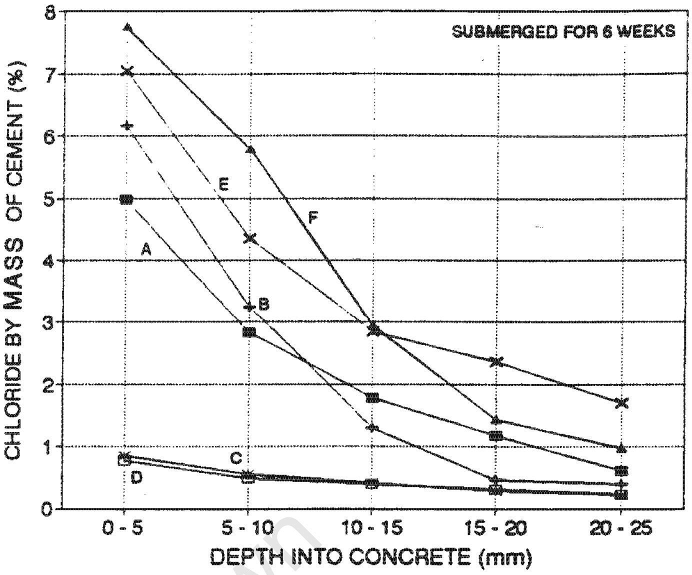

Figure 6.6b: Chloride Profiles After Six Weeks Submersion

A reasonable correlation was found between chloride conductivity results and chloride contents measured at a depth of 20-25 mm after six weeks of either wet-dry cycles or complete submersion in the 5M NaCl solution. Figure 6.7 shows the relationship between chloride levels and 28 day chloride conductivity values with sprayed samples having higher chloride levels at 25 mm than submerged concrete. The more rapid chloride ingress of the spray samples may have been partly due to capillary forces causing absorption at the surface but the harsh drying (50 °C and 15 % R.H.) may have caused microstructural damage to the concrete allowing easier chloride ingress.

Durability index and accelerated test results indicate that the coating system proposed for the bridge should improve the durability of the structure. The benefits might not be as significant as the laboratory testing suggests however as only the short-term perfomance was assessed. The effect of influences such as UV exposure, shrinkage and thermal cracking, unstable formulation and biological attack on the long-term stability of the coatings needs to be determined before reliable durability predictions can be made. The coating system should however improve the resistance of the concrete to chloride penetration in the short and medium-term and increase the potential durability of the structure.

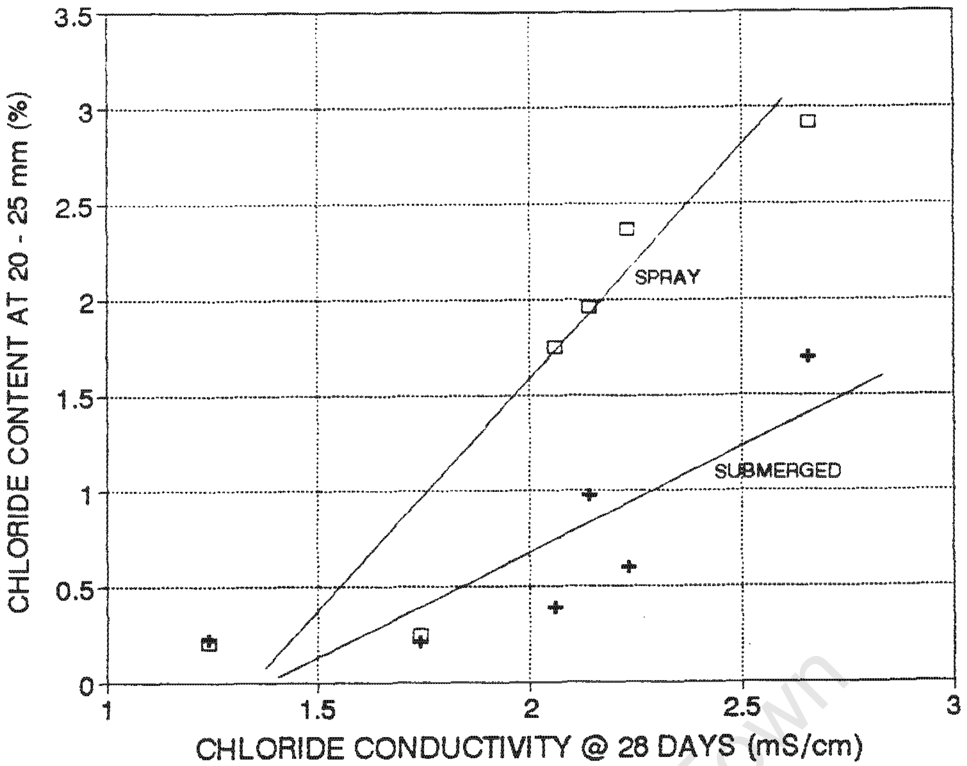

Figure 6.7: Chloride Content versus Chloride Conductivity

6.3.2 Corrosion of Mild and High Tensile Steel

Rebar potentials of samples in salt and sea water were generally more negative than -350 mV after 200 days indicating a high risk of corrosion. Grade 20 concrete had rebar potentials less than -600 mV within ten days of exposure in sea and salt water and there was evidence of salt crystallization on the top surface of the concrete, particularly samples partially immersed in salt water. All samples exposed to tap water had moderate rebar potentials initially which increased consistently with time to around -100 mV after 200 days. Rebar potentials of mild and high yield steel in grade 20 and 40 MPa concrete in the three solutions are shown in Figure 6.8a - 6.8d and Table 6A2.

Concrete samples exposed to salt water had more negative rebar potentials than specimens in sea water which was probably due to the higher concentration of chlorides in the 5 % NaCl solution. Concrete samples in sea water initially showed evidence of salt crystallization on the top surface but after approximately sixty days this phenomenon ceased and the top surface was relatively dry which might have been due to pore blocking of the concrete surface due to the seawater. Concrete partially immersed in salt water, by contrast showed evidence of salt crystallization throughout the test period.

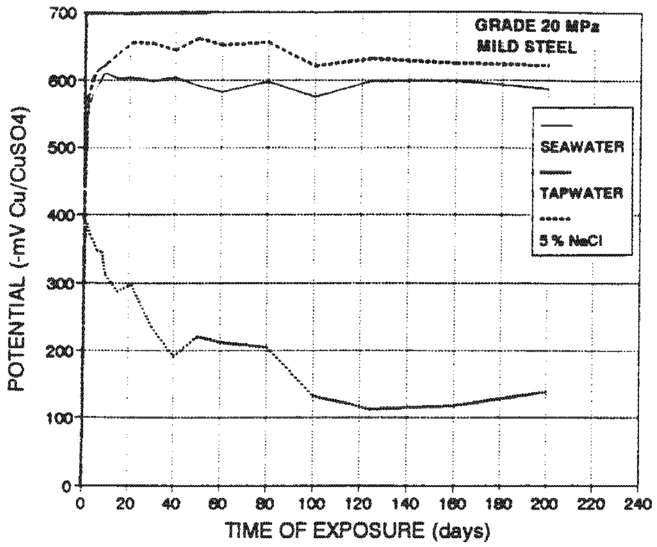

Figure 6.8a: Rebar Potentials for Mild Steel in Grade 20 Concrete

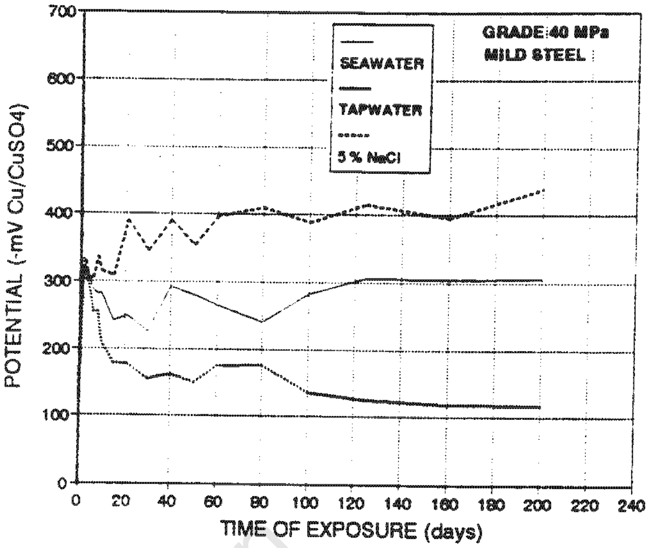

Figure 6.8b: Rebar Potentials for Mild Steel in Grade 40 Concrete

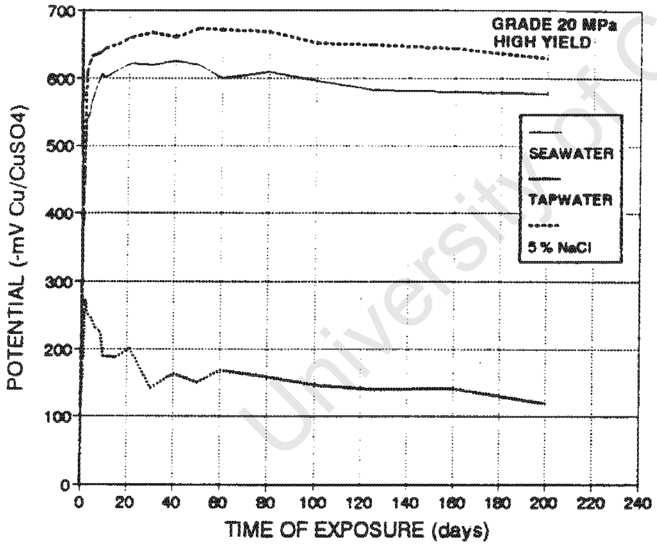

Figure 6.8c: Rebar Potentials for High Yield Steel in Grade 20 Concrete

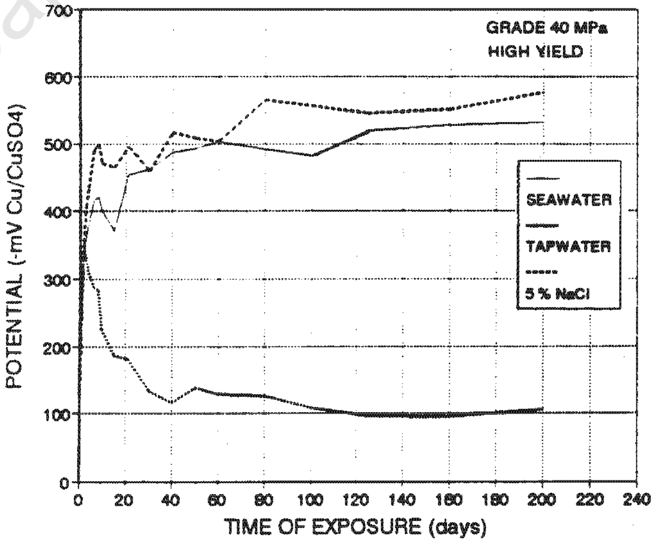

Figure 6.8d: Rebar Potentials for High Yield Steel in Grade 40 Concrete

High yield steel generally had more negative rebar potentials than mild steel for each environment and grade of concrete. The difference in rebar potentials between the two types of steel was particularly evident for grade 40 concrete where after 200 days immmersion rebar potentials of mild steel in sea water were around -300 mV while high yield steel was -530 mV. These results would indicate that high yield steel is less corrosion resistant than mild steel in terms of time to corrosion activation but the difference in corrosion rates was not assessed.

Gravimetric measurement of corrosion of the reinforcement was only done on a limited number of samples after 200 days exposure as negligible levels of corrosion were observed. A full analysis was therefore not done since no conclusive results could be drawn at such an early age. Corrosion pits were located immediately adjacent to the concrete surface even though a coal tar epoxy paint was used to try and prevent corrosion at the concrete surface. The steel elsewhere in the concrete was in good condition with no evidence of corrosion.

6.3.3 Corrosion of Steel in Different Concrete Types

Rebar potentials of high yield steel in different concrete types and exposed to wetting and drying with a 5 % NaCl solution exhibited a number of consistent trends. Grade 20 OPC, fly ash and slag concrete had highly negative rebar potentials after as little as 20 days, and remained constant for the remaining test period. Grade 40 OPC concrete exhibited a similar trend to the grade 20 MPa concrete but fly ash and slag concrete had high negative potentials at early ages, reducing to moderate levels at later ages. All reinforcement in grade 60 MPa concrete had moderate potentials in the range of -300 to -200 mV indicative of a low risk of corrosion. Figures 6.9a - 6.9c shows the rebar potentials of the steel in OPC, fly ash and slag concrete while rebar potentials and chloride contents are given in Table 6A3.

Reinforcement in OPC concrete had decreasing rebar potentials with time indicating an increasing risk of corrosion. Fly ash and slag concrete on the other hand had a rapid decrease of rebar potentials within the first 20 days of exposure. At later ages, the rebar potentials of fly ash and slag concrete samples became less negative and only the grade 20 concrete had a high risk of corrosion after 200 days. The increase in rebar potentials could have been due to chloride binding by fly ash or slag concrete. These longer-term effects reduced the free chloride content at the steel resulting from rapid absorption at early ages. The reduction of free chloride levels in the concrete would have reduced the depassivating effect at the steel surface and resulted in less negative rebar potentials. The change of weather conditions from summer to winter during the test period was not thought to have been responsible for the observed trend since OPC concrete did not exhibit increasing rebar potentials with time.

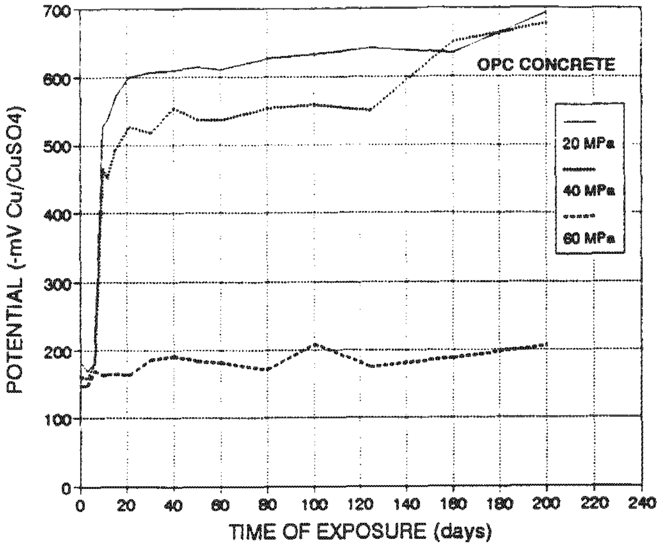

Figure 6.9a: Rebar Potentials in OPC Concrete

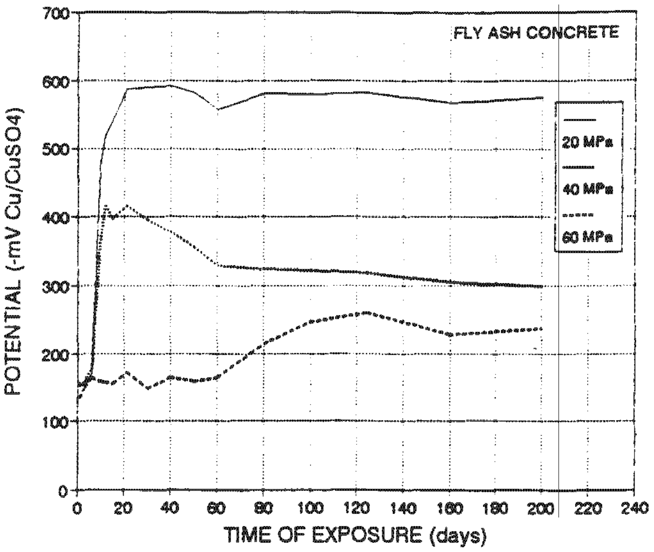

Figure 6.9b: Rebar Potentials in Fly Ash Concrete

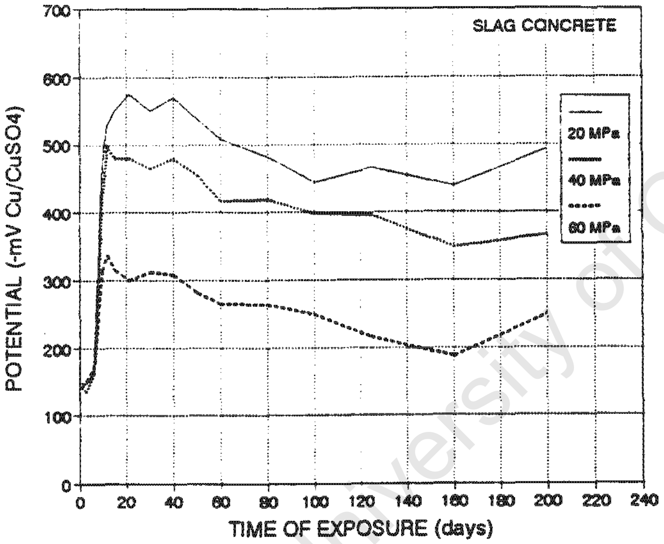

Figure 6.9c: Rebar Potentials in Slag Concrete

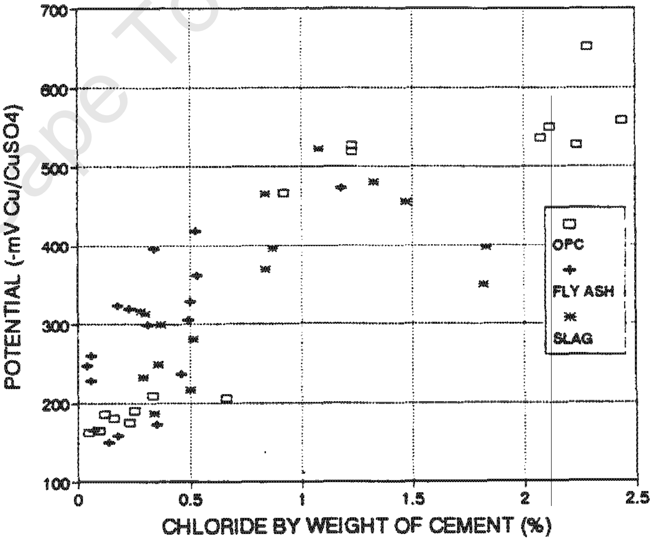

Figure 6.9d: Rebar Potentials versus Chloride Content at Rebar

A comparison of chloride levels at the reinforcement with rebar potentials for OPC, fly ash and slag concrete are shown in Figure 6.9d. Generally rebar potentials were between -150 and -400 mV for chloride levels below 0.5 % at the steel while potential ranged between -350 and -650 mV for chloride contents above 0.5 % at the reinforcement. OPC concrete had consistently less negative rebar potentials (>-250 mV) for chloride levels up to 0.7% compared with fly ash and slag concrete. This would indicate a lower potential for corrosion of rebar in OPC concrete at low chloride levels but the information provides no indication of the risk of depassivation at the steel surface. At chloride levels greater than 0.7%, OPC negative rebar potentials (<-450 mV) than those of fly ash and slag concrete. The results may be interpreted to indicate that corrosion thresholds for OPC are between 0.7% and 0.9% while thresholds for fly ash and slag concrete are in the region of 0.5% chloride by mass of cement. Insufficient data (particularly in the transition between passive and active corrosion conditions) was available however to make reliable estimates of corrosion thresholds from this information. The general trend when assessing all concrete types was in reasonable agreement with currently quoted threshold levels. Replicate samples could have improved the correlation between chloride content and rebar potentials by providing more data.

6.3.4 Electrochemical Chloride Extraction

After 56 days of electrochemical extraction, chloride levels in all concrete blocks had reduced significantly but chloride beyond the reinforcement was not always extracted. The process was most successful in high grade concrete that had low initial chloride levels. Grade 20 concrete still had high chloride levels after 56 days of electrochemical chloride extraction. Both bound and free chlorides appeared to be removed from the concrete during the extraction process and there was no significant difference between OPC and fly ash concrete. Chloride contents measured at 14 day intervals are shown in Appendix Table 6A4. Chloride profiles before and after electrochemical extraction are shown in Figures 6.10a-f. Initial chloride profiles had low surface concentrations caused by washing out during winter rainfall.

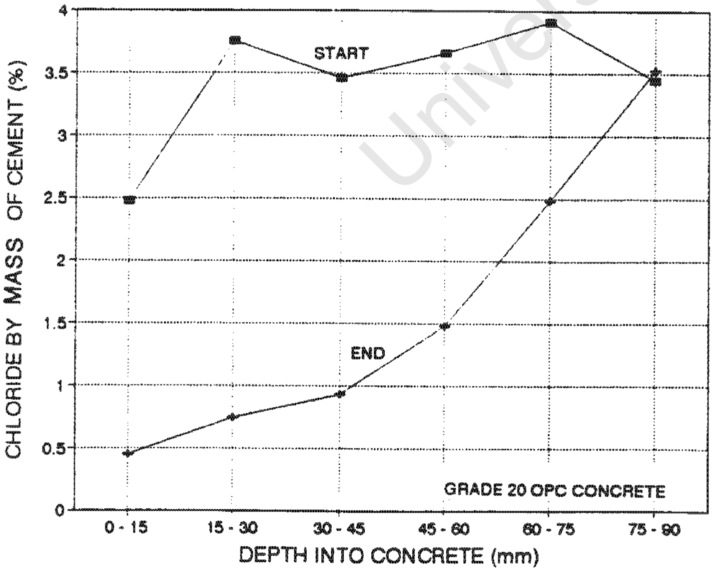

Figure 6.10a: Chloride Profiles for Grade 20 OPC Concrete

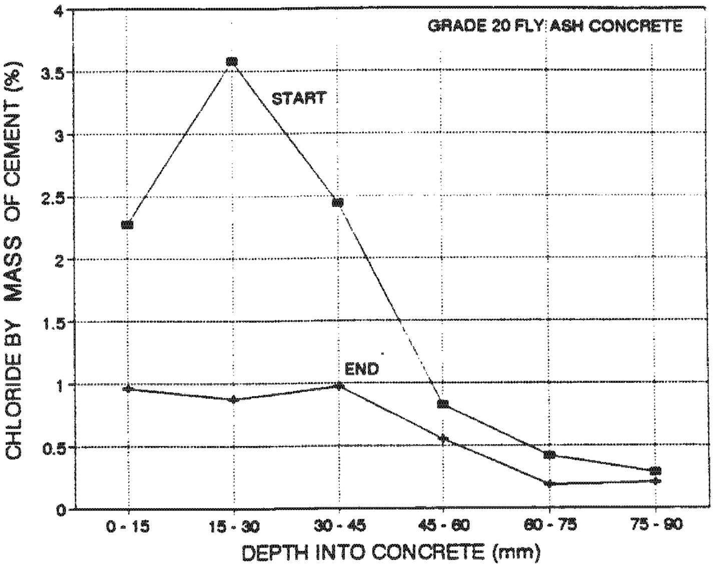

Figure 6.10b: Chloride Profiles for Grade 20 Fly Ash Concrete

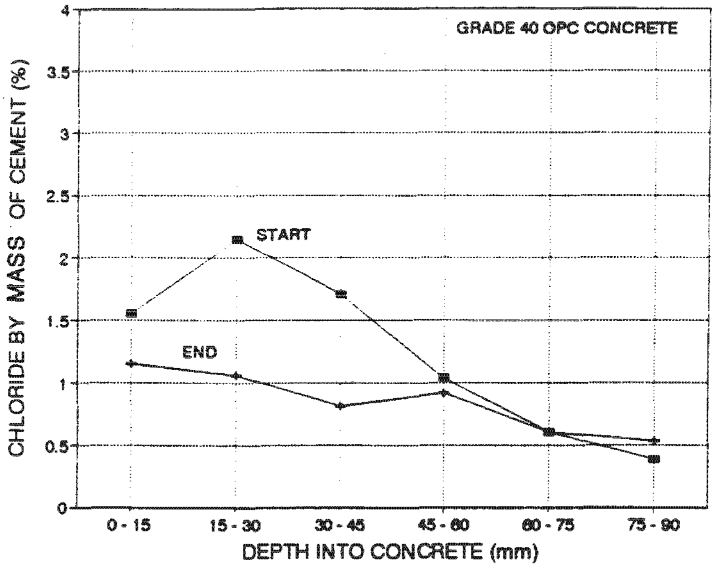

Figure 6.10c: Chloride Profiles for Grade 40 OPC Concrete

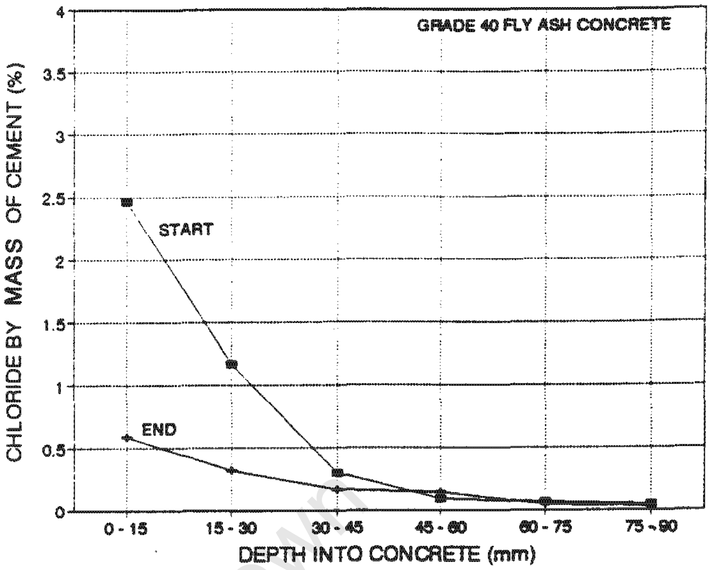

Figure 6.10d: Chloride Profiles for Grade 40 Fly Ash Concrete

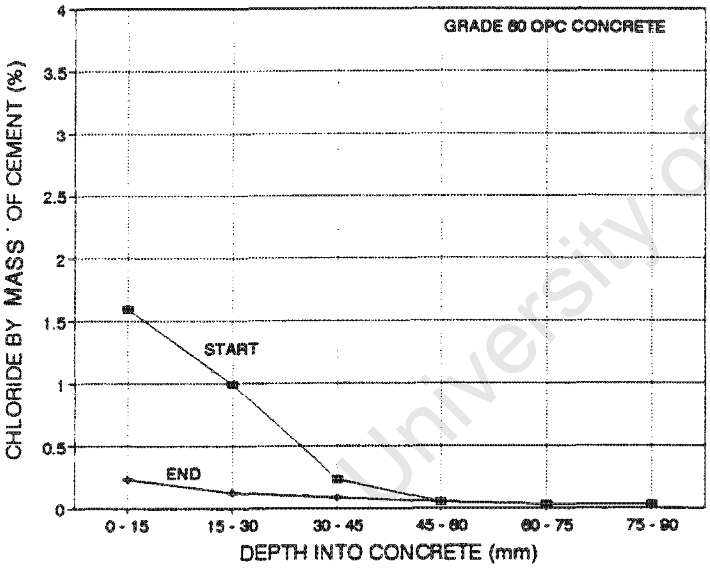

Figure 6.10e: Chloride Profiles for Grade 60 OPC Concrete

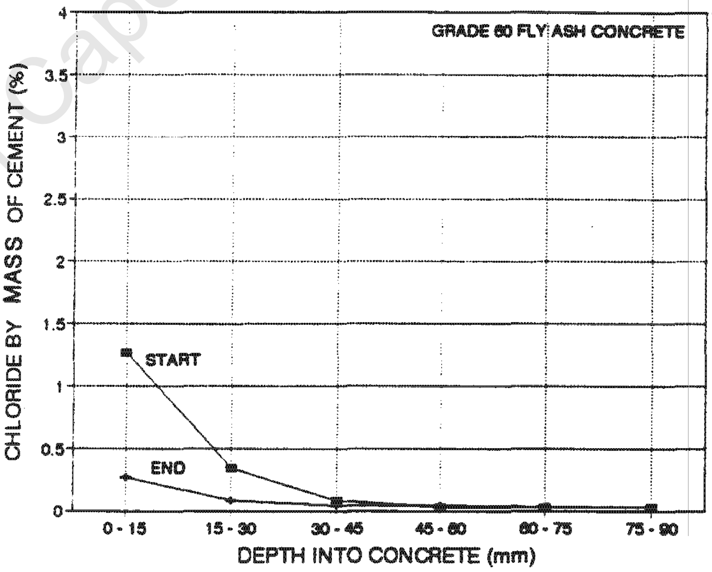

Figure 6.10f: Chloride Profiles for Grade 60 Fly Ash Concrete

Several blocks showed evidence of hydrogen evolution (in the form of bubbles on the concrete surface) and rebar potentials (< - 1150 mV) indicated that the current density was too high for the reinforcement configuration. A more rational approach toward selecting the applied current density would be to specify the current density in terms of steel surface area rather than concrete surface area. Elecrochemical extraction may need to be applied for periods longer than two months when the concrete is heavily contaminated with chlorides.

6.3.5 Effect of Sea Water on the Surface of Concrete

Chloride contents in the concrete were similar for samples exposed to seawater and NaCl solutions indicating that little pore blocking from brucite or aragonite deposition had occurred in concrete exposed to seawater. The solutions were replaced every two weeks which resulted in the pH of the solutions being high (pH 11 - 12) before replacement. The high pH of the seawater in the laboratory may have been responsible for the lack of pore blocking at the concrete surface. Buenfeld and Newman found that aragonite precipitation did not occur in seawater with a pH above 8.1 and the magnesium ions needed to form the brucite deposits became depleted unless the seawater was replaced regularly [43]. Conditions in the laboratory were clearly different from natural seawater exposure and did not promote deposition of insoluble salts. Chloride profiles recorded after six months exposure of OPC, fly ash and slag concrete in seawater and NaCl solution are shown in Figure 6.11a and 6.11b and in Table 6A5.

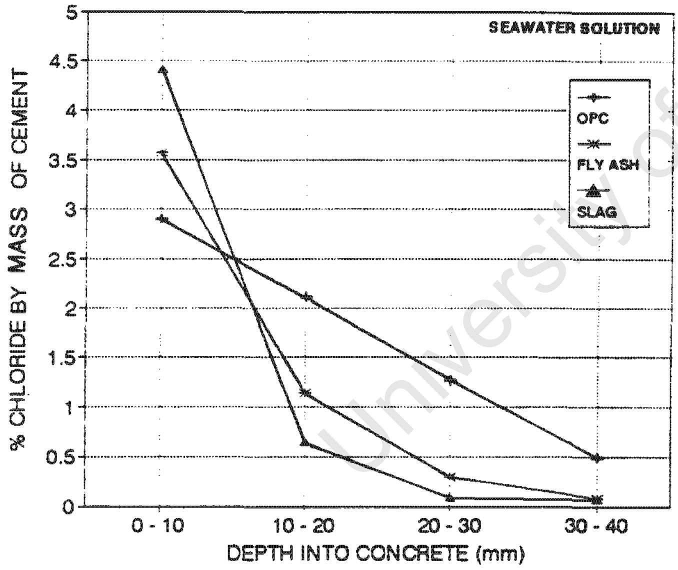

Figure 6.11a: Chloride Profiles in Sea Water

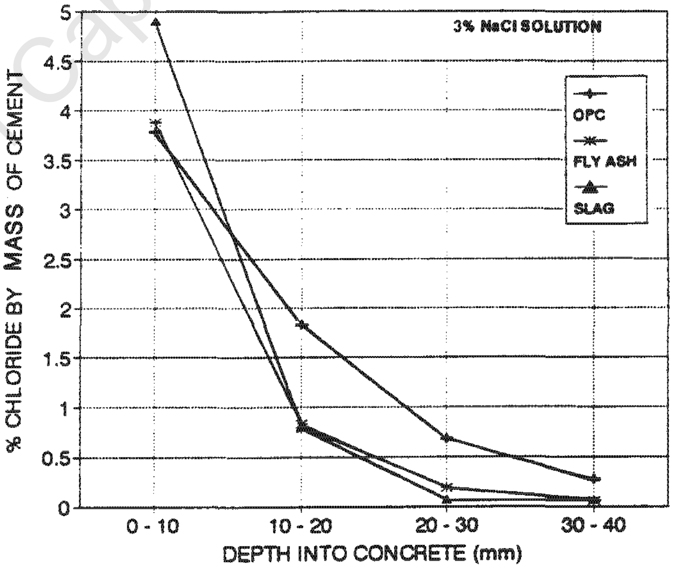

Figure 6.11b: Chloride Profiles in Salt Water

OPC concrete had lower chloride resistance than either fly ash or slag concrete as was expected from the results of marine exposure testing reported in Chapter Four. Chloride concentrations at the concrete surface were higher for fly ash and slag concretes than OPC concrete but at depth chloride levels were significantly lower. The higher surface concentration of cement extender concretes may be ascribed to the higher chloride binding capacity of these materials. The increased chloride binding capacity of blended cement-based concretes allows higher concentrations to be held in the pore structure. Surface concentrations were also higher in the NaCl solution compared to seawater even though both solutions had similar chloride levels. The surface concentrations of concrete exposed to seawater were similar to those measured during marine exposure testing of concrete (Chapter Four). Comparisons of diffusion coefficients from this laboratory study with values achieved for field exposure concrete was not done due to the different curing conditions and age of concretes when initially exposed.

Compressive strengths recorded after six months showed that fly ash and slag concrete benefit more from extended curing than OPC concrete. Compressive strengths of concrete cured in either seawater or salt water were lower than those cured in tap water for all three types of concrete. Admixed chlorides are known to accelerate cement hydation but external chloride appear to have a slightly detrimental effect on concrete strength development although the mechanism is not clear. Compressive strengths after six months exposure are shown in Figure 6.12. The 28 day compressive strengths for these concretes are given in section 3.4.2a.

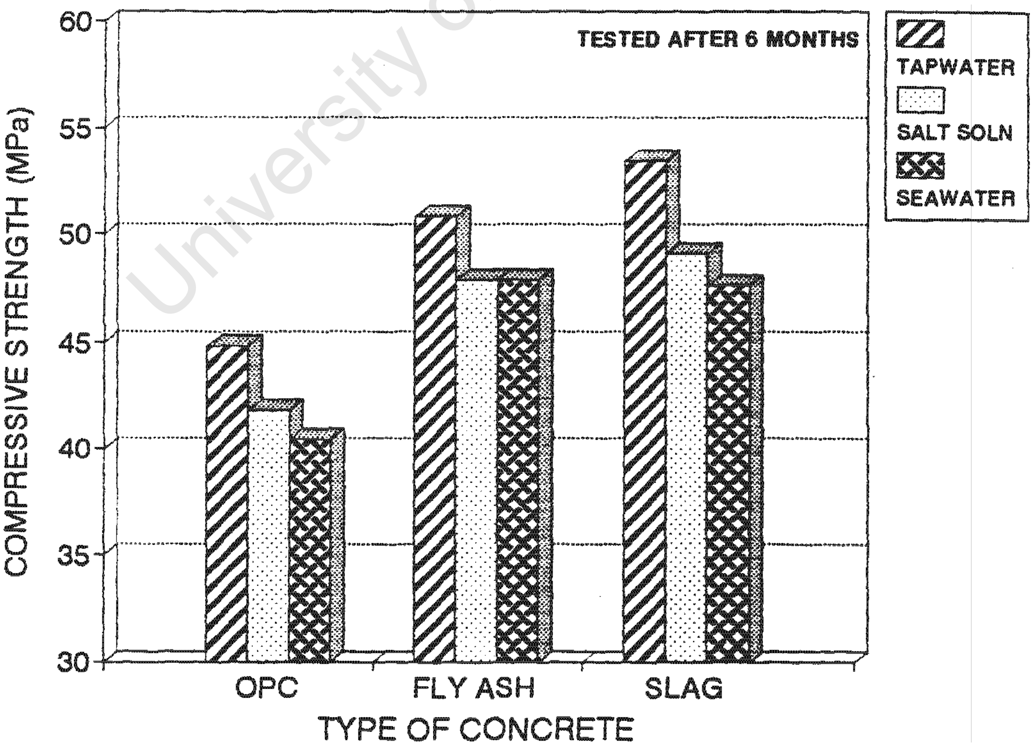

Figure 6.12: Compressive Strength Results after Six Months

6.4 Conclusions

Sealants such as silane coatings may provide protection to concrete from chloride penetration and improve the potential durability of reinforced concrete structures in the marine environment. The performance of sealants under service conditions was not assessed however and no conclusions about long-term characteristics could be drawn. The sealant system did improve the chloride resistance of concrete under extremely harsh laboratory conditions. Results from the sealant testing programme would seem to indicate that the use of the sealants may promote durable properties in concrete.

The corrosion investigations would indicate that the time to corrosion activation of reinforcement in concrete may be increased with high grade concrete and when using cement extenders such as fly ash and slag. Suitable test techniques must be developed for corrosion studies since the severity of exposure, with respect to steel corrosion, may be higher for NaCl solutions of the same chloride concentration as seawater. This has implications for laboratory simulations of marine conditions. OPC concrete samples exposed to wetting with a salt solution had corrosion thresholds 0.7 to 0.9 % chloride by mass of cement while fly ash and slag concrete had corrosion threshold of approximately 0.5% chloride by mass of cement. While these differences in corrosion thresholds for OPC compared with fly ash and slag concrete were not in agreement with corrosion thresholds reported elsewhere, insufficient data was available to make reliable estimates of corrosion threshold levels. The general trend would indicate however that currently acceptable corrosion threshold levels are reasonable. The complexity of steel corrosion in concrete makes accelerated test techniques extremely vulnerable to external interference which may cause erroneous results.

Electrochemical extraction of chlorides from concrete was found to be a feasible repair technique which should reduce the risk of corrosion of reinforced concrete structures. The process should be applied before chloride ions have diffused past the reinforcement in significant quantities and major corrosion damage has occurred. Careful monitoring of the driving voltage and current density is essential to prevent potentially dangerous side-effects such as alkali aggregate reaction and hydrogen embrittlement of the steel. The laboratory study successfully removed both bound and free chlorides from OPC and fly ash concrete but extraction periods of greater than two months are required for some concretes. The current density commonly used in this technique ($1A/m^2$ of concrete surface area) may be excessive in some circumstances and a more rational method needs be to used for specifying current densities (ie. in terms of steel surface area).

The investigation of the interaction of sea water with concrete was inconclusive and served only to show that laboratory simulations of marine conditions require a fundamental understanding of the mechanisms involved in order to accurately model service conditions. There appears to be sufficient justification for using field exposure testing rather than laboratory simulations given the complexity of chemical and physical interactions between concrete and seawater. Laboratory exposure tests need to be assessed critically and limitations clearly defined before the implication of these tests are considered.

The results of these accelerated tests provided useful data for confirming observations from field exposure and case study investigations but when reviewed in isolation can not be used to make confident predictions of concrete durability. Further refinement of these techniques would improve the accuracy of the results but this approach has only limited applicability to durability studies and should only be used to complement other durability studies.

6.5 References

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6.6 Appendices

SAMPLE No.	CURING / COATING	TYPE OF EXPOSURE	DEPTH INTO CONCRETE (mm)				
			0 - 5	5 - 10	10 - 15	15 - 20	20 - 25
A	Moist curing for 7 days	Spray cycles	6.78	5.01	4.18	2.91	2.36
		Submerged	4.98	2.83	1.79	1.17	0.60
B	Curing comp.	Spray cycles	4.41	2.95	2.10	1.82	1.75
		Submerged	6.16	3.24	1.29	0.46	0.39
C	Curing comp. and silane	Spray cycles	2.33	1.39	0.87	0.41	0.25
		Submerged	0.85	0.55	0.41	0.28	0.22
D	Curing comp. silane, render	Spray cycles	1.83	1.37	1.11	0.65	0.21
		Submerged	0.77	0.49	0.39	0.31	0.23
E	Dry curing for 7 days	Spray cycles	6.96	6.28	4.01	3.70	2.92
		Submerged	7.05	4.36	2.84	2.35	1.70
F	Shutter cured for 7 days	Spray cycles	7.18	6.33	4.26	3.06	1.96
		Submerged	7.76	5.79	2.95	1.42	0.98

Table 6A1: Chloride Contents For Spray and Submerged Exposure (%)

SOLUTION	REBAR POTENTIALS ($Cu/CuSO_4$) IN mV SEA WATER				TAP WATER				SALT WATER			
STEEL	MILD STEEL		HIGH YIELD		MILD STEEL		HIGH YIELD		MILD STEEL		HIGH YIELD	
GRADE	20	40	20	40	20	40	20	40	20	40	20	40
TIME (days)												
0	127	108	133	120	115	126	122	138	129	138	115	130
1	406	226	350	226	362	187	168	292	431	279	465	210
2	568	324	536	320	387	332	273	352	575	328	609	356
3	568	319	545	363	371	301	251	328	587	329	623	408
4	582	310	565	381	365	299	246	308	602	304	632	436
6	592	287	588	415	348	254	231	289	614	305	635	490
8	607	281	607	422	346	254	225	282	619	336	639	499
10	609	282	601	400	312	211	190	226	623	315	644	471
15	602	241	612	371	287	178	187	185	639	308	651	465
21	603	248	622	454	297	177	201	181	656	391	660	497
30	598	226	619	463	235	155	142	134	654	346	668	460
40	603	292	625	487	192	162	164	117	645	391	660	517
50	591	280	619	494	221	150	150	139	662	354	674	508
60	583	266	600	504	212	174	168	130	652	397	672	504
80	597	240	609	492	205	176	159	127	656	410	669	564
100	576	282	596	482	133	137	147	109	621	388	652	556
125	598	306	583	519	114	126	140	97	631	415	650	545
160	599	305	580	528	118	120	142	95	624	396	644	551
200	588	306	578	532	139	118	120	106	622	439	629	577

Table 6A2: Rebar Potentials for Mild and High Yield Steel (-mV)

CONCRETE TYPE		OPC ONLY			30 % FLY ASH			50 % SLAG		
GRADE (MPa)		20	40	60	20	40	60	20	40	60
TIME (days)	TEST									
0	Ecorr	179	160	148	157	153	135	149	141	142
	% Cl									
3	Ecorr	168	158	148	151	158	149	148	136	151
	% Cl									
6	Ecorr	187	178	168	168	179	165	165	162	169
	% Cl									
10	Ecorr	528	466	162	472	361	158	475	521	315
	% Cl	2.24	0.92	0.05	1.18	0.53	0.18	2.73	1.08	0.28
15	Ecorr	571	494	165	541	397	155	550	480	315
	% Cl									
21	Ecorr	600	527	164	588	418	172	575	480	300
	% Cl		1.23	0.10		0.52	0.35		1.33	0.37
30	Ecorr	607	518	185	590	395	149	550	465	312
	% Cl		1.23	0.12		0.34	0.14		0.84	0.30
40	Ecorr	610	553	191	592	378	165	570	480	308
	% Cl									
50	Ecorr	614	537	183	583	357	158	538	454	281
	% Cl									
60	Ecorr	611	536	180	557	329	165	508	416	264
	% Cl		2.08	0.16		0.50	0.07		1.47	0.51
80	Ecorr	626	553	171	582	324	215	482	420	263
	% Cl									
100	Ecorr	633	558	209	580	322	246	444	395	232
	% Cl		2.44	0.33		0.18	0.04		0.87	0.29
125	Ecorr	641	550	175	583	318	259	466	397	216
	% Cl		2.12	0.23		0.23	0.06		1.83	0.50
160	Ecorr	634	651	189	567	305	228	439	350	186
	% Cl		2.29	0.25		0.49	0.06		1.82	0.34
200	Ecorr	692	677	206	576	298	236	494	368	248
	% Cl			0.66		0.46	0.31			0.36

Table 6A3: Rebar Potentials (-mV) and Chloride Contents (%) of Concrete

SAMPLE NAME	AGE (days)	DEPTH INTO CONCRETE (mm)					
		0 - 15	15 - 30	30 - 45	45 - 60	60 - 75	75 - 90
GRADE 20 OPC	0	2.48	3.76	3.46	3.66	3.91	3.45
	14	1.07	2.07	2.23	3.25	3.93	3.60
	28	0.84	2.02	2.36	2.99	2.77	3.80
	42	0.69	0.85	1.44	1.86	2.19	3.24
	56	0.45	0.74	0.93	1.48	2.49	3.53
GRADE 40 OPC	0	1.55	2.15	1.71	1.04	0.60	0.38
	14	2.06	1.29	1.72	1.31	0.96	0.55
	28	1.04	0.90	0.91	0.78	0.36	0.17
	42	0.51	0.76	0.75	0.85	0.99	0.59
	56	1.15	1.06	0.81	0.92	0.60	0.53
GRADE 60 OPC	0	1.59	0.99	0.23	0.05	0.03	0.03
	14	1.36	0.48	0.13	0.03	0.03	0.03
	28	0.70	0.31	0.10	0.02	0.03	0.03
	42	0.48	0.22	0.12	0.08	0.03	0.03
	56	0.23	0.12	0.08	0.05	0.03	0.03
GRADE 20 FLY ASH	0	2.28	3.58	2.44	0.83	0.42	0.28
	14	1.98	4.00	2.32	1.19	0.47	0.44
	28	2.77	3.51	2.15	0.88	0.26	0.13
	42	1.58	0.86	0.69	0.53	0.50	0.29
	56	0.96	0.87	0.97	0.55	0.18	0.20
GRADE 40 FLY ASH	0	2.47	1.16	0.29	0.09	0.06	0.04
	14	0.85	0.48	0.07	0.05	0.05	0.03
	28	1.54	0.49	0.06	0.04	0.05	0.06
	42	0.86	0.54	0.19	0.05	0.04	0.04
	56	0.58	0.31	0.16	0.14	0.04	0.03
GRADE 60 FLY ASH	0	1.27	0.34	0.08	0.03	0.03	0.03
	14	0.90	0.20	0.07	0.05	0.03	0.03
	28	0.84	0.12	0.03	0.02	0.03	0.03
	42	0.59	0.10	0.04	0.03	0.03	0.02
	56	0.27	0.08	0.04	0.04	0.03	0.03

Table 6A4: Chloride Contents for Electrochemical Extraction (%)

CONCRETE TYPE	EXPOSURE SOLUTION	CHLORIDE BY WT. OF CEMENT AT DEPTH			
		0 - 10 mm	10 - 20 mm	20 - 30 mm	30 - 40 mm
OPC ONLY	Sea water	2.90	2.11	1.27	0.50
	Salt water	3.79	1.83	0.68	0.27
30 % FLY ASH	Sea water	3.57	1.14	0.30	0.08
	Salt water	3.88	0.83	0.18	0.06
50 % SLAG	Sea water	4.41	0.65	0.08	0.06
	Salt water	4.89	0.80	0.06	0.06

Table 6A5: Chloride Contents After Sea and Salt Water Exposure for Six Months

CHAPTER SEVEN

PREDICTIONS OF REINFORCED CONCRETE DURABILITY

7.1 Introduction

The marine environment provides a severe durability test for reinforced concrete structures due to the interaction of several physical and chemical processes. The complex and dynamic interaction of these deterioration mechanisms with concrete makes predictions of durability difficult and durability failures are common-place. The predominant form of deterioration of reinforced concrete structures in the marine environment is chloride-induced steel corrosion, resulting in cracking and spalling of concrete. The activation and propagation of steel corrosion in concrete is influenced by a variety of internal and external factors that complicates assessing the overall risk of corrosion. Few reliable indicators of corrosion have been found and the most effective appears to be the chloride content at the steel surface.

The durability performance of reinforced concrete may be estimated at various stages during the service life of a structure; either at construction, during the initiation phase of steel corrosion or once corrosion damage has occurred. Durability assessments are more reliable when done after extensive exposure but the information becomes less valuable with age as the effectiveness of remedial action reduces. The durability performance of a structure which has experienced extensive deterioration may be accurately determined but at this stage repairs may be expensive and sometimes ineffective due to the advanced level of damage. Early estimates of potential durability are more useful since repairs may be instituted before significant damage has occurred. Ideally, estimates of potential durability should be made at the design stage and checked during construction so that adequate serviceability may be guaranteed before exposure to the environment.

Predictions of durability are often unreliable due to variable material and environmental effects and because of differing interpretations of terms such as durability and service life. Frohnsdorff and Masters stated that 'The prediction of the service life or durability of building materials and components is complicated and has many possible sources of error. The errors are likely to be minimised if the nature of the material or component is well-defined, the service conditions are properly characterized, and good knowledge of the degradation mechanism is available'[1] . Recommendations were made that concepts such as durability and service life be clearly defined and unambiguous so that useful comparisons can be made between various durability studies.

A number of different approaches have been used to formulate prediction models for concrete durability. Clifton listed five main methods: general experience, comparative assessment with structures in service, accelerated testing, stochastic methods and mathematical modelling [2]. Reviewing these prediction methods, Clifton maintains that there is a lack of verification of these methods on existing structures and recommends that existing concrete structures be used to validate new prediction models. Provision must be allowed in prediction models for modifications to be made in the light of this validation process to improve the proposed models.

Before a prediction model is formulated, a number of aspects need to be considered and suitably incorporated. Byfors proposed a flowchart shown in Figure 7.1 where material and environmental data are combined with an understanding of deterioration mechanisms and test methods to produce a prediction model [3]. Special care needs to be taken when using accelerated test methods so that the results are not misleading or unreliable. Mullick proposed a framework within which accelerated tests may be analysed and modified for use as indicators of potential durability [4].

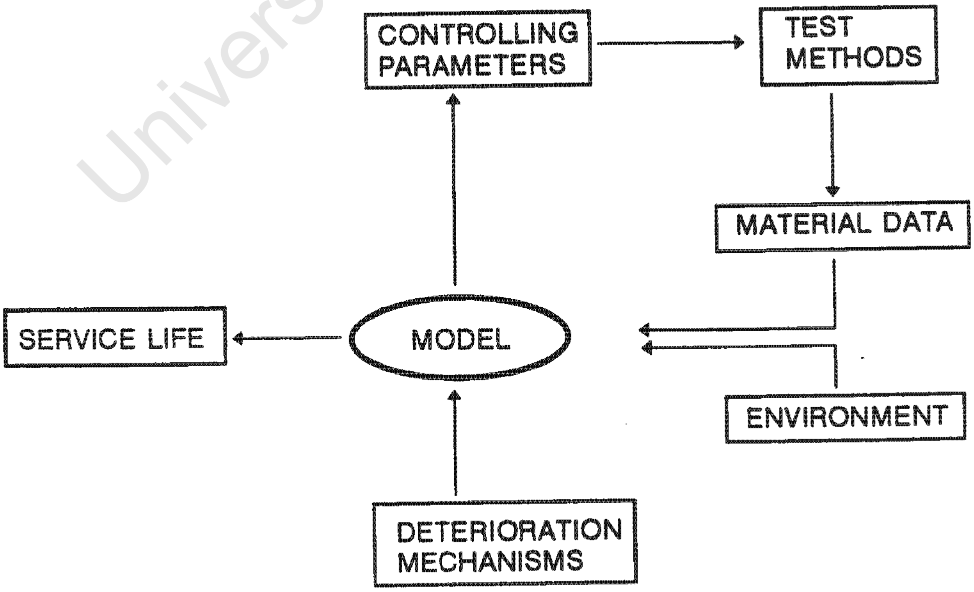


Figure 7.1: Flowchart For Prediction Model of Durability [3]

Predictions of durability are made by defining the material, measuring the environmental conditions and monitoring the durability performance of the material in that environment. There is therefore a requirement that prediction models should be validated with long-term data to ensure that extrapolations are realistic. The model should also be fairly robust so that it can be applied to a variety of materials exposed to a range of environments in order that the predictions have practical use beyond the limited data used to formulate the model. To achieve this the model must have a sound theoretical basis rather than being merely constructed from empirical evidence. Given these constraints it is not surprising that limited progress has been made in formulating reliable prediction models for estimating the durability of marine concrete structures.

Theoretical predictions of concrete durability have not been investigated extensively due to the complexity of deterioration mechanisms, and most prediction models have been empirically based. Theoretical models may provide a greater understanding about deterioration mechanisms and could possibly be used to validate other predictive methods. Empirically based durability predictions may be seen as a pragmatic solution to the complexity of concrete deterioration mechanisms. The approach has the advantage of being useful practically as design recommendations may be formulated but many issues which require a more theoretical and fundamental understanding may be left unresolved. There is however an urgent need for reliable prediction models of practical relevance in the short-term while many of the more complex issues are still being resolved. Highly complicated prediction models may also require extensive input data which may not easily be obtained on construction sites thereby limiting the practical implementation of these methods.

The potential durability of concrete has been implicitly linked to strength and this association still prevails in many durability specifications today despite lack of evidence of a well defined link between the two properties. More recently, the resistance of the cover concrete to aggressive ingress of agents such as chlorides, carbon dioxide and sulphate has been assumed to be the main factor determining the durability performance of reinforced concrete. The premise for this assumption is that the cover concrete protects the reinforcement from those agents which activate corrosion so that their relative rate of ingress may be related to the potential durability performance. This is the basis of several existing prediction models which are reviewed in more detail below.

7.2 Existing Prediction Models - A Critical Literature Review

Early prediction models for estimating the service life of structures were generally based on anecdotal evidence from case studies and test samples. Browne developed a simple nomogram for determining the time to corrosion activation from carbonation and chloride ingress, shown in Figure 7.2 [5]. The family of curves predicts the service life of a range of concrete grades with cover depths up to 100 mm exposed to a temperate marine environment. These design curves are applicable to UK conditions and would probably be inadequate for more severe conditions in the Middle East or tropical climates. The model was only a rough guide and no allowance was made for different cement types or the severity of marine exposure.

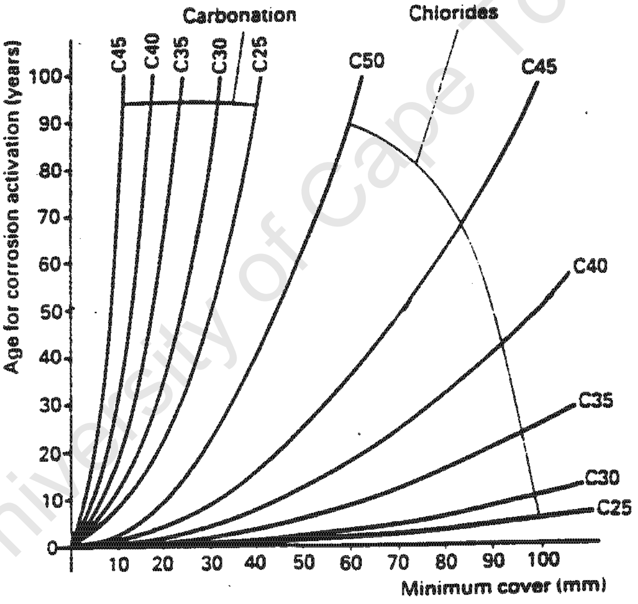


Figure 7.2: Browne's Prediction Model For Corrosion Initiation [5]

Dhir et al used Fick's Law to develop nomograms for estimating future chloride levels in concrete exposed to chloride solutions [6]. These prediction models were based on experimental work where diffusion coefficients derived from classical diffusion tests (two cell, steady state conditions) were compared with diffusion coefficients from chloride penetration tests (diffusion coefficient determined from chloride ingress after penetration). Both diffusion tests exposed concrete to a range of concentrations of NaCl from 0.1 to 5.0 M but no marine exposure work was attempted. Diffusion coefficients determined from the classical diffusion tests were lower than those found from the penetration tests. This discrepancy between diffusion tests should be expected given the differences between steady state conditions of the classical diffusion test and the unsteady state diffusion of the penetration tests. Diffusion coefficients derived from the penetration tests reduced with time and after 12 months were approaching the levels determined using the classical diffusion test. A direct correlation between the diffusion coefficients measured using the two tests was not found but a useful relationship between the two parameters was established, as seen in Figure 7.3.

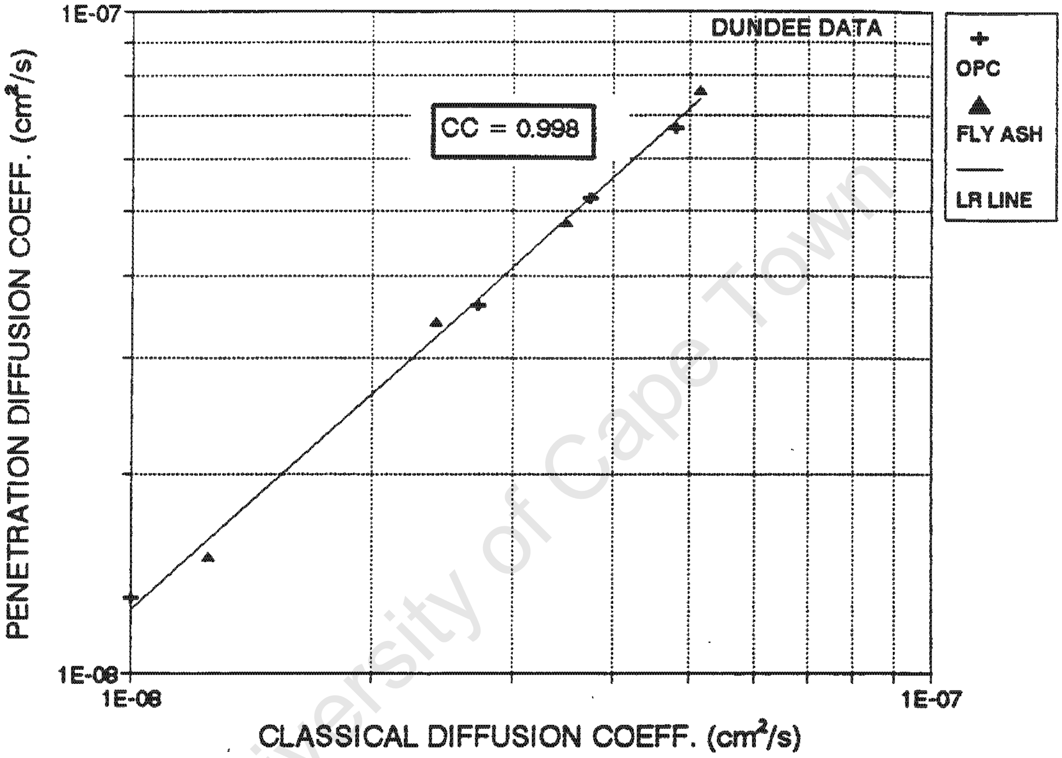


Figure 7.3: Comparison Between Diffusion Coefficients [6]

The practical relevance of these predictions may be questioned since only NaCl solutions were used which means the results might only be applicable to deicing effects and not marine concrete. The classical diffusion test produces a good correlation with results from chloride penetration but the diffusion test may take several months to reach steady state conditions. The extended test period is impractical for site applications and may reduce the deleterious effect of poor curing and inadequate compaction due to continued hydration. A rapid chloride diffusion test was developed by Dhir et al in response to the need for a faster test but several design problems such as hydrogen evolution, corrosion product deposition and acid generation at the anode interfered with the accuracy of the test.

The suitability of using Fick's Law for predicting chloride levels in concrete exposed to the marine environment has been questioned by Mustafa and Yusof [7]. Results from a relatively short-term investigation (12 months) of chloride penetration into concrete in the marine spray zone, indicated that surface concentrations increased with time while diffusion coefficients decreased. The standard solution of Fick's Law assumes the surface concentration is established almost immediately and remains relatively constant together with the diffusion coefficient during exposure. Recommendations were made to modify the solution of Fick's Law to allow for the change in surface concentration and diffusion coefficient with time. The modified equation may be represented as follows:

$$C_x = C_s(t)\left(1 - \mathrm{erf}\left[\frac{x}{2\sqrt{D_c(t)t}}\right]\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7.1)$$

where $C_s(t)$ represents the increasing surface concentration, $D_c(t)$ is the decreasing diffusion coefficient with time t (seconds) and x is the depth into the concrete (cm).

Sandberg and Tang investigated the effectiveness of accelerated test methods in determining the diffusion coefficient of concrete exposed to real service conditions [8]. Laboratory samples were tested using the Danish accelerated diffusion test which involves exposing cut concrete surfaces to a 3M NaCl solution for 35 days followed by measurement of the acid soluble chloride profile and determination of the diffusion coefficient [9]. Chloride testing was also done on a four year old bridge column exposed to tidal action with similar concrete to the laboratory samples. Diffusion coefficients determined from the bridge column were found to be as much as ten times lower than those determined from the accelerated tests. The lack of direct correlation between the accelerated test and real conditions was not expected by the researchers who cast doubt on these tests and Fick's Law.

Diffusion coefficients determined from accelerated tests using concentrated NaCl solutions are abnormally high in comparison to service conditions due to the following effects:-

a) The concentrated NaCl solution does not model sea water diffusion accurately since chloride levels are far in excess of normal levels (102 g/l versus 20 g/l). The absence of ions such as magnesium and calcium would also not allow the formation of insoluble salts and pore blocking at the concrete surface.

b) Short lengths of exposure often result in only the poor quality surface layers being tested (<30 mm) whereas long-term chloride profiles are usually done at a greater depth (50 - 100 mm).

c) Accelerated diffusion tests may start with a partially dry concrete sample which results in rapid absorption initially.

d) Laboratory temperatures are in the range of 20 - 25 °C which is as much as double average site temperatures. Such temperature differences would affect diffusion given that chemical reactions generally double with every 10 °C increase in temperature.

Given the limitations of accelerated chloride diffusion tests and the wide range of exposure conditions on site it is not surprising that there is no direct correlation between diffusion coefficients from accelerated tests and real structures. Simple extrapolations of diffusion coefficients produced by accelerated tests will over-estimate the long-term chloride levels in concrete and may result in restrictive and uneconomical specifications.

Sandberg and Tang found that the diffusivity of laboratory and unexposed field concrete was similar but the diffusivity of exposed field concrete dropped rapidly during exposure [8]. There appeared to be considerable interaction between concrete and diffusant which altered the chloride resistance with time. The results showed that accelerated laboratory tests can only be used as indicators of short-term performance and it was stated that 'Laboratory testing of chloride permeability cannot reveal any useful information regarding service life predictions unless the results are calibrated against long-term field performance of relevant structures in the appropriate environment'.

Further discussion of the use of accelerated tests for service life predictions was instigated by Johansen et al who expressed serious doubts and called for a 'paradigm shift' in the concept of chloride penetration into concrete [10]. Reservations about accelerated chloride diffusion tests and the suitability of Fick's Law were based on the lack of direct correlation between diffusion coefficients from accelerated laboratory and site concrete and the reduction of diffusion coefficients with time. A suggestion was put forward that chloride diffusion did not comply with the boundary conditions implicit in the solution of Fick's Law due to capillary discontinuity which essentially stops further ingress of chlorides into concrete. Chlorides were said to accumulate in surface capillary clusters and further ingress only occurs through cracks which exposed internal capillaries to chlorides.

While the observations about reducing diffusion coefficients with time are generally accepted, the theoretical explanation given by Johansen et al appears to be flawed. Chloride diffusion takes place through the entire pore system (capillary and larger gel pores) which means that capillary discontinuity will reduce the diffusivity of the material but will not stop the diffusion process. The emphasis of the role of capillary discontinuity and cracks is misleading as diffusion will occur through even dense, uncracked concrete. The reduction of diffusion coefficient with time may be better explained by the process of chloride binding and pore structure changes which effectively depletes the diffusant concentration with increasing depth and appears to reduce the material diffusivity with time.

Berke and Hicks developed a simple empirical model for predicting chloride profiles in concrete based on the durability index approach [11]. The predictions were based on a relationship established between rapid chloride permeability values (ASTM C1202 [12]) and diffusion coefficients determined experimentally on laboratory and site data. The predicted diffusion coefficients were modified for temperature and the presence of corrosion inhibitors but other factors such as severity of exposure and material properties were not considered in the predictions. Given the limitations and inacuracies of the ASTM test detailed in Chapter Three, the reliability of these long-term predictions is questionable. The empirical relationship between the rapid chloride test and diffusion coefficients should at least make allowance for the different long-term characteristics of concretes while no allowance was made for diffusion coefficients reducing with time.

Mangat and Molloy used a variety of simulated and actual marine exposure environments to show that concrete diffusion coefficients decrease with time of exposure [13]. The rate of decrease of diffusion coefficient was dependent on the type of concrete and a modified solution of Fick's Law was proposed. The theory may be summarized as follows:

Fick's second law of diffusion states

$$\frac{\partial C}{\partial t}=D_c\frac{\partial^2 C}{\partial x^2}\text{..........................}(7.2)$$

where D_c is the diffusion coefficient, C is the concentration, x is depth and t is time. For unidirectional diffusion into a semi-infinite medium, the standard solution is as follows

$$C_x = C_s\left[1 - \mathrm{erf}\left(\frac{x}{2\sqrt{D_c t}}\right)\right] \quad \text{..........................(7.3)}$$

where C_x is the concentration at time t and depth x and C_s is the surface concentration.

Mangat and Molloy observed a linear relationship between the logarithms of diffusion coefficient and time which gave rise to the following equation

$$\log D_c = \log D_i - m \log t \quad \text{............................(7.4)}$$

where D_i is the diffusion coefficient at time 1 second (y axis intercept) and m is an empirical material coefficient related to the reduction of diffusion coefficient with time (slope of the line). This gives an empirical relationship of the form

$$D_c = D_i t^{-m} \quad \text{..(7.5)}$$

Combining equations 7.2 and 7.5 gives

$$\frac{\partial C}{\partial t} = (D_i t^{-m}) \frac{\partial^2 C}{\partial x^2} \quad \text{............................(7.6)}$$

Rearranging the equation produces the following

$$\frac{1}{(D_i t^{-m})} \frac{\partial C}{\partial t} = \frac{\partial^2 C}{\partial x^2} \quad \text{..............................(7.7)}$$

Defining the denominator of the left hand side of equation 7.7 as follows

$$\partial T = (D_i t^{-m}) \partial t \quad \text{................................(7.8)}$$

Substituting equation 7.8 into equation 7.7 gives

$$\frac{\partial C}{\partial T}=\frac{\partial^2 C}{\partial x^2}\text{...}(7.9)$$

Equation 7.9 has a standard solution as follows

$$C_x=C_s\left[1-\mathrm{erf}\left(\frac{x}{2\sqrt{T}}\right)\right]\text{...............................}(7.10)$$

Integrating ∂T as follows

$$T=\int_0^t D_i t^{-m}\,\partial t\text{...}(7.11)$$

$$T=\frac{D_i}{1-m}t^{(1-m)}\text{...}(7.12)$$

Substituting equation 7.12 into equation 7.10 produces the modified solution to Fick's Law.

$$C_x=C_s\left(1-\mathrm{erf}\left[\frac{x}{2\sqrt{\frac{D_i}{1-m}t^{(1-m)}}}\right]\right)\text{.......................}(7.13)$$

This modified equation allows for more realistic estimates of future chloride levels, particularly for concretes which show a marked reduction in diffusion coefficient with time. The method is however limited to structures already in service since D_c values must be determined from existing chloride profiles. While this method has the advantage of being able to quantify the severity of exposure and material resistance simultaneously, the method cannot be used to predict the potential durability of new structures since chloride profiles must be measured after exposure to the environment. The modified solution of Fick's Law formulated by Mangat and Molloy implicitly assumes that the measured diffusion coefficients are instantaneous whereas in reality measured diffusion coefficients are time-integrated functions. No distinction between instantaneous and integrated diffusion coefficients was given which suggests that the authors did not consider the difference. The resultant error using this assumption is discussed in section 7.3.2.

The use of D_c reduction factors (m) determined from short-term exposure work for long-term predictions must be done with caution due to changing surface concentrations. The reduction factors determined by Mangat and Molloy for fly ash, slag and silica fume concrete were found to be greater than 1.0 which is theoretically impossible assuming a constant surface concentration. Results indicated that within the first 270 days of exposure, surface concentrations had not stabilized. The changing surface concentrations with time had implications on the apparent D_c values determined. This can be observed in Figure 7.4 which shows how misleading the D_c values may be before the surface concentration stabilizes. Surface concentration had not reached stable levels after 270 days of exposure but appeared to be approaching constant values which is in agreement with other findings where stable surface concentrations occurred after one to two years [7,14].

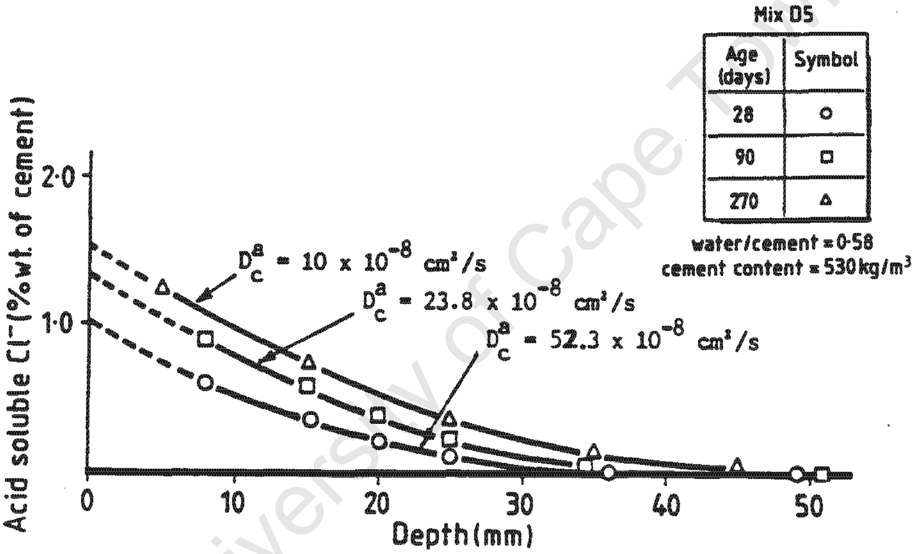


Figure 7.4 : Chloride Profiles For OPC Concrete [13]

The concept of a changing diffusion coefficient with time has been noted by several researchers. The D_c reduction factor, m is a useful parameter for long-term predictions of chloride levels and is dependent on the type and grade of concrete. A range of values for m has been determined as part of this work from the results of several studies of marine exposure samples and marine structures, shown in Table 7.1 and Appendix Table 7A2. Determination of these values was done by regression analysis of plots of the logarithms of diffusion coefficients and time. The slope of the linear regression lines was determined graphically thereby producing the required diffusion coefficient reduction factors for the concretes.

Table 7.1: D_c Reduction Factor, m For Different Concrete Types [13,15,16]

CONCRETE TYPE	RESULTS	DIFFUSION COEFFICIENT REDUCTION FACTOR (m)				
		CONCRETE EXPOSURE SPECIMENS			MARINE STRUCTURES	
		CAPE TOWN	ABERDEEN	FOLKESTONE	S.A.	OTHERS
OPC	LOWEST	- 0.11	0.44	- 0.30		
	HIGHEST	1.35	0.74	0.11		
	AVERAGE	0.40	0.55	-0.08	0.39	0.18
FLY ASH	LOWEST	0.54	0.86	0.72		
	HIGHEST	2.09	1.34	0.94		
	AVERAGE	1.04	1.10	0.81	-	0.67
SLAG	LOWEST	0.24	-	0.78		
	HIGHEST	1.80	-	1.09		
	AVERAGE	0.90	1.23	0.95	-	0.68
SILICA FUME	LOWEST		-	- 0.39		
	HIGHEST		-	- 0.13		
	AVERAGE	-	1.13	-0.28	-	-

The modified solution of Fick's Law formulated by Mangat and Molloy appears to produce higher than anticipated chloride levels due to the erroneous value of the initial diffusion coefficient, D_i derived from equation 7.4. The discrepancy resulted from intregrated diffusion coefficients being applied to a modified solution formulated for instantaneous diffusion coefficients. This results in extremely high initial rates of chloride ingress when using a constant surface concentration. The difference between predicted chloride levels using the modified solution (equation 7.13) and those determined using the standard solution of Fick's Law (equation 7.3) with reducing diffusion coefficients is shown in an example in Table 7.2. In the example, concrete exposed to the marine environment was assumed to have a diffusion coefficient of 1.0E-7 cm²/s after one year's exposure, a surface concentration of 3.0 % and a D_c reduction factor of 0.5. Using equation 7.4 an initial diffusion coefficient, D_i of 1.26E-2 cm²/s was determined and future chloride levels predicted at various time intervals using equation 7.13 in a standard spreadsheet application. Chloride contents were also calculated using the standard solution of Fick's Law using the integrated diffusion coefficient values for comparison (ie. using equations 7.3 and 7.4). Diffusion coefficients were calculated at each time interval using equation 7.4 and the chloride contents obtained from the standard solution.

Table 7.2: Chloride Levels With Changing Diffusion Coefficient

TIME (years)	Dc (cm^2/s)	PREDICT METHOD	CHLORIDE CONTENT AT DEPTH (mm)				
			0	10	20	30	40
0.01	1.0E-06	Mangat	3.00	1.83	1.03	0.52	0.22
		Fick	3.00	0.66	0.04	0.00	0.00
0.1	3.2E-07	Mangat	3.00	2.15	1.49	0.98	0.62
		Fick	3.00	1.40	0.52	0.13	0.01
1	1.0E-07	Mangat	3.00	2.39	1.88	1.44	1.09
		Fick	3.00	1.99	1.24	0.72	0.38
10	3.2E-08	Mangat	3.00	2.57	2.18	1.84	1.54
		Fick	3.00	2.40	1.89	1.46	1.10
100	1.0E-08	Mangat	3.00	2.70	2.42	2.16	1.92
		Fick	3.00	2.65	2.33	2.03	1.77

The modified solution to Fick's Law over-estimates the short-term chloride levels considerably while at later ages the discrepancy is less significant. The difference is due to the measured diffusion coefficients not being instantaneous values but integrated values over the entire time. This means that the instantaneous diffusion coefficient will always be lower than the average levels recorded and the effect will be most noticeable at early ages when there is a rapid decrease in diffusion coefficient. The discrepancy can be corrected by determining the instantaneous diffusion coefficient from the measured integrated value (see section 7.3.2)

Bamforth formulated a prediction model for chloride penetration into concrete, based on data obtained from marine exposure samples and a review of marine concrete structures [17]. The prediction model uses two nomograms, one for selecting the appropriate long-term integrated diffusion coefficient and the other a graphical form of the standard solution of Fick's Law. The long-term diffusion coefficients allow for the reduction with time, particularly for fly ash and slag concrete as shown in Figure 7.5. By using long-term diffusion coefficients which allow for a reducing diffusivity with time it is possible to retain the standard solution of Fick's Law without the modifications done by Mangat and Molloy. The method also provides guidance about the selection of the surface chloride concentration which is assumed to increase with increasing cement content due to the effects of chloride binding.

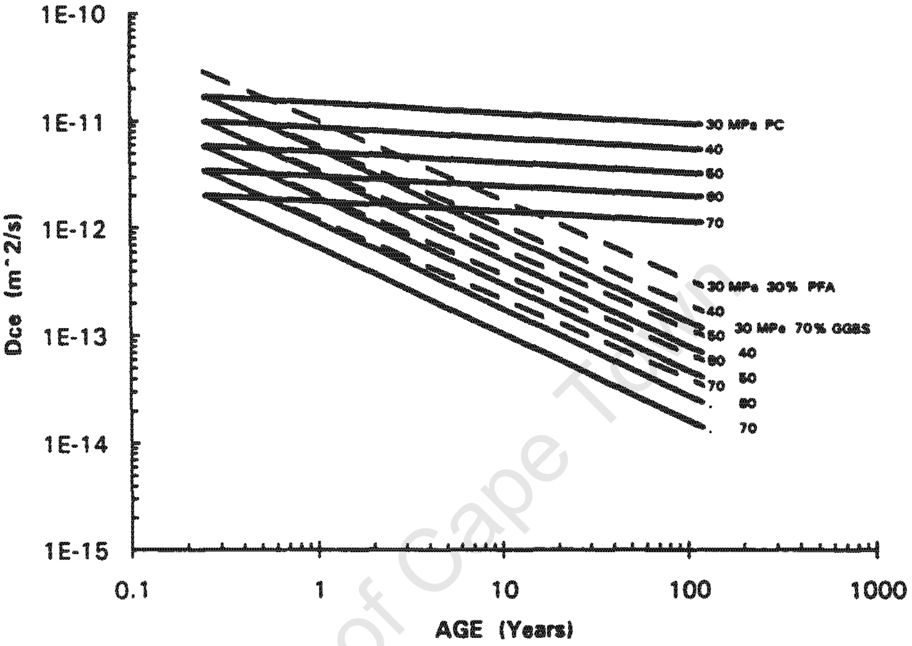


Figure 7.5: Long-Term Diffusion Coefficient Selection (Bamforth)

Reinforced concrete blocks used by Bamforth for the marine exposure work were initially characterized at 28 days using a number of durability tests such as permeability, sorptivity and chloride diffusion [18]. A reasonable correlation was found when comparing the results of the diffusion coefficient determined using a classical diffusion test (see section 3.3.2) and the diffusion coefficient determined after six years marine exposure. The classical diffusion test was not used in the predictive model proposed by Bamforth probably due to the excessive length of the test which may take several months to run.

The prediction model provides valuable guidance for designers of marine structures but is limited in that there is no provision for assessing different quality cementing materials and there is no allowance for differing marine environments. The effects of curing have also been ignored which may be due to the lack of evidence of the detrimental effect of poor curing from exposure results. The method does illustrate the advantage of using cement extenders such as fly ash and slag and may be used to optimize concrete materials at the design stage of a marine concrete structure.

7.3 Formulation of a Prediction Model for Chloride Ingress

For a prediction model of chloride ingress into concrete to be useful, provision must be made for different cementitious materials, construction effects and severity of exposure. The existing prediction models reviewed generally fail to address all of these issues adequately, while some models fail to address any of these factors properly. There is a need for a more suitable predictive model of chloride ingress. From the findings of Chapters Four and Five there appears to be sufficient reason to believe that a more reliable and quantitative model may be devised which could have sufficiently broad application to be practical for specifications and predictions.

7.3.1 Estimation of two year diffusion coefficient

The chloride conductivity test was found to be the most reliable indicator of chloride resistance as well as being sufficiently rapid for practical use on site concrete (Chapters Three and Four). The chloride conductivity of concrete measured at 28 days determined the resistance of the concrete pore structure to chloride ingress but did not adequately quantify long-term chemical interactions such as chloride binding and continued cement hydration. A modification factor was proposed in Chapter Four based on the reduction of chloride conductivity between 28 and 98 days of wet cured concrete saturated in a 5M NaCl solution. After 98 days chloride conductivity values of concrete had stabilized but longer periods may be required for some high performance concretes. This factor allowed the chloride conductivity determined under accelerated laboratory conditions to be related to long-term characteristics of the concrete such as chloride binding, continued hydration and pozzolanic activity.

Using two separate tests (chloride conductivity test and long-term modification test) to determine the modified chloride conductivity has the advantage of allowing the modification factor for a particular concrete to be determined in advance during optimization of the concrete mix thereby allowing rapid chloride conductivity testing to be done during construction. The chloride conductivity test was found to be sensitive to material and construction effects (Chapters Four and Five) and may be used on concrete from the laboratory or site. The method is relatively simple and inexpensive so systematic characterization of concrete could be done allowing new materials and technologies to be assessed and incorporated into designs.

Reasonable correlations were found between the modified chloride conductivity and diffusion coefficients recorded after 24 months marine exposure as shown in Chapter Four. The relationships between modified chloride conductivity and diffusion coefficients for OPC, fly ash and slag concrete exposed at Granger Bay and Simonstown were combined and are shown in graphical form in the experimental nomogram in Figure 7.6. Linear regressions of the data indicate that Granger Bay tidal zone (GBT) was the most aggressive environment followed by the spray zone at Granger Bay (GBS) and the tidal zone at Simonstown (SIM). Results from the spray zone at Simonstown were considered unreliable given the low surface concentrations and low levels of chloride penetration. The nomogram also includes chloride conductivity results determined from moist and dry cured OPC, fly ash and slag concrete at 28 days. Details of the concrete types and curing details are given in Chapter Three.

The experimental nomogram may be divided into three distinct graphs; the concrete grade and curing graph (Chapter Three), the concrete type graph (Chapter Four) and the severity of exposure graph (Chapter Four). The nomogram essentially shows in graphical form the relationship between early age material property of concrete (chloride conductivity) and the medium-term diffusion coefficient (after two years). The two year diffusion coefficient has little intrinsic value except for comparative purposes and as an intermediate parameter from which long-term diffusion coefficients can be determined.

Entry points into the nomogram will depend on whether concrete optimization or a desk-top study is being undertaken. When undertaking a concrete optimization procedure, the start point for predicting diffusion coefficients is the measured chloride conductivity values (the route through the nomogram may be represented by the arrows 1,2 and 3). When doing a desk-top study using data gained elsewhere, the start point for predicting diffusion coefficients is the grade of concrete (the route through the nomogram may be represented by the arrows 1a,1,2 and 3).

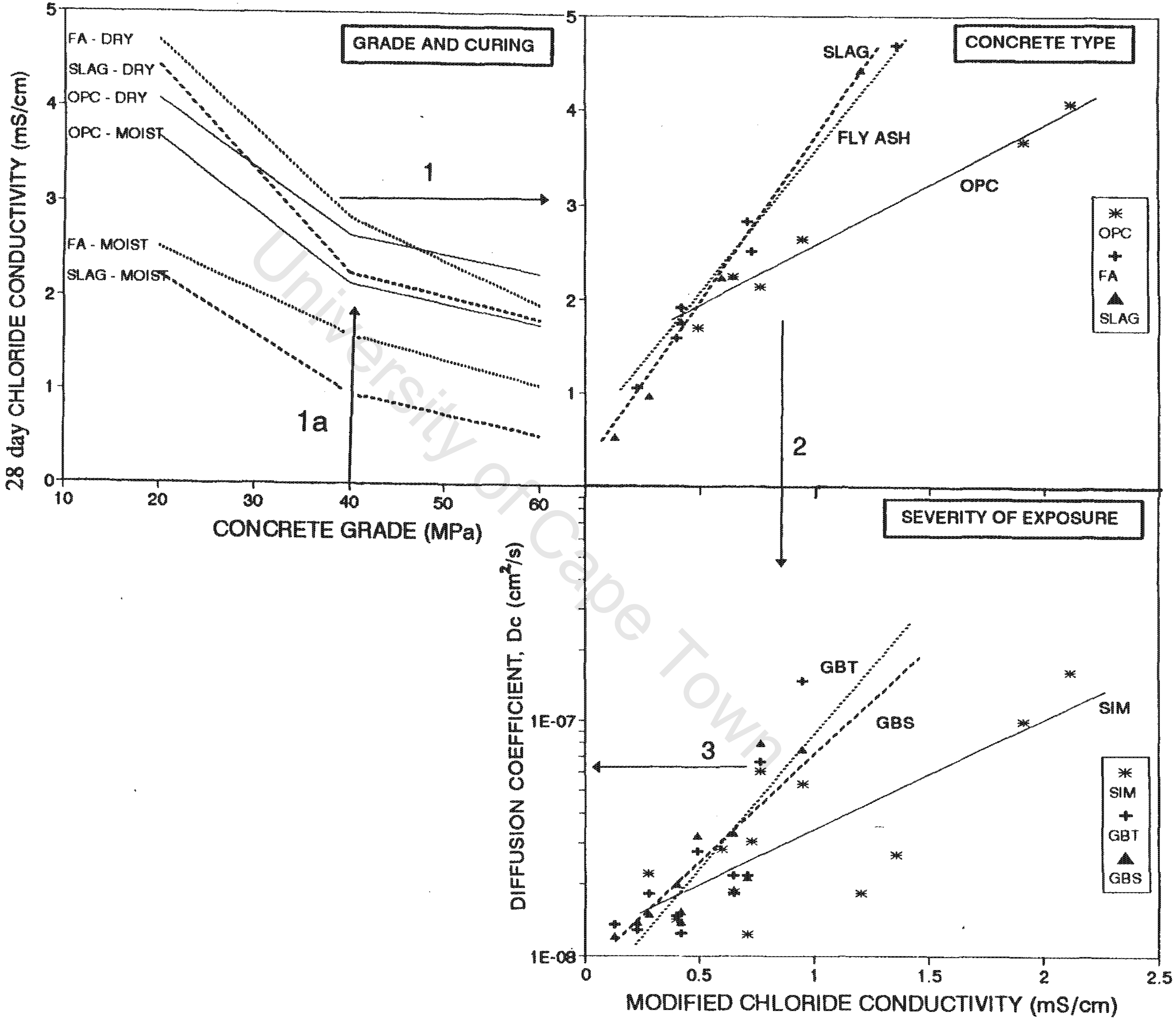


Figure 7.6: Experimental Nomogram For Determining Two Year D_c Values

The variability inherent in the nomogram may be assessed by plotting predicted versus actual diffusion coefficients after 24 months exposure, plotted in Figure 7.7. A large proportion of the variability was due to lower than anticipated diffusion coefficients of dry cured concrete, particularly fly ash and slag concrete. The prediction limits shown represent the range of possible values for predicted diffusion coefficients when estimated from actual diffusion coefficients for a given confidence level. More details of confidence and prediction intervals are given in Appendix 7A6.

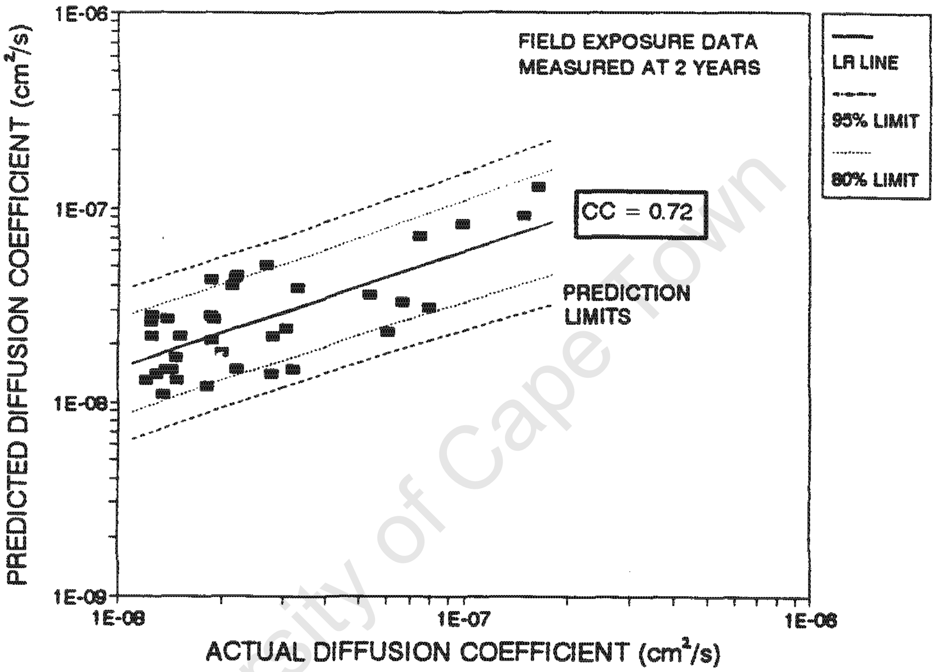


Figure 7.7 : Actual versus Predicted Diffusion Coefficients at 24 Months

Exposure results indicated that the extent of moist curing had only a minor effect on the diffusion coefficient. Early age carbonation was unlikely to have been responsible for the similarity of chloride ingress levels for moist and dry cured concrete. Similar diffusion coefficient results were also reported by Bamforth for concrete exposed to the splash zone in Folkestone [16]. A possible explanation for this phenomenon could be that the concrete was exposed to very moist or saturated conditions after 28 days which allowed additional curing especially for fly ash and slag concrete. The possible detrimental effects of poor curing should not be overlooked simply in light of the marine exposure results since the environmental conditions were limited. Concrete cast in summer in Cape Town may be exposed to severe drying effects initially before regular wetting during winter which might produce more rapid chloride ingress due to poor quality surface layers of concrete.

7.3.2 Long-term predictions of chloride levels (greater than 2 years)

Long-term chloride concentrations may be determined using the two year diffusion coefficient obtained from the experimental nomogram, and knowing (a) the reduction of the diffusion coefficient with time and (b) the surface concentration. The modification of the standard solution of Fick's Law by Mangat and Molloy has been shown to be incorrect as it assumes that measured diffusion coefficients were the instantaneous values rather than the integrated value over the whole time period. This resulted in an erroneous initial diffusion coefficient (D_i) being calculated which over-estimated chloride levels. Equation 7.13 needs to be rewritten in terms of an instantaneous initial diffusion coefficient (D_{ii}) to be correct

$$C_x = C_s\left[1-\mathrm{erf}\left(\frac{x}{2\sqrt{\frac{D_{ii}}{1-m}t^{1-m}}}\right)\right] \qquad (7.14)$$

Comparing equations 7.3 and 7.14 gives

$$D_c t = \frac{D_{ii}}{1-m}t^{1-m} \qquad (7.15)$$

The integrated diffusion coefficient may therefore be written as

$$D_c = \frac{D_{ii}}{1-m}t^{-m} \qquad (7.16)$$

Combining equation 7.5 and 7.16 gives

$$D_i = \frac{D_{ii}}{1-m} \qquad (7.17)$$

The instantaneous diffusion coefficient may therefore be expressed as follows

$$D_{ii} = (1-m)D_i \qquad (7.18)$$

Substituting equation 7.18 into equation 7.14 the correct modified solution of Fick's Law may be defined in terms of integrated diffusion coefficients (ie. measured values).

$$C_x = C_s\left(1 - \mathrm{erf}\left[\frac{x}{2\sqrt{D_i t^{(1-m)}}}\right]\right) \text{..................................(7.19)}$$

Boundary conditions implicit in this solution are that m may only vary between 0 and 1 (when m=0, instantaneous and integrated diffusion coefficients are equal and constant while when m=1, there is theoretically no ingress of chloride into concrete).

The equation (7.19) does not make allowance for an increasing surface concentration with time. Evidence from this research and other studies suggest that surface levels stabilize within the first few years of marine exposure [14]. The rate of increase of surface concentration would appear to be dependent on environmental factors such as wetting action, rainfall and drying rates which are difficult to quantify. The effect of ignoring an increasing surface concentration when determining chloride levels may be significant at early ages of a few years but is negligible at later ages.

The advantage of the corrected modified solution of Fick's Law (Equation 7.19) is that future chloride levels may be determined at any time and depth while allowing for a reduction of the diffusion coefficient with time. Chloride levels may be predicted directly without having to factor the reducing diffusion coefficient with respect to time. The alternative approach is to determine the long-term diffusion coefficient using equation 7.4 and then apply the standard solution of Fick's Law. This forms the basis of Bamforth's approach where the long-term diffusion coefficient is selected from Figure 7.5 for a particular concrete type and grade together with the relevant length of exposure. Long-term diffusion coefficients may be incorporated in the nomogram for convenience and these are shown later in the design nomogram (Figure 7.13)

The reliability of the prediction model was independently assessed using the results from case studies of marine concrete structures detailed in Chapter Five. Two parameters had to be determined before predictions were possible, the diffusion coefficient reduction factor (m) and the surface concentration (C_s). A rational system for selecting these parameters is discussed below.

a) Diffusion coefficient reduction factor (m)

Diffusion coefficient reduction factors (m) determined from the current marine exposure samples may be unreliable due to increasing surface concentrations and the relatively short exposure periods. Full details of D_c reduction factors recorded at Simonstown and Granger Bay are shown in Appendix Table 7A1 and in Figure 7.8. The D_c reduction factors were determined by linear regression through data points plotted on the log D_c versus log time graph. Results from marine structures had lower m values than marine exposure samples probably due to the influence of early unstable surface concentrations on exposure samples. Results are shown in Table 7.1. Exposure results of fly ash and slag concrete were found to have D_c reduction factors greater than 1.0 which is theoretically not possible if the surface concentration remains constant during exposure.

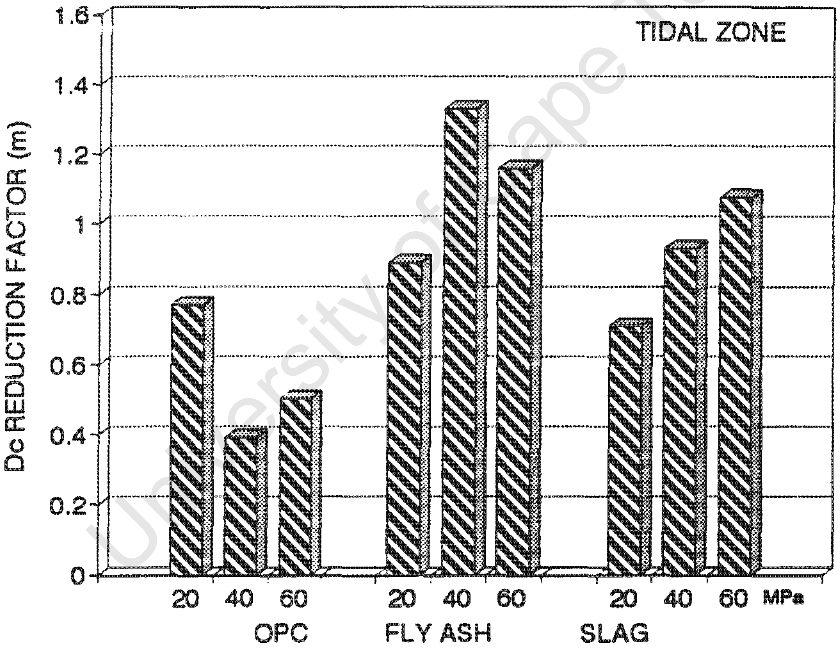


Figure 7.8: D_c Reduction Factors (m) For Exposure Samples - Two Years

Exposure samples were found to have unstable D_c reduction factors which could not be used with confidence so it was decided to use factors determined from case studies of marine concrete from South Africa and overseas in order to be sufficiently conservative (see Table 7.1). These values may be seen to represent long-term conditions as opposed to short-term values determined for exposure samples. Details of the D_c reduction factors for OPC, fly ash and slag concrete determined from case studies are shown below:

Concrete Type	Reduction Factor (m)
100 % OPC	0.29
30 % Fly Ash	0.68
50 % Slag	0.68

b) Surface concentration (C_s)

The surface concentration of chlorides in concrete is reported to be dependent on the cement content, cement type and curing. In the United States guidance is given for determining the surface concentration resulting from deicing operations based on the environmental exposure conditions, with higher levels for increasing severity [19]. The highest surface concentration recommended is only approximately 2.5 % by mass of cement. The reason for linking surface concentrations with exposure conditions appears to be due to the fact that severe environments are exposed to more deicing operations than milder environments. Bamforth proposed a relationship for marine concrete where surface concentration increased with cement content using the premise that 'the surface chloride level is related to the amount of hydrated cement in the surface layer' [20]. Bamforth's design curve which represents the highest expected surface concentrations is shown in Figure 7.9. The top-most data point appears to have a major influence on the design line and if ignored the results may be taken to indicate an almost constant surface concentration by weight of concrete for increasing cement content.

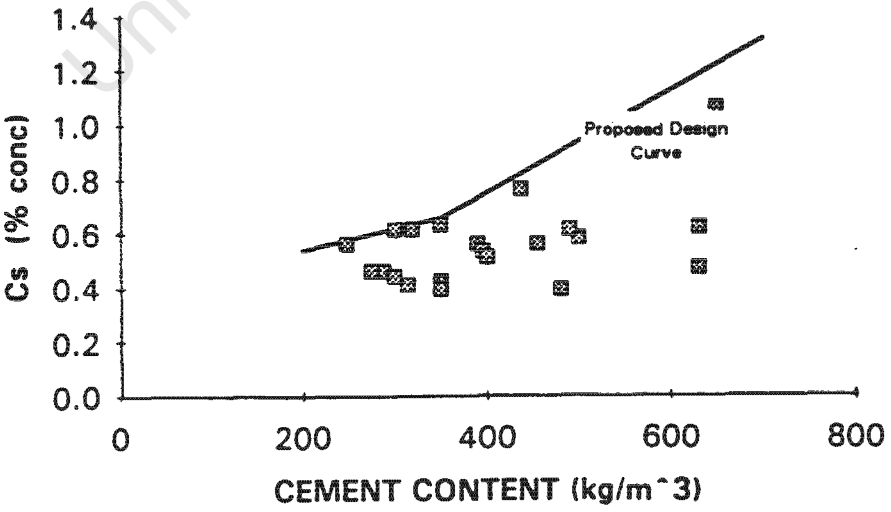


Figure 7.9: Bamforth's Design Curve for Selecting Surface Concentration [20]

Surface concentrations obtained from marine exposure samples at Simonstown and Granger Bay were found to depend on the cement type with fly ash and slag concrete having higher levels than OPC concrete. Surface concentrations for moist and dry cured concrete in the tidal and splash zones are shown in Figure 7.10a and Appendix Table 7A3.

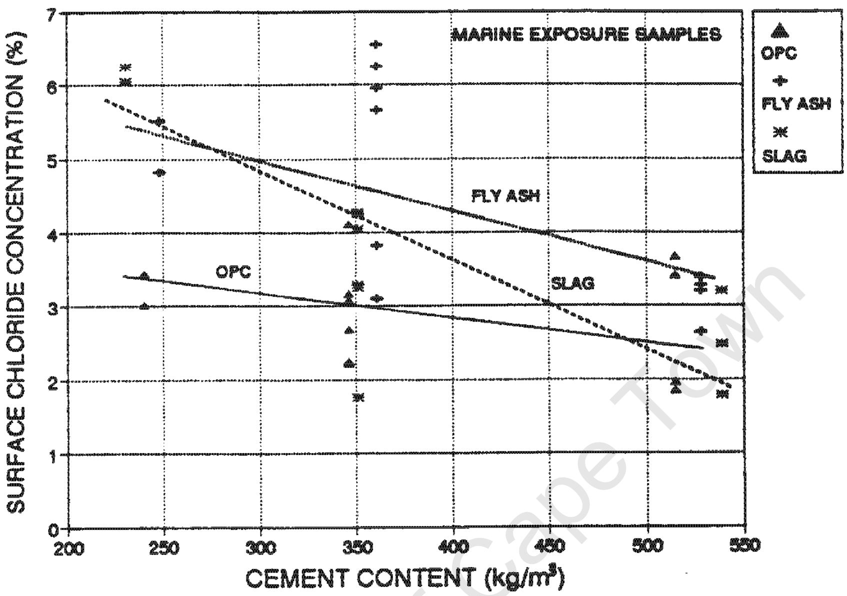


Figure 7.10a: Surface Concentrations - Exposure Samples

Increasing cement contents appeared to reduce the surface concentration, particularly for fly ash and slag concrete. This has been found by some researchers[21] but is contrary to other research where surface concentration increased with increasing cement content [13,20]. Care must be taken when comparing surface concentration data however as values may be expressed either in terms of cement or concrete content. OPC concrete in the tidal and splash zone had surface concentrations of approximately 3.0 % while lower levels were found in the marine spray zones. No consistent trend was established with regard to curing for all three concrete types.

It should be noted that the results represent relatively early age concrete and later testing may produce different trends. The low surface concentrations of high grade concretes in particular may be the result of unstable surface concentrations not yet having reached their maximum long-term concentrations.

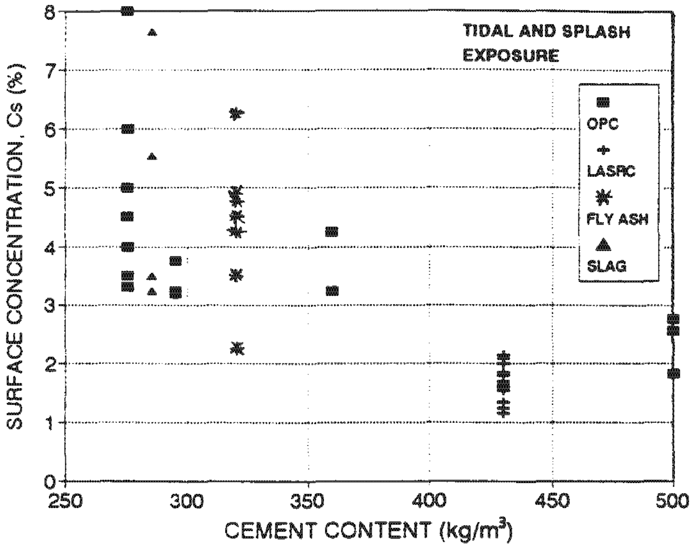


Figure 7.10b: Surface Concentrations - Tidal and Splash Zone Structures

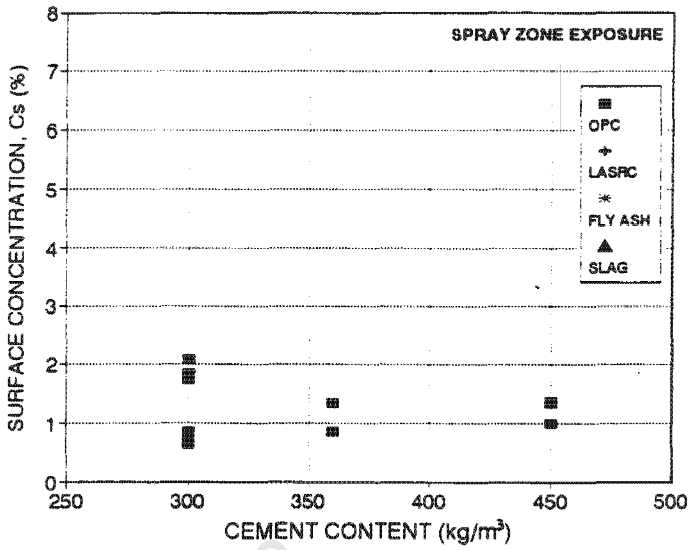


Figure 7.10c: Surface Concentrations - Spray Zone Structures

Determinations of surface concentrations from case studies of South African structures were variable but analysis of average values from each structure showed trends as seen in Figure 7.10b and 7.10c. Observations from case studies revealed that fly ash and slag concrete had higher surface concentrations than OPC in the tidal and splash zones while low alkali sulphate resisting cement had lower values. Similar results have been reported by other researchers [22,23]. OPC concrete exposed to moderate spray zone conditions had low surface chloride levels which were appoximately half of those found in the tidal and splash zones. Surface concentrations were found to be extremely variable particularly for low grade concretes. Concrete with high cement contents generally had lower surface concentrations by weight of cement but it should be noted that these were new structures less than six years old and older structures might have been exposed to salt crystallization at the surface.

Surface concentrations appear to be dependent on the chloride binding capacity of the near-surface concrete together with the interaction of the environment with the concrete pore structure. Concrete exposed to direct sea water action appears to have fairly consistent surface concentrations which is dependent on the cement type. Concrete exposed to wind-borne chlorides appears to have more variable chloride levels which are linked to salt deposition rates, rainfall, drying and the concrete pore structure. From the results of marine exposure investigations and case studies, surface concentrations were recommended for

Western Cape conditions using exposure categories defined in Chapter Four. No allowance was made for concrete grade or initial curing although these factors are likely to have some effect on surface concentrations. The lack of clear trends in this respect prevented suitable allowance being made for these factors at this stage.

SURFACE CONCENTRATION BY MASS OF CEMENT

Concrete Type	Tidal, splash, very severe spray	Severe, moderate, and mild spray
100 % OPC	3.00 %	1.50 %
30 % Fly Ash	4.50 %	2.25 %
50 % Slag	5.00 %	2.50 %

7.3.3 Validation of Prediction Model

Validation of the prediction model was done using the chloride content results determined from case studies reported in Chapter Five. The validation process was not completely independent of the model formulation since surface concentrations were partly determined from case-study results but other factors were obtained from separate sources. Ideally the validation process should have been done on structures not used in any way to formulate the model to ensure the impartiality of the data obtained. Only structures less than thirty years old were considered as older structures did not have reliable initial data and deterioration in some cases had progressed to an extent where extensive cracking of the concrete had already occurred. Concrete which had received no initial moist curing was assumed to be equivalent to laboratory dry cured concrete while other structures which received moderate levels of curing were assumed to be equivalent to the average of laboratory moist and dry cured concrete results. Low alkali sulphate resisting cement is reported to have lower chloride binding capacity (approximately 25 % lower) than OPC concrete so the modification factor used for chloride conductivity results was reduced by 20 % below that of OPC cement [24]. Input parameters used to determine chloride conductivity and diffusion coefficients for the structures are shown in Table 7.3 and Appendix Table 7A4.

Table 7.3: Prediction of Two Year Diffusion Coefficients

STRUCTURE NAME	AGE (years)	CONCRETE TYPE	Dc REDUCT FACTOR m	DIFFUSION COEFFICIENT (cm²/s)		
				PREDICTED Dc		ACTUAL Dc
				After 2yrs	Current	
KOGEL BAY	3	OPC	0.29	5.2E-08	4.6E-8	5.8E-8
TABLE BAY	6	LASRC	0.29	9.0E-8	6.5E-8	5.1E-8
KOEBERG	12	OPC	0.29	6.9E-8	4.1E-8	7.2E-8
STRANDFONT.	13	15% FA	0.49	3.7E-8	1.5E-8	2.0E-8
CAMPS BAY	18	OPC	0.29	2.6E-8	1.4E-8	1.9E-8
OUDEKRAAL	19	OPC	0.29	9.2E-8	4.7E-8	3.2E-8
EAST LONDON	28	OPC	0.29	1.8E-7	8.4E-8	6.0E-8
EAST LONDON	28	50% SLAG	0.68	5.3E-8	9.8E-9	9.3E-9

The predicted chloride profiles for the structures were determined using details from the original records, from which an initial 28 day chloride conductivity was estimated. The chloride conductivity value was applied in the experimental nomogram to estimate a two year diffusion coefficient. Long-term diffusion coefficients were determined from equation 7.4 using the D_c reduction factor m recommended in 7.3.2 a.

A reasonable correlation was found between predicted and actual diffusion coefficients for the marine concrete structures as shown in Figure 7.11. Surface concentrations and D_c reduction factors were selected based on the criteria listed above and chloride profiles for the structures were predicted for the structures with equation 7.19 using a spreadsheet application. Predicted chloride profiles were found to compare favourably with average chloride profiles measured from each structure. Figures 7.12a-7.12h show the comparison between predicted and average chloride profiles for each structure which would seem to indicate that the prediction model has a good degree of reliability. The reliability of the prediction model is assessed using statistical methods in Section 7.4.

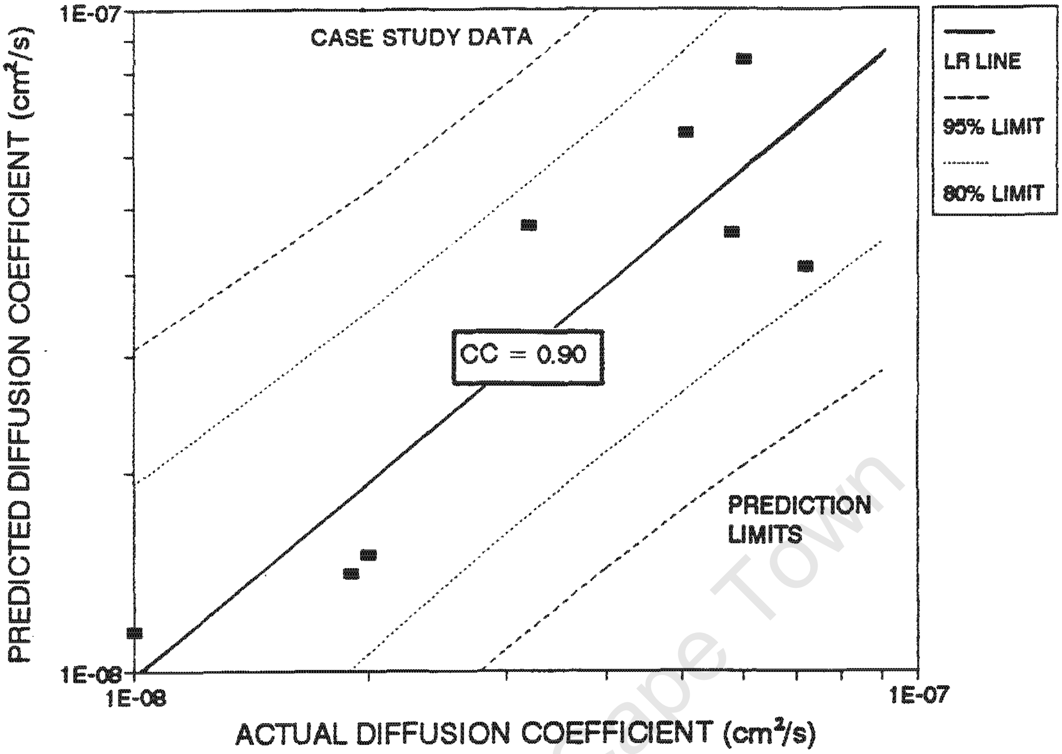


Figure 7.11: Predicted versus Actual Diffusion Coefficients

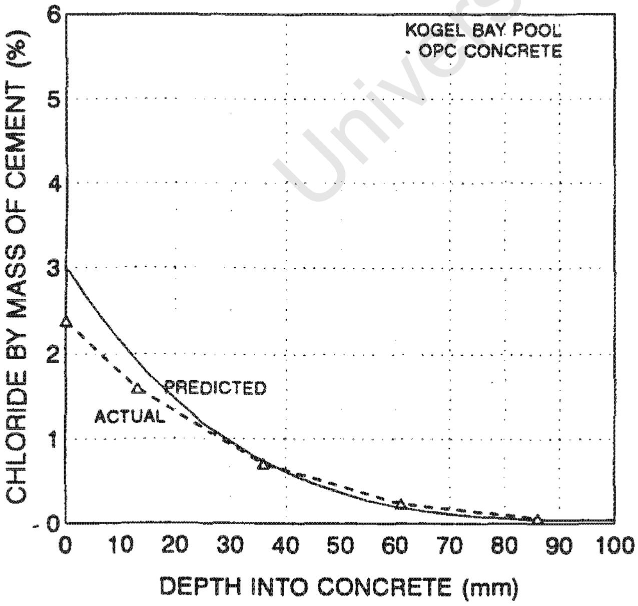


Figure 7.12a: Chloride Profiles - Kogel Bay (3 years)

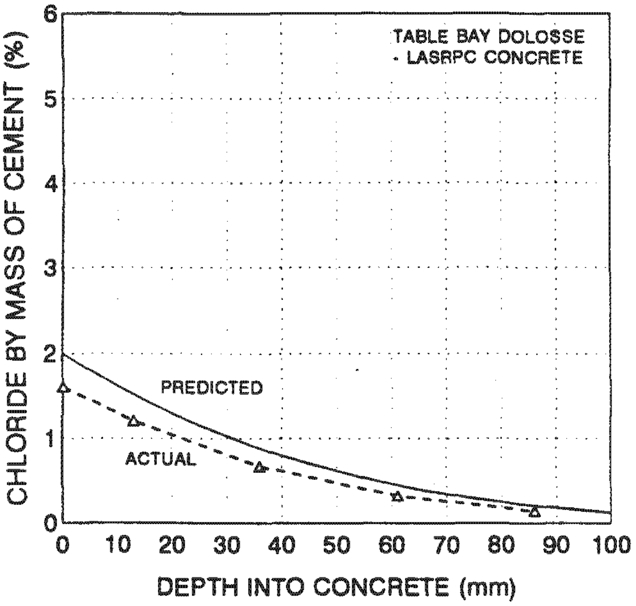


Figure 7.12b: Chloride Profiles - Table Bay Breakwater (6 years)

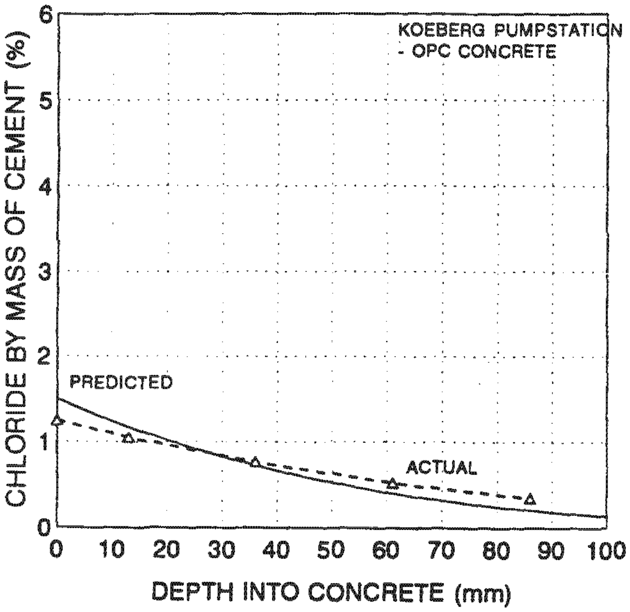


Figure 7.12c: Chloride Profiles -
Koeberg Pumpstation (12 years)

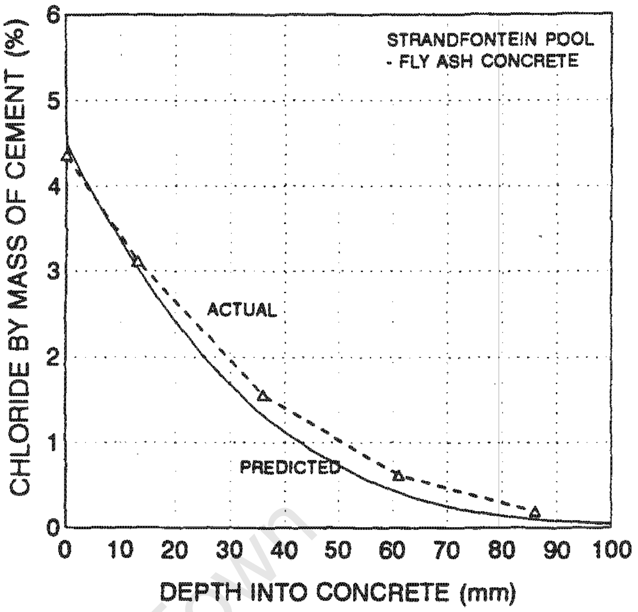


Figure 7.12d: Chloride Profiles -
Strandfontein Pool (13 years)

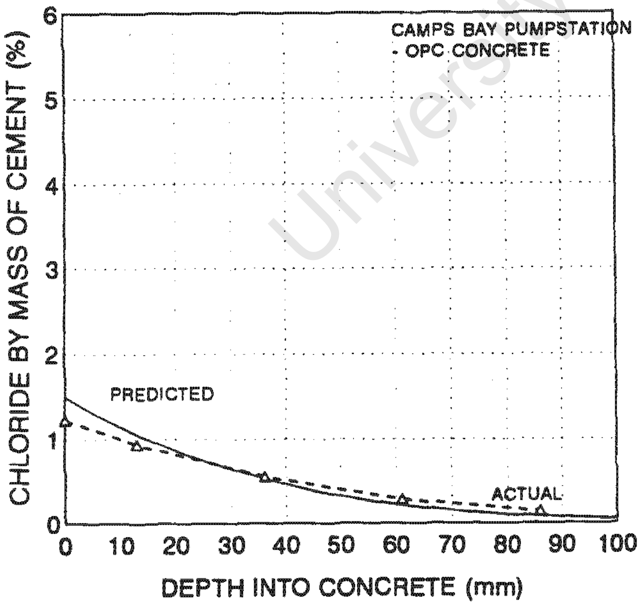


Figure 7.12e: Chloride Profiles -
Camps Bay Pumpstation (18 years)

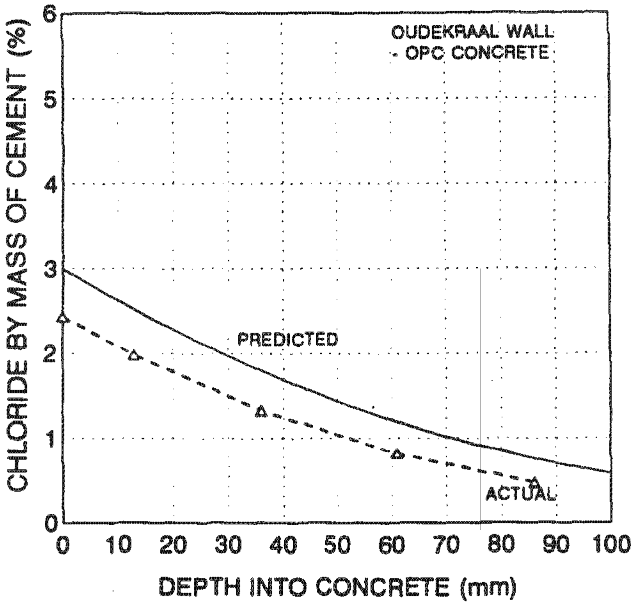


Figure 7.12f: Chloride Profiles -
Oudekraal Wall (19 years)

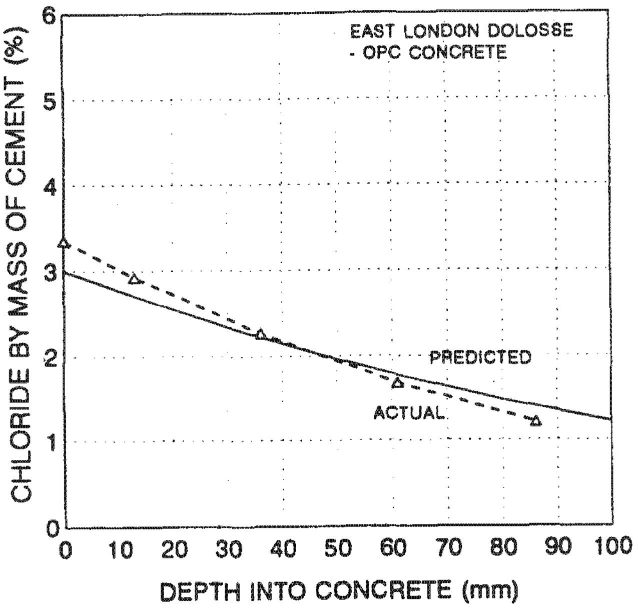


Figure 7.12g: Chloride Profiles - East London OPC (28 years)

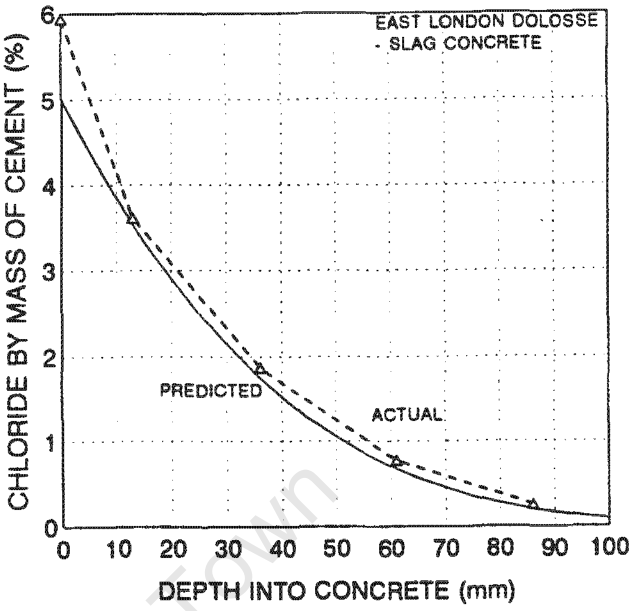


Figure 7.12h: Chloride Profiles - East London Slag (28 years)

Bamforth's method of predicting chloride ingress was compared using OPC and slag concrete exposed to splash action on the East London breakwater. Diffusion coefficients after 28 years exposure were determined using Figure 7.5 (OPC - 1.5E-7 cm²/s, Slag - 6.0E-8 cm²/s). Surface concentrations for the two concrete types were determined using Figure 7.9 and the values converted to a percentage by weight of cement (OPC - 4.9 %, Slag - 5.0 %). Chloride profiles were determined using the standard solution of Fick's Law and are shown in Figures 7.12i and 7.12j. A comparison of actual versus predicted chloride contents indicates that the method over-estimates the chloride levels in OPC concrete but under-estimates levels in slag concrete. The method was however formulated for different climatic conditions and concretes which would help to explain some of the discrepancies.

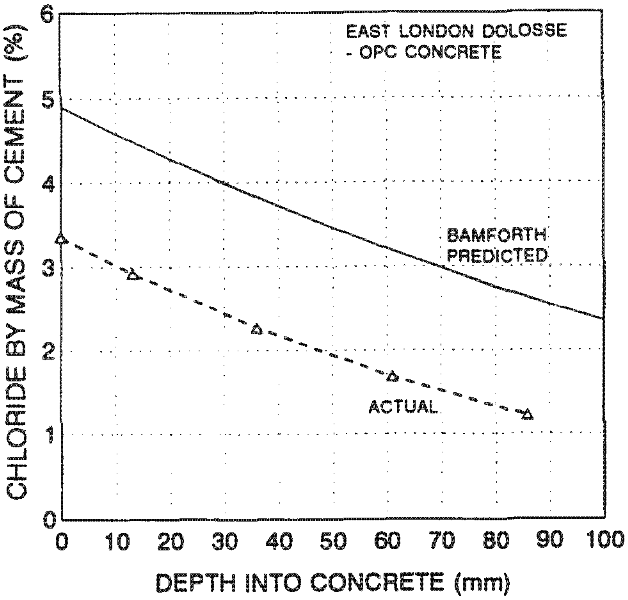


Figure 7.12i: Chloride Profiles - East London OPC (Bamforth)

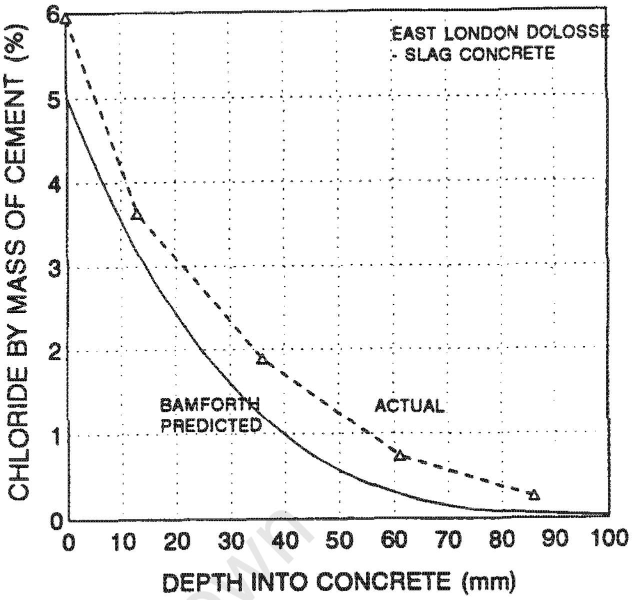


Figure 7.12j: Chloride Profiles - East London Slag (Bamforth)

7.3.4 Refinement of Prediction Model: Design Nomogram

Given the basic soundness of the prediction model, a few minor modifications were proposed to make the model more suitable for general application and in order to construct a design nomogram. The severity of exposure was expressed in general terms rather than being specific to one local environment as shown in the original experimental nomogram. The proposed exposure categories selected were those given in BS 8110 and are defined in Table 7.4 from the results of Chapter Four. The marine exposure categories for tidal and splash conditions are similar to those specified in several codes of practice but considerably more information is given from this work regarding the severity of exposure in the marine spray zone. Given the variable nature of spray zone conditions these recommendations should be used as general guidelines and each site must be examined for local effects which may affect exposure conditions.

Table 7.4: Definition of Exposure Categories (After BS 8110)

EXPOSURE CATEGORY	MARINE TIDAL AND SPLASH ZONES	MARINE SPRAY ZONE
EXTREME	Exposed to sea water and heavy wave action abrasion or heavy splash action	-
VERY SEVERE	Exposed to sea water in a sheltered location or exposed to minor splash action	Adjacent to sea (<50 m) with heavy wave action, strong onshore winds and splashing
SEVERE	-	Structure near sea (50-1000 m) with wave action and onshore winds
MODERATE	-	Near sea (< 1000 m) in sheltered location or inland marine conditions (1-30 km)

Three lines representing extreme, very severe and severe exposure were drawn based on the results of marine exposure samples in the marine tidal, splash and spray zones. The lines represent the upper bound conditions rather than being linear regressions through the data as with the experimental nomogram. No guidance was given for moderate exposure due to insufficient reliable data. The design nomogram for diffusion coefficient predictions is shown in Figure 7.13. Chloride conductivity values may be determined either by a concrete optimization process or typical values for Western Cape materials may be taken from Chapter Three. The nomogram allows two approaches to be used for predictions; two year D_c values may be determined from the nomogram and equations 7.4 and 7.19 used to determine long-term chloride levels; or long-term D_c values may be determined from the nomogram and the standard solution of Fick's Law (equation 7.3) used to determine chloride levels at that age. Using an initial chloride conductivity value to enter the nomogram the route followed to obtain the required diffusion coefficient is along arrows 1, 2 and 3 as shown in Figure 7.13

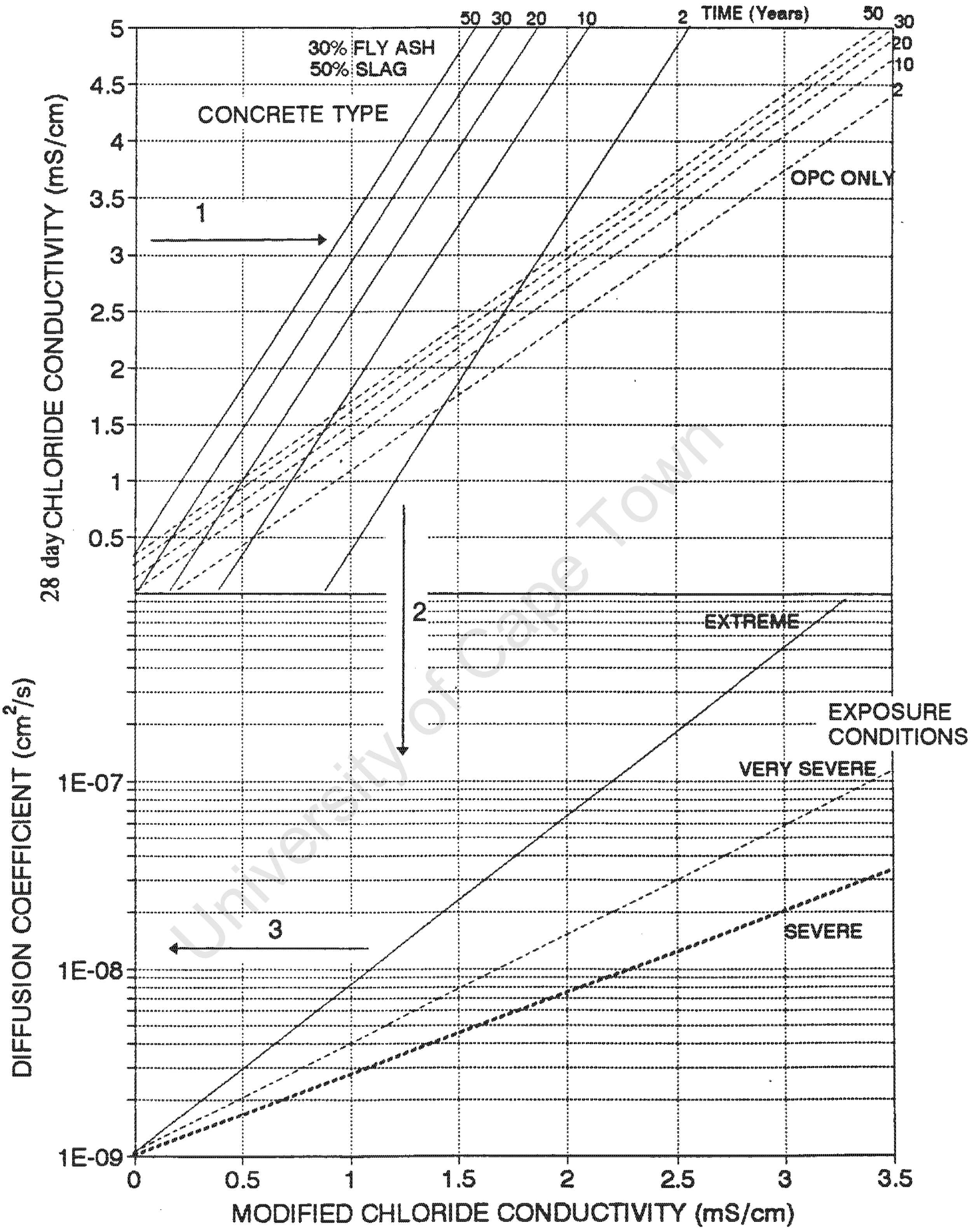


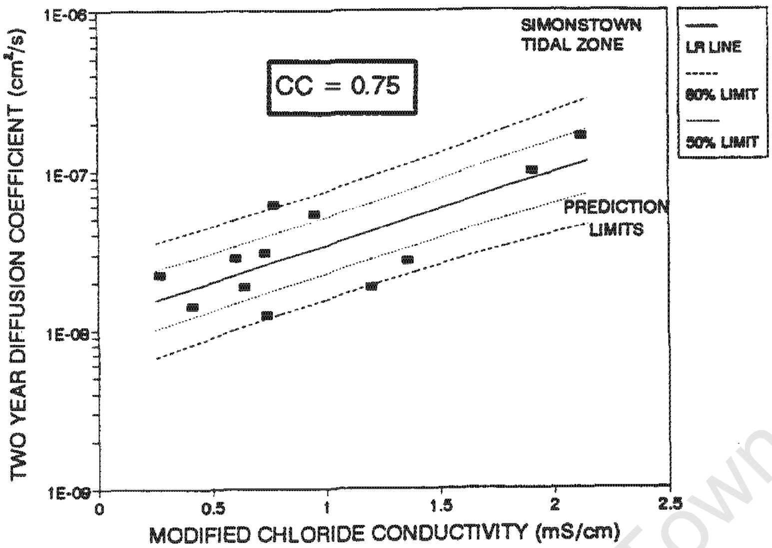
Figure 7.13: Design Nomogram For Predicting Diffusion Coefficients

7.4 Reliability Considerations for Durability Predictions

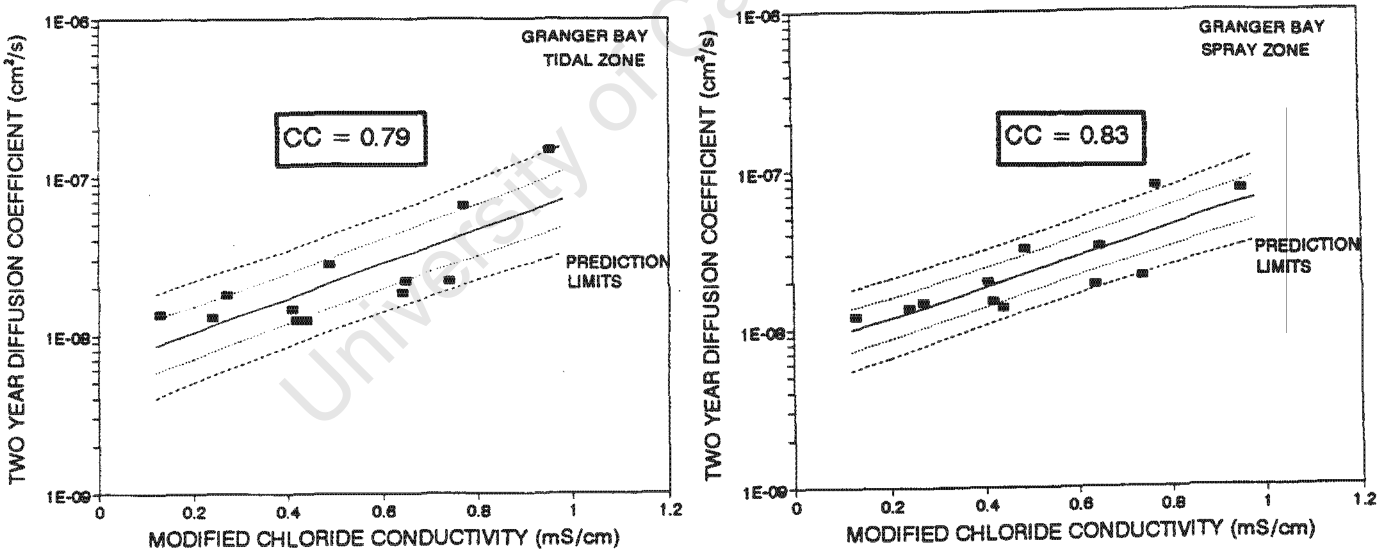
The prediction model allows the long-term performance of concrete to be assessed rather than short-term properties which may have little bearing on durability. There currently appears to be an undue emphasis on short-term properties of concrete using techniques which quantify early age transport properties of the material and assume these to be "durability properties". This approach will naturally favour cementitious materials which mature rapidly such as silica fume concrete and the short-term advantage might not be apparent in the longer term. In order to assess the potential durability of concrete, suitable material characteristics which reflect long-term conditions need to be determined.

The reliability of service life predictions produced from the prediction model need to be assessed in some way before general recommendations and durability specifications are considered using the approach. The prediction model is formulated from a number of sources of data and the resultant model needs to be assessed in the light of the inherent variability of the data. The input parameters all contribute to the overall variability of the model and therefore affect the reliability of the predictions. The three main parameters used in the model are diffusion coefficient (D_c), surface concentration (C_s) and D_c reduction factor (m). The reliability of the prediction model may theoretically be assessed by defining the variability of these parameters and determining the overall effect on the model using a multivariate non-linear analysis. This technique requires extensive data however and may be exceedingly complex when variables are inter-related.

Using the model, two year diffusion coefficients for different concretes and environmental conditions can be determined from modified chloride conductivity values. Figures 7.14 a-c show the correlations between modified chloride conductivity and the two year diffusion coefficients measured from field exposure specimens at Simonstown and Granger Bay. The prediction intervals shown represent the limits with which predictions of diffusion coefficient can be made within a given level of confidence from the modified chloride conductivity data. (Confidence intervals on the other hand represent the limits within which the true correlation between the variables occurs for a given confidence level. The distinction between these two statistical properties is given in more detail in Appendix 7A6). The relationship between the two parameters was based on twelve data points and further observations might help to improve the correlation between the two parameters.



a) Simonstown Tidal Zone

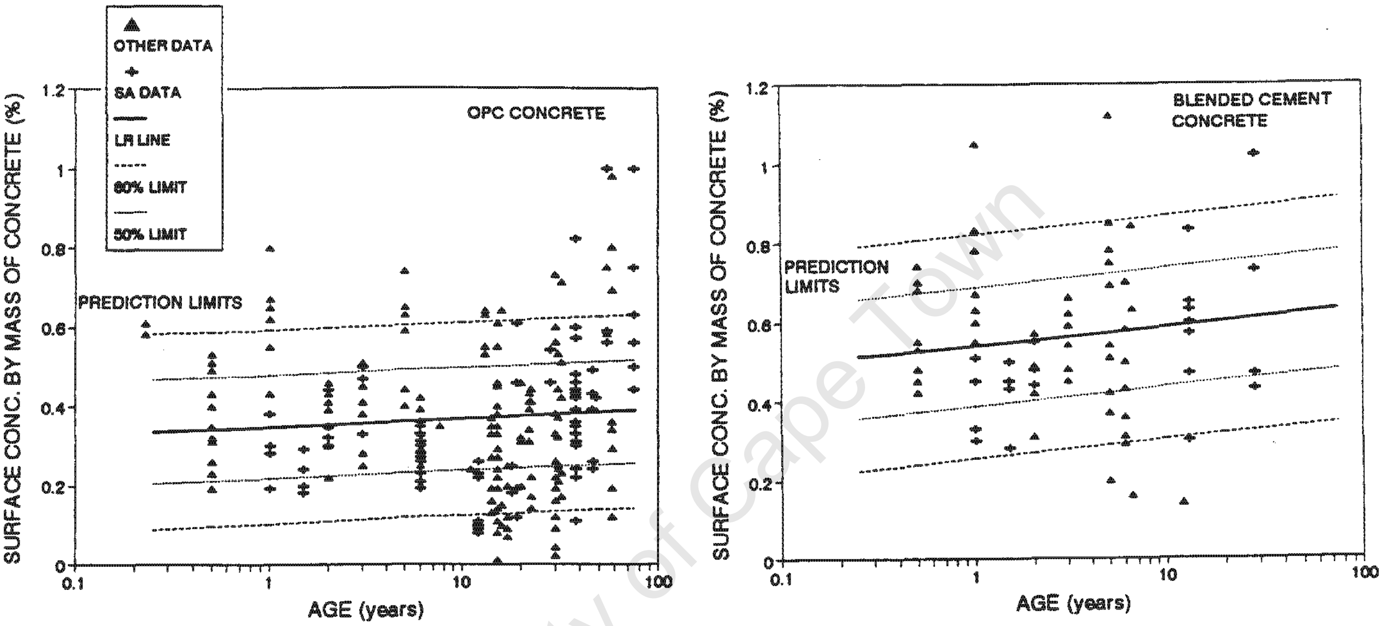


b) Granger Bay Tidal Zone

c) Granger Bay Spray Zone

Figure 7.14: Modified Chloride Conductivity versus Diffusion Coefficient

Surface concentrations recommended in section 7.3.2b were determined from the results of case studies and field exposure specimens using local and overseas data [26]. The variability of the data for OPC and blended cement concretes is shown in Figure 7.15 a and b. Fly ash and slag concrete had similar surface concentrations and results were combined as there was insufficient data to consider the data separately. Surface concentration levels were found to be extremely variable with some structures exhibiting a wide range of surface concentrations despite having similar concrete and being exposed to the same macro-climate. The data does not make allowance for marine zones although evidence presented in section 7.3.2b suggests a link between these properties (this was considered in the prediction model).

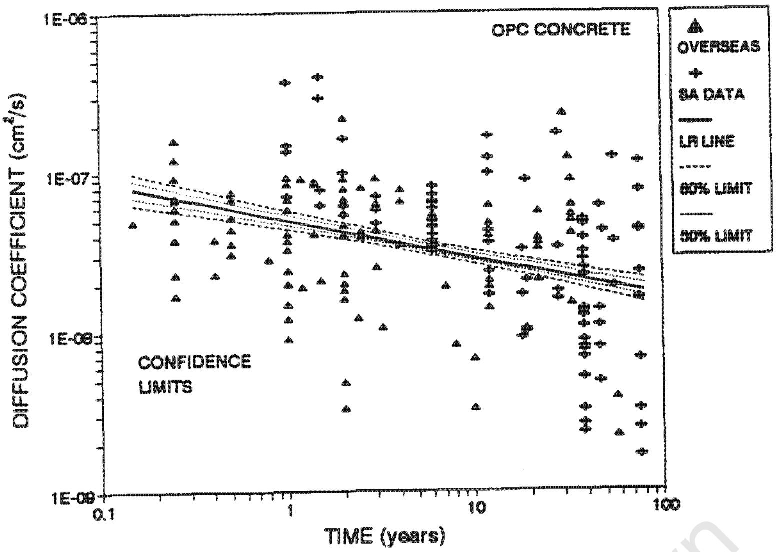


a) OPC Concrete

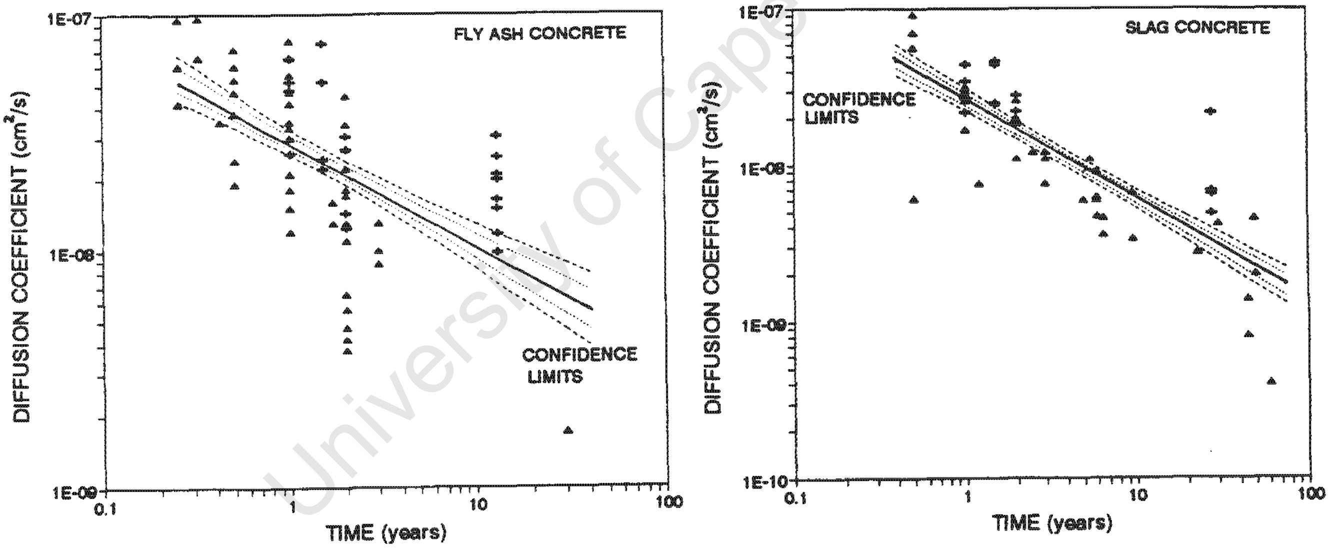
b) Blended Cement Concrete

Figure 7.15: Surface Concentrations versus Time

D_c reduction factors for OPC, fly ash and slag concrete were determined from local and overseas data collected from marine structures and field exposure specimens [15]. The data together with confidence intervals associated with the correlations are shown in Figure 7.16 a-c. Confidence intervals were used since the slope of the linear regression line was important and not the range of data around the line. The observed variability of the data was greatest for OPC concrete and lowest for slag concrete while data for fly ash concrete was limited particularly at older ages (ie ages greater than 10 years). The data shown represents a wide range of concrete grades, binder properties and exposure categories which contributes to the observed variability. D_c reduction factors should preferably be determined from the same concrete but long-term testing is required which would take many years to generate.



a) OPC Concrete



b) Fly Ash Concrete

c) Slag Concrete

Figure 7.16: Diffusion Coefficient versus Time

The reliability of the prediction model may be assessed by using extreme values of D_c, C_s and m in equation 7.19 to determine minimum and maximum chloride levels for a given level of confidence. The approach is complicated by the inter-related nature of the three input parameters and the equation being non-linear. The relationship between diffusion coefficient and surface concentration is shown in Figure 7.17a. The data shown represents a wide range of concrete types and exposure conditions. The general trend would indicate that a high surface concentration would reduce the likelihood of the diffusion coefficient also being high and vice versa. A similar inverse relationship between diffusion coefficient and D_c reduction factor was apparent as shown in Figure 7.17b. The data shown is from marine site exposure specimens located at Simonstown and Granger Bay.

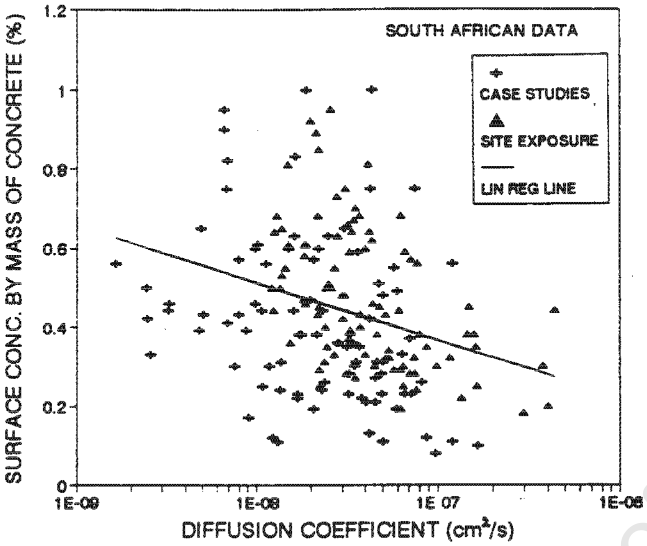


Figure 7.17a: Surface Concentration versus Diffusion Coefficient

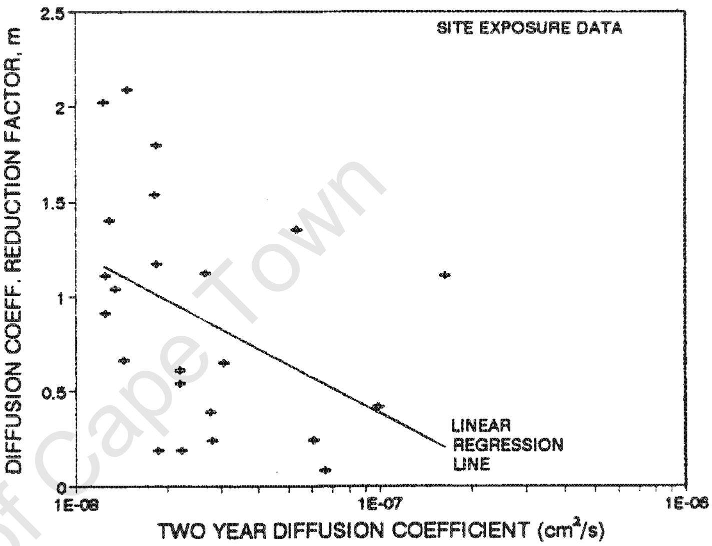


Figure 7.17b: D_c Reduction Factor versus Diffusion Coefficient

The overall reliability of service life predictions when using say, 50% prediction limits of D_c, C_s and m in equation 7.19 is difficult to determine due to many uncertain factors (including the sources of the data, the inter-related nature of the variables and the non-linear equation). Making the assumption that the three parameters are not strongly related together, and that each parameter equally affects chloride ingress, an estimate of the reliability of service life predictions can be made. General probability theory suggests that the confidence inherent in predictions when using the 50% prediction limits for D_c, C_s and m values of concrete is in excess of 90% (probability $= 1 - 0.5^3 = 0.875$, for three linearly related, independent variables). A 10% risk of failure is not excessive for prediction purposes and would represent a 5% risk of early failure occurring. This simplified statistical approach was used since a rigorous multivariate non-linear analysis was not considered viable or practical due to limited data available at relatively early ages.

An example illustrating the range of predicted service lives for concretes with an initial modified chloride conductivity of 0.5 mS/cm is shown in Table 7.5. Mean and extreme values of D_c, C_s and m using the 50% prediction limits of these parameters, were used to generate upper, mean and lower estimates of chloride levels for different concretes and environments. The service life was defined as the time for the corrosion threshold level (0.4% chloride by mass of cement) to be exceeded at a depth of 50 mm. Chloride levels were predicted using equation 7.4 and 7.19 in the standard manner.

Table 7.5: Service Life Predictions for Different Concretes

CASE No.	CONCRETE TYPE AND EXPOSURE	RANGE	MOD. CHLORIDE CONDUCTIVITY = 0.5 mS/cm			
			Dc (cm^2/s)	Cs (%)	m	Time to corrosion at 50mm (years)
1	SIM OPC	UPPER	3.1E-08	3.43	0.21	6
		MEAN	2.0E-08	2.59	0.24	15
		LOWER	1.4E-08	1.75	0.27	43
2	SIM FLY ASH	UPPER	3.1E-08	5.11	0.39	6
		MEAN	2.0E-08	4.13	0.45	17
		LOWER	1.4E-08	3.15	0.51	67
3	SIM SLAG	UPPER	3.1E-08	5.11	0.59	9
		MEAN	2.0E-08	4.13	0.64	54
		LOWER	1.4E-08	3.15	0.68	>100
4	GBT OPC	UPPER	3.1E-08	3.43	0.21	6
		MEAN	2.2E-08	2.59	0.24	13
		LOWER	1.6E-08	1.75	0.27	36
5	GBT FLY ASH	UPPER	3.1E-08	5.11	0.39	6
		MEAN	2.2E-08	4.13	0.45	15
		LOWER	1.6E-08	3.15	0.51	51
6	GBT SLAG	UPPER	3.1E-08	5.11	0.59	9
		MEAN	2.2E-08	4.13	0.64	42
		LOWER	1.6E-08	3.15	0.68	>100
7	GBS OPC	UPPER	3.0E-08	3.43	0.21	6
		MEAN	2.3E-08	2.59	0.24	12
		LOWER	1.7E-08	1.75	0.27	33
8	GBS FLY ASH	UPPER	3.0E-08	5.11	0.39	6
		MEAN	2.3E-08	4.13	0.45	13
		LOWER	1.7E-08	3.15	0.51	45
9	GBS SLAG	UPPER	3.0E-08	5.11	0.59	10
		MEAN	2.3E-08	4.13	0.64	37
		LOWER	1.7E-08	3.15	0.68	>100

Predicted service lives for concrete having an initial modified chloride conductivity of 0.5 mS/cm were found to vary considerably depending largely on cement type and to a lesser extent on exposure conditions. Upper, mean and lower values of the range of service life predictions are shown in Figure 7.18a. More consistent trends were apparent when service lives were normalized with respect to the mean service life value as shown in Figure 7.18b. Considering average results from this data, it would appear that service lives may range from a factor of 0.65 below to 2.9 above the mean (ie. a predicted service life of 50 years could have a range from 18 to 145 years). It should be noted that while the prediction limits represent the range of possible values for a given level of confidence, data is more likely to be scattered around the mean than at the upper and lower limits.

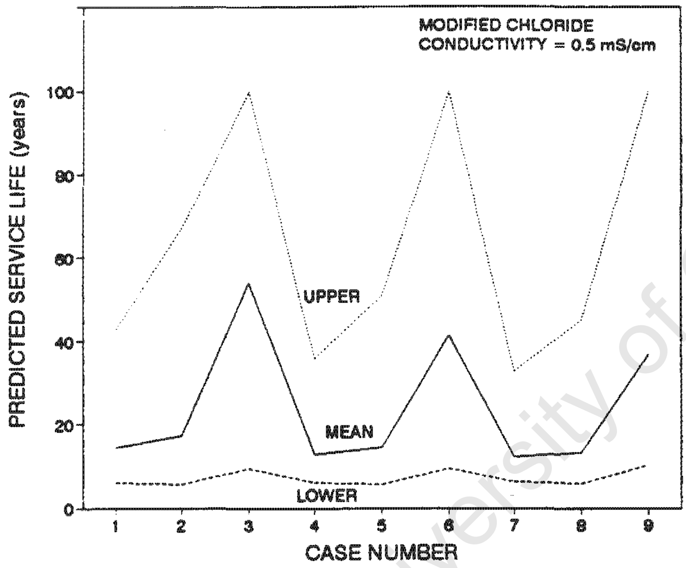


Figure 7.18a: Service Life Ranges for Different Cases

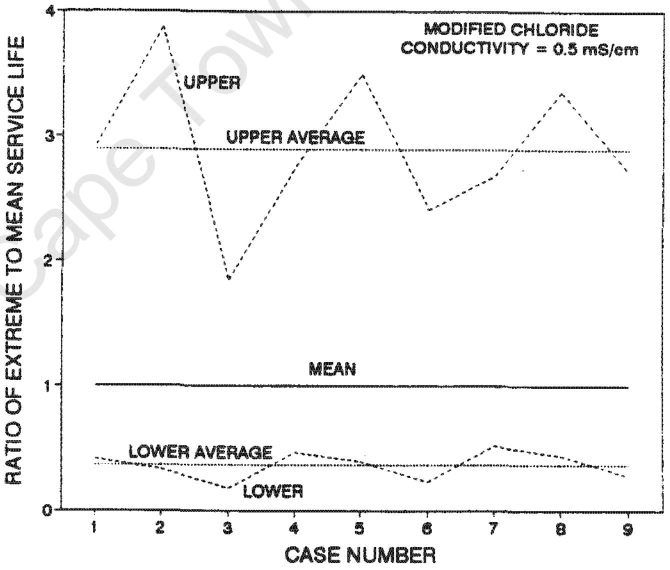


Figure 7.18b: Ratio of Extreme to Mean Service Life

The statistical approach outlined above can only give a broad assessment of reliability and more sophisticated methods using more data would be required for greater accuracy. The complexity of the problem, poor correlations between some variables and the number of factors which need to be considered does not however lend itself to rigorous statistical methods at this stage. It would appear better to rely more heavily on a process of independent validation as outlined in section 7.3.3 to assess the reliability of the prediction model. Results from the validation process were very encouraging with good correlations between actual and predicted chloride levels. These correlations might not have been expected from the findings of the statistical analysis and indicate that the prediction model may be fairly reliable for Western Cape conditions. For instance, from the statistical analysis chloride levels predicted at 50 mm depth in concrete may vary by more than 100% from the mean while chloride levels from the validation process were all within 45% of the mean. The reason why the statistical approach under-estimates the reliability of the prediction model is probably due to the inter-connected nature of the three variables. While it was acknowledged that the variables are inter-connected no specific allowance was made for this relationship when determining the service life ranges.

In engineering and science, models (mathematical, numerical and stochastic models primarily) appear to be replacing more intuitive interpretations of complex processes and it is vital that the underlying principles of these systems are examined in the light of improved understanding of the processes involved. This is particularly important given the confidence that models tend to inspire due to their simplicity and ease of use. Durability predictions by their nature are imprecise and subject to the vagaries of nature and materials. Prediction models must therefore be related to long-term material behaviour using a suitable validation process before any confidence can be placed in the resulting durability predictions. Findings suggest that prediction models should be used to assess general trends regarding durability performance of reinforced concrete and not be used to guarantee service lives within narrow limits.

7.5 Application of the Prediction Model

The general procedure for using the empirical prediction model for both laboratory and site testing is shown in simplified form in Figure 7.19. The prediction model requires a large amount of input data which is crucial to the accuracy of the resultant predictions. The model may be used as part of a desk top study (without laboratory testing) to estimate the durability performance of a range of concrete types without any laboratory testing being contemplated using typical chloride conductivity results shown in the experimental nomogram. More accurate predictions require a limited experimental investigation so that the effects of specific materials, construction practices and curing could be quantified using the chloride conductivity test.

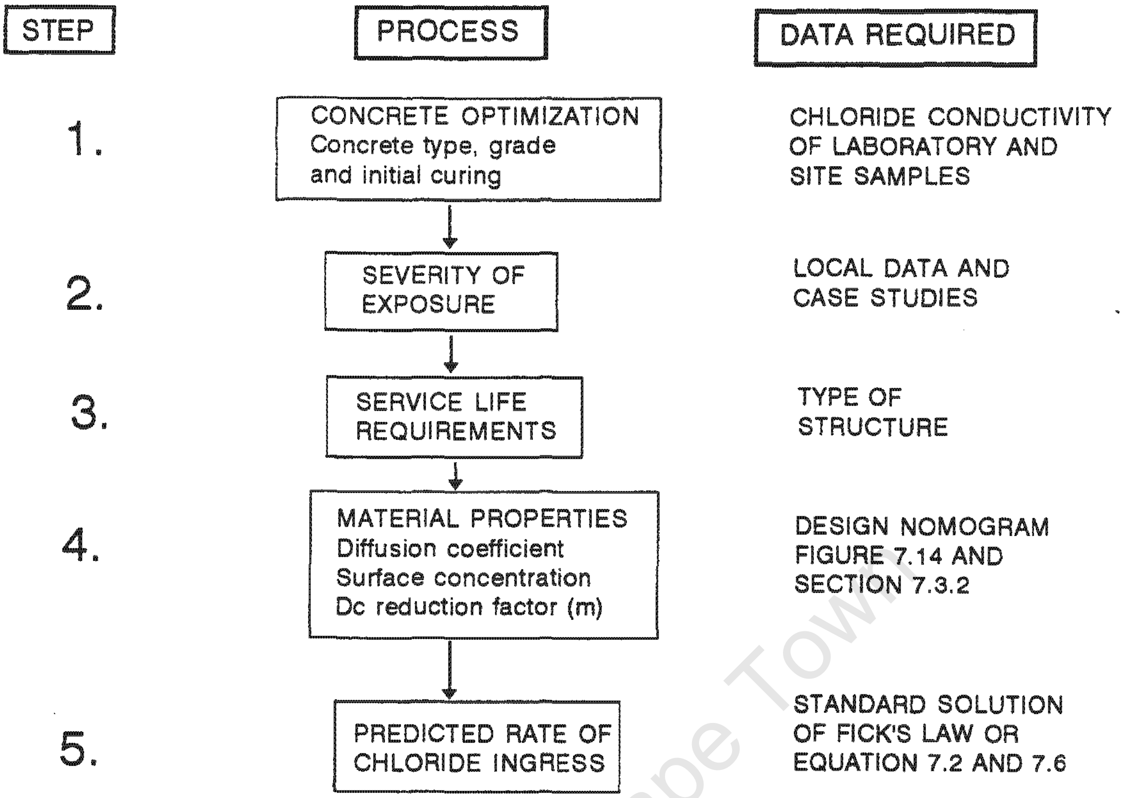


Figure 7.19: Procedure For Using Prediction Model

Using the design nomogram and the modified solution of Fick's Law (equation 7.21) it is possible to predict long-term chloride contents in different marine environments. Table 7.6 shows the predicted medium-term (two year) diffusion coefficients for a range of concrete types exposed to extreme, very severe and severe marine conditions. Long-term chloride levels were estimated for the different concretes and the time to activation of corrosion calculated which would provide an indication of the potential durability of each concrete. Determining intermediate diffusion coefficients (two years) allows more flexibility when used in a spreadsheet application for calculating time to activation of corrosion than having to continuously iterate graphically using long-term diffusion coefficients. This is because once the initial diffusion coefficient, reduction factor and surface concentration have been determined, chloride contents may be determined at any age and depth directly using equation 7.19.

Table 7.6: Predicted Two Year D_c Values

CONCRETE TYPE	EXPOSURE ZONE	D_c VALUE FOR GRADE OF CONCRETE				
		20MPa	30MPa	40MPa	50MPa	60MPa
OPC ONLY	EXTREME	5.7E-7	2.1E-7	6.5E-8	4.9E-8	3.5E-8
	VERY SEVERE	1.4E-7	7.3E-8	4.1E-8	2.9E-8	2.4E-8
	SEVERE	7.2E-8	4.4E-8	2.8E-8	2.2E-8	1.8E-8
30% FLY ASH	EXTREME	8.2E-8	5.2E-8	3.5E-8	2.7E-8	2.2E-8
	VERY SEVERE	4.0E-8	3.1E-8	2.4E-8	2.0E-8	1.7E-8
	SEVERE	2.8E-8	2.3E-8	1.8E-8	1.6E-8	1.5E-8
50% SLAG	EXTREME	6.8E-8	3.9E-8	2.3E-8	2.0E-8	1.7E-8
	VERY SEVERE	3.6E-8	2.6E-8	1.7E-8	1.6E-8	1.5E-8
	SEVERE	2.6E-8	2.0E-8	1.6E-8	1.5E-8	1.4E-8

Corrosion of reinforcement in concrete is affected by several factors but there appears to be general consensus that the qualitative risk of corrosion may be assessed from the level of chloride at the steel surface. Activation of corrosion has generally been found to occur at chloride levels of 0.4 to 0.5 % by mass of cement while high corrosion rates may only occur at higher levels [25]. It may therefore be prudent to define the maintenance free design life as that period where chloride levels remain below the corrosion threshold level. The time to activation of corrosion (assumed to be 0.4 % chloride by mass of cement) was assessed using equations 7.4, 7.18 and 7.19 in an iterative spreadsheet application (Appendix Table 7A5) shown in Figures 7.20a to 7.20i. The advantage of fly ash and slag is clearly illustrated from these results where OPC concrete was found to have inadequate durability in extreme and very severe environments.

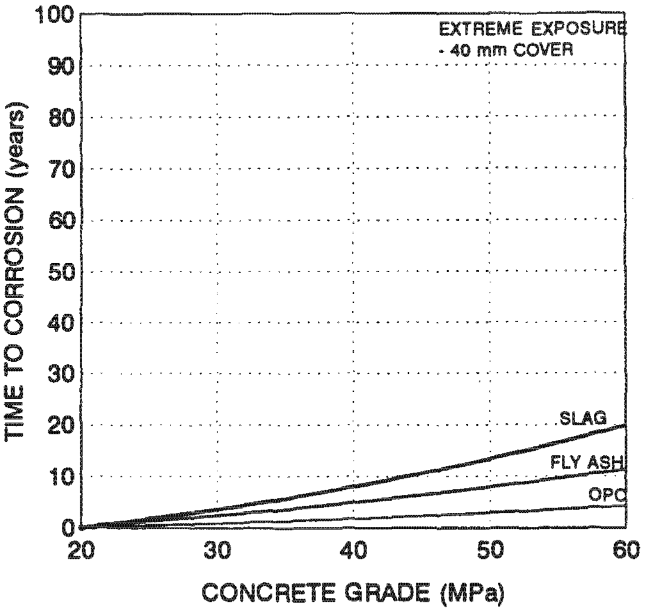


Figure 7.20a: Time to Corrosion - Extreme Exposure, Cover = 40 mm

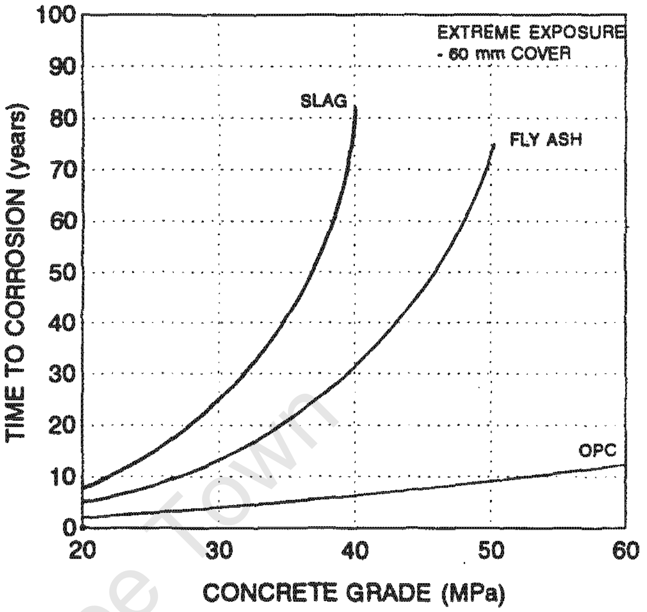


Figure 7.20b: Time to Corrosion - Extreme Exposure, Cover = 60 mm

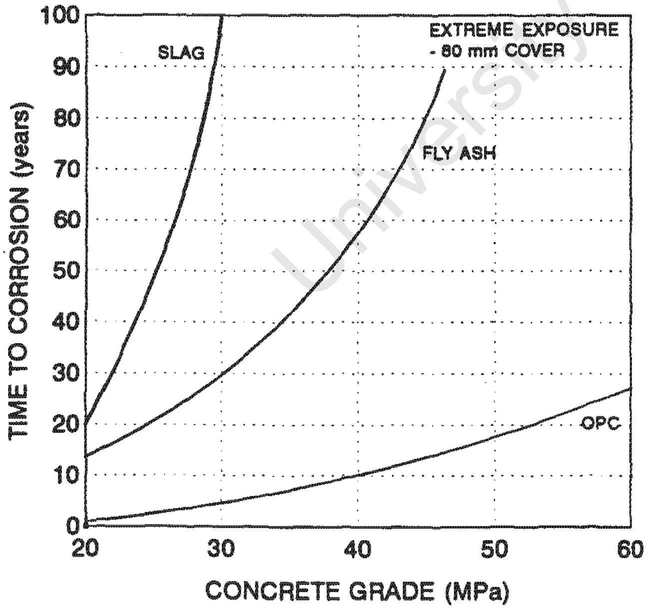


Figure 7.20c: Time to Corrosion - Extreme Exposure, Cover = 80 mm

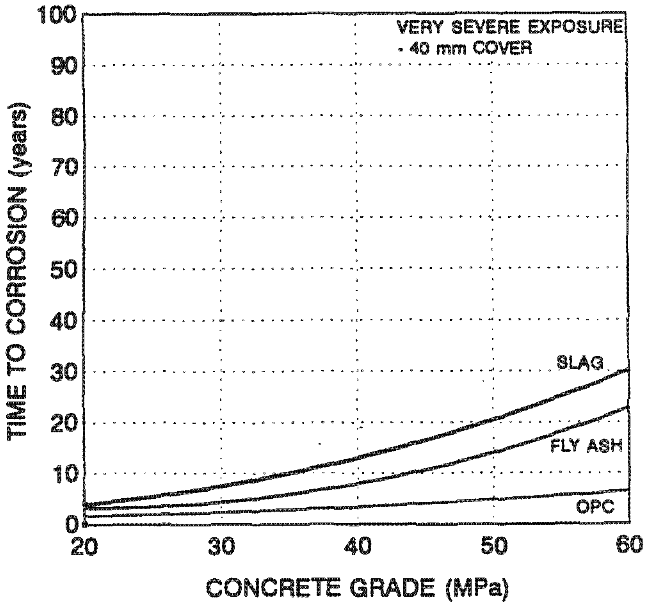


Figure 7.20d: Time to Corrosion - Very Severe Exposure, Cover = 40 mm

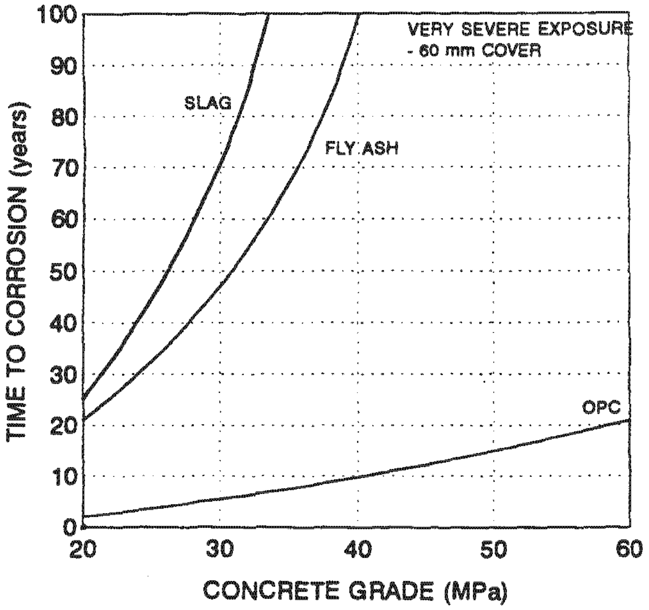


Figure 7.20e: Time to Corrosion - Very Severe Exposure, Cover 60 mm

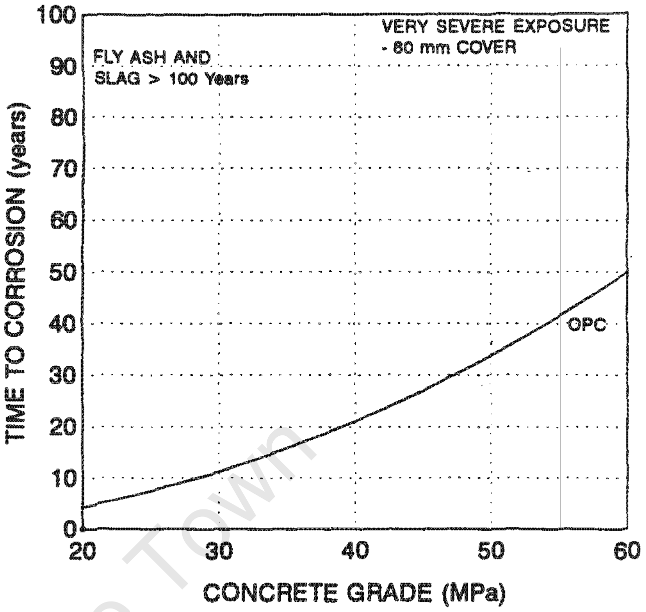


Figure 7.20f: Time to Corrosion - Very Severe Exposure, Cover = 80 mm

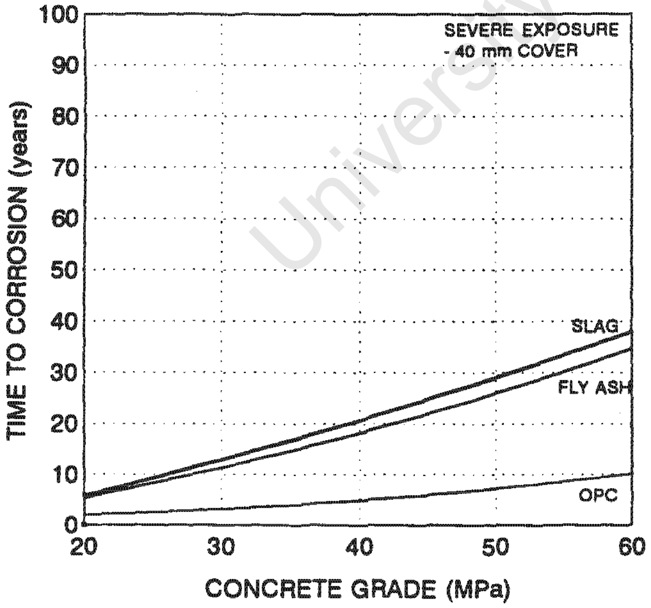


Figure 7.20g: Time to Corrosion - Severe Exposure, Cover = 40 mm

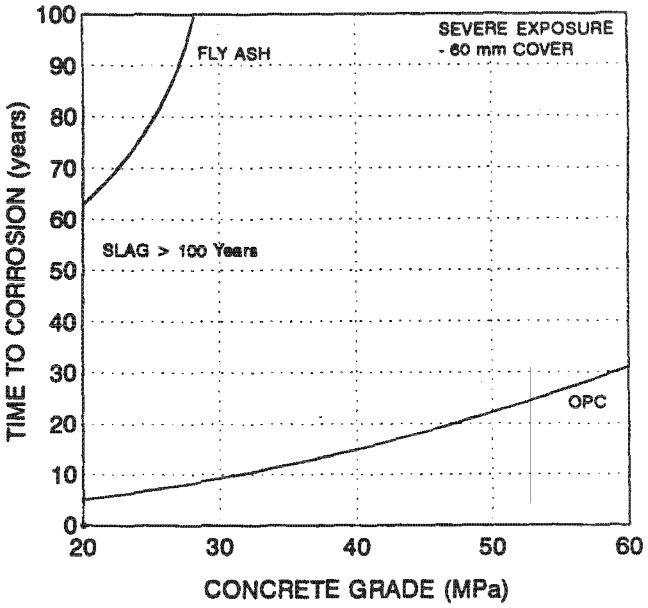


Figure 7.20h: Time to Corrosion - Severe Exposure, Cover = 60 mm

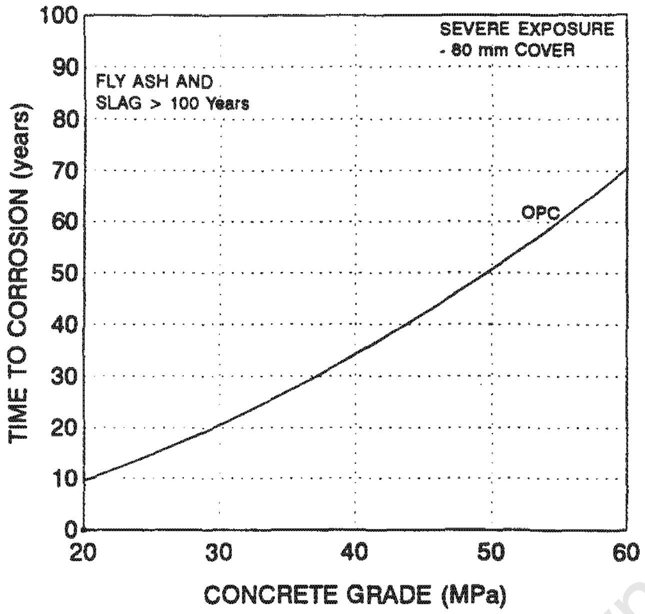


Figure 7.20i: Time to Corrosion -
Severe Exposure, Cover = 80 mm

Recommendations can be made about concrete types, grades, cover depth and curing for different exposure environments but the approach should ideally be performance-based rather than prescriptive so that optimum use of materials and practice may be assessed in terms of the specific structure being considered. The approach described has the advantage of allowing easy optimization of concrete during design and reliable assessment of concrete quality during construction. The advantages of using the prediction model may be illustrated by considering two examples investigated during 1995.

a) Harbour Structure

A harbour structure was built in early 1995 on the Cape West coast using precast concrete elements which were placed into the sea after one month. A grade 40 silica fume concrete (10 % csf dispersed with superplasticiser) was used with a minimum cover depth of 80 mm being specified to withstand the extreme marine conditions. During construction, cover depths were found to vary between 46 and 90 mm and the implication for the potential durability was investigated. Two precast units were tested approximately 28 days after casting by extracting core samples which were tested for core strength and chloride conductivity. Core strengths were found to range from 35 to 40 MPa which was lower than expected for a specified grade 40 material. Chloride conductivity results were between 1.68

and 2.08 mS/cm which indicated a concrete of relatively high chloride resistance. No modification factors were available for silica fume concrete so OPC modification factors were used since no additional chloride binding was expected [25]. Using the average chloride conductivity value of 1.88 mS/cm and OPC modification factors, a two year diffusion coefficient of 2.8E-8 cm^2/s was determined from the design nomogram (Figure 7.13). The assumption that silica fume has similar long-term behaviour to that of OPC concrete was made since silica fume is generally believed to have low chloride binding capacity and relatively short-term reactivity [25].

The service life of the structure was defined as the time taken for the chloride content at the steel to reach a level high enough to cause rapid corrosion resulting in cracking and spalling of the concrete. Rapid corrosion may occur in OPC concrete when chloride levels exceed 1.0 % but the equivalent value for silica fume concrete has not been established. Silica fume is known to substantially reduce the hydroxyl concentration and corrosion threshold levels may therefore be reduced by as much as 50 % below that of OPC concrete [25]. It was decided to use an upper chloride limit of 0.5 % at the reinforcement even though silica fume concrete has been shown to resist corrosion damage better than OPC concrete under accelerated conditions [27]. These conservative assumptions were considered essential since insufficient reliable data has been generated about the corrosion performance of silica fume concrete under normal service conditions. Assuming a surface concentration of 3.0 % and a D_c reduction factor of 0.29, future chloride contents were calculated from equation 7.19 and are shown in Figure 7.21.

Indications from the chloride profiles were that a cover of 80 mm should protect the structure from corrosion damage for almost fifty years but a reduced cover of 60 mm may allow corrosion damage after twenty years. Clearly the cover requirements were not excessive and a lowering of specification could not be justified. Recommendations were made to maintain the specified cover of 80 mm and units with inadequate cover depths should only be used in permanently submerged conditions where steel corrosion rates are likely to be low due to the restricted oxygen concentrations. The testing and analysis allowed the designer to make informed decisions regarding durability and ensured that the likely serviceability of the structure was not reduced by poor construction.

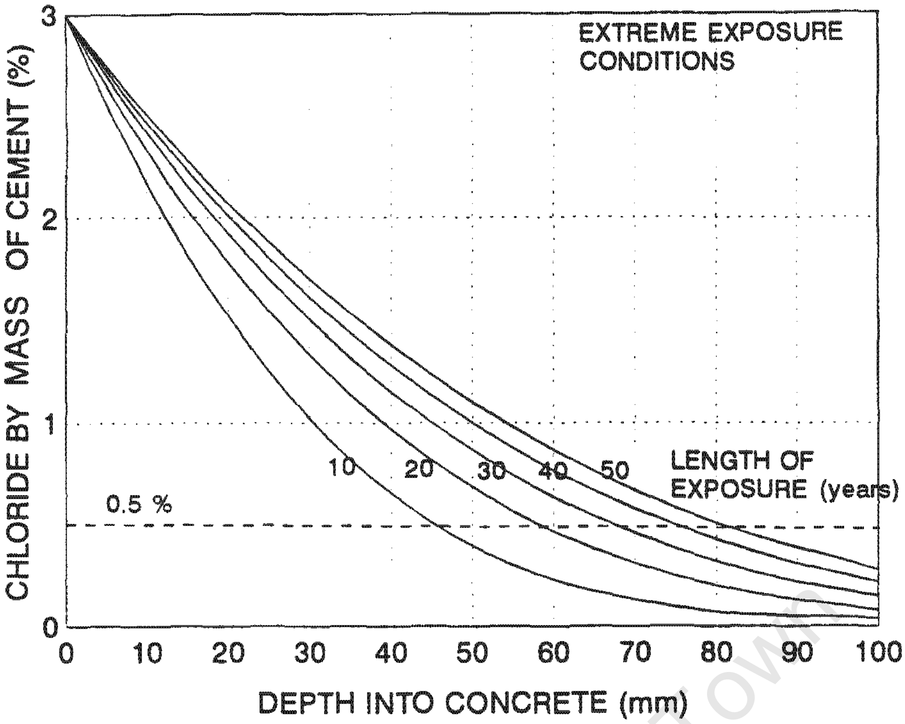


Figure 7.21: Future Chloride Profiles For Harbour Structure

b) Aquarium Building

The durability requirements for a new aquarium building being planned in Cape Town were investigated in order to optimize the concrete materials. The structural design required cover to reinforcement of only 40 mm which therefore needed a chloride-resistant concrete to protect the steel. A barrier coating was to be used on all concrete directly exposed to seawater but provision was made for spillage and normal operating problems by assuming very severe marine conditions for all concrete.

Adequate durability was assumed to be provided by the concrete as long as chloride levels remain below 1.0 % by mass of cement at the reinforcement for a service life of fifty years. Considering the performance of concrete in a very severe environment as shown in Figure 7.20d, it was assumed that OPC concrete would not have sufficient chloride resistance to provide adequate protection to the reinforcement. Using fly ash or slag concrete with a surface concentration of 5.0 %, the standard solution of Fick's Law was applied to determine the diffusion coefficient after fifty years with a chloride level of 1.0 % at a depth of 40 mm. An iterative spreadsheet method was used to determine the diffusion coefficient. The maximum allowable chloride profile is shown in Figure 7.22. The fifty year diffusion coefficient determined was 2.8E-9 cm²/s which was equivalent to a two year diffusion coefficient of 2.5E-8 cm²/s using Equation 7.4 with a D_c reduction factor of 0.68. Plotting backwards on the design nomogram with a two year diffusion coefficient of 2.5E-8 cm²/s for very severe exposure, minimum grades for fly ash of slag concrete were determined.

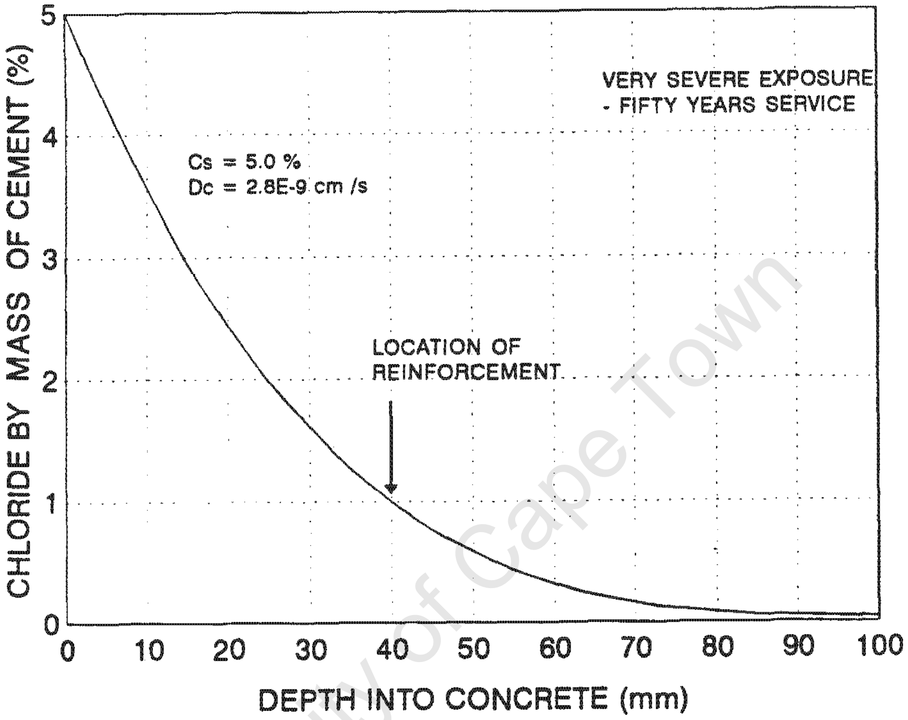


Figure 7.22: Chloride Profile After Fifty Years For Aquarium

Indications from the design nomogram suggest that a grade 30 slag or grade 40 fly ash concrete should provide adequate durability. These recommendations assume reasonable construction practice in terms of curing, compaction and crack control. Low design cover depths require special attention as surface defects may allow easy access of chloride to the reinforcement unless proper supervision is done during construction and neccessary remedial work undertaken before exposure.

7.6 Summary

The proposed prediction model for chloride ingress was found to be reliable when independently validated and has the potential for practical application as a design aid or as part of a performance specification. Since accurate service life predictions are necessary before specifications can be implemented further work is essential to improve the reliability of the prediction model. The predictions have a certain amount of variability associated with the tests and materials but further testing and validation may be able to reduce the inherent variability. A statistical approach was able to help quantify the variations inherent in durability predictions and estimated the reliability of the prediction model. A full multivariate non-linear reliability analysis was not attempted due to the limitations of the data available at this stage.

The basic approach used in the model is similar to several other prediction models but the proposed model is more widely applicable to concrete types and marine environments. Advantages of the proposed model include the following:

- the test procedures are simple, inexpensive and rapid
- allowance is made for different concrete types
- allowance is made for different exposure conditions
- concrete quality may be assessed on site
- the model allows easy optimization of materials and construction practices
- the model allows a more flexible approach for ensuring adequate durability
- the model was validated from case study data
- the basic approach may be expanded and refined.

Limitations of the prediction model which need to be considered and allowed for include:

- empirical nature of the model requires assumptions to be checked theoretically
- the model is based on limited information, further validation is essential
- construction defects such as cracks and poor compaction may nullify predictions
- experience may be required in estimating the severity of exposure
- chloride content may not always be the controlling parameter causing corrosion.
- the variability of data from input parameters is high

7.7 Conclusions

Current approaches to specifying for concrete durability are clearly not achieving the required durability and design recommendations are often misleading. The persistant reliance on concrete grade as an indicator of potential durability without regard to concrete type has been shown to be irrational. Cement extenders such as fly ash and slag which provide greater chemical resistance to concrete than OPC are essential for ensuring chloride resistant concrete. Grade 30 fly ash and slag concrete were shown to have greater chloride resistance than grade 60 OPC concrete and long-term benefits of cement extenders must be considered when assessing the increased capital costs at construction.

An empirical model was proposed for predicting long-term chloride levels in concrete using a modified solution of Fick's Law which allows for a reduction of diffusion coefficient with time. The model relates early-age properties of concrete with long-term material characteristics in order to predict chloride levels which activate reinforcement corrosion. The prediction model allows the long-term performance of concretes to be assessed rather than short-term properties which may have little bearing on durability. Clearly if the potential durability of concrete is to be compared, then long-term characteristics must be assessed. The prediction model is not considered to be the definitive solution for predicting chloride contents in marine concrete but given its wide application should form the framework for further developments and improvements of reinforced concrete durability preditions in the marine environment.

The basic premise of the model is that steel corrosion is the primary source of deterioration and the chlorides are the primary agents causing corrosion. This may not be true in all situations but generally this assumption seems to be fair and should solve many more problems than it creates. Concrete durability should be improved using concrete of high chloride resistance since other properties such as permeability and carbonation resistance should also be improved. A holistic approach should be maintained however to ensure that attention is not focused exclusively on one deterioration mechanism to the detriment of other concrete properties.

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7.9 Appendices

CONCRETE TYPE	GRADE (MPa)	SIMONSTOWN TIDAL		GRANGER BAY TIDAL		AVERAGE VALUE
		MOIST	DRY	MOIST	DRY	
OPC	20 w/c=0.83	0.42	1.11			0.77
	40 w/c=0.56	0.24	1.35	0.08	- 0.11	0.39
	60 w/c=0.38			0.39	0.61	0.50
FLY ASH	20 w/c=0.71	0.65	1.12			0.89
	40 w/c=0.46	0.66	2.02	2.09	0.54	1.33
	60 w/c=0.34			1.40	0.91	1.16
SLAG	20 w/c=0.80	0.24	1.17			0.71
	40 w/c=0.51	0.19	0.19	1.54	1.80	0.93
	60 w/c=0.35			1.04	1.11	1.08

Table 7A1: D_c Reduction Factors (m)- UCT Results

CONC-RETE TYPE	FOLKESTONE DATA				(ABERDEEN DATA)		
	w/c RATIO	CURING TYPE	Dc Red. (m)	AVG. (m)	w/c RATIO	Dc Red. (m)	AVG. (m)
OPC	0.66	WET	- 0.30	- 0.19	0.40	0.44	0.55
		DRY	- 0.06		0.45	0.47	
		MEMB.	- 0.20		0.58	0.53	
					0.58	0.74	
OPC + PLAST.	0.62	WET	0.00	0.05			
		DRY	0.11				
		MEMB.	0.05				
OPC + WATER PROOFER	0.60	WET	- 0.14	- 0.13			
		DRY	- 0.19				
		MEMB.	- 0.07				
30 % (25 %) FLY ASH	0.54	WET	0.72	0.80	0.40	0.86	1.10
		DRY	0.75		0.58	1.34	
		MEMB.	0.94				
70 % (60 %) SLAG	0.48	WET	1.09	0.95	0.58	1.23	1.23
		DRY	0.78				
		MEMB.	0.98				
8 % (15 %) CSF	0.72	WET	- 0.33	- 0.28	0.58	1.13	1.13
		DRY	- 0.13				
		MEMB.	-0.39				

Table 7A2: D_c Reduction Factor (m) - Bamforth, Mangat and Molloy

CONCRETE TYPE	GRADE (MPa)	INITIAL CURING	SIMONSTOWN		GRANGER BAY	
			TIDAL	SPLASH	TIDAL	SPLASH
OPC ONLY	20	MOIST	3.02	0.40		
		DRY	3.45	0.39		
	40	MOIST	3.08	0.48	4.11	2.69
		DRY	2.22	0.42	3.16	2.24
	60	MOIST			3.66	1.85
		DRY			3.42	1.96
30 % FLY ASH	20	MOIST	4.82	1.15		
		DRY	5.51	0.51		
	40	MOIST	3.82	0.19	5.65	6.55
		DRY	3.10	0.52	5.95	6.25
	60	MOIST			3.40	3.27
		DRY			3.20	2.65
50 % SLAG	20	MOIST	6.25	0.22		
		DRY	6.05	0.63		
	40	MOIST	1.76	0.15	3.29	4.28
		DRY	3.25	0.36	4.05	4.25
	60	MOIST			2.48	2.48
		DRY			3.20	1.78

Table 7A3: Surface Concentrations - UCT Results

NAME	CONCRETE		INITIAL CURING (Est.)	MARINE EXPOSURE CATEGORY	CHLORIDE CONDUCT. (mS/cm)	MOD. CHLORIDE CONDUCT.	2 Year Dc (cm^2/s)
	TYPE	GRADE (MPa)					
KOGEL BAY POOL	OPC	55	None	Extreme	2.33	0.82	5.2E-8
TABLE BAY BREAK.	LASRC	50	Minor	Extreme	2.15	1.04	9.0E-8
KOEBERG PUMP-HOUSE	OPC	30	None	Very severe	3.35	1.61	6.9E-8
STRAND-FONTEIN POOL	15% FA	35	Minor	Extreme	2.61	0.71	3.7E-8
CAMPS BAY PUMPST.	OPC	45	Minor	Very severe	2.26	0.73	2.6E-8
OUDE-KRAAL WALL	OPC	35	Minor	Extreme	2.74	1.10	9.2E-8
EAST LONDON BREAK-WATER	OPC 50% SLAG	30 25	Minor Minor	Extreme Extreme	3.10 2.93	1.38 0.82	1.8E-7 5.3E-8

Table 7A4: Long-Term D_c Determination For Case Studies

CONCRETE TYPE 50% SLAG

INPUT PARAMETERS

Dc	1.00E-07	cm^2/s	[From Nomogram]
Cs	5.00	%	[From 7.3.2b]
m	0.68		[From 7.3.2a]
AGE	2	Years	

LONG-TERM PREDICTIONS OF CHLORIDE INGRESS

AGE 100 Years [Service Life]

METHOD 1: CORRECTED MODIFIED SOLUTION OF FICK'S LAW

Di	2.01E-02	cm^2/s	$[\mathrm{Log}\,D_i = \mathrm{Log}\,D_c + m\,\mathrm{Log}\,t]$
Dii	6.44E-03	cm^2/s	[Dii = (1-m)Di]

USING EQUATION

$$C_x = C_s\left(1 - \mathrm{erf}\left[\frac{x}{2\sqrt{D_i t^{(1-m)}}}\right]\right)$$

DEPTH (mm)	0	20	40	60	80	100
CHLORIDE Cx (%)	5.00	3.68	2.63	1.81	1.19	0.74

METHOD 2: STANDARD SOLUTION OF FICK'S LAW

$(D_c)_{100}$	6.99E-09	cm^2/s	$[\mathrm{Log}(D_c)_{100} = \mathrm{Log}(D_c)_2 - m(\mathrm{Log}\,t_{100} - \mathrm{Log}\,t_2)]$

USING EQUATION

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

DEPTH (mm)	0	20	40	60	80	100
CHLORIDE Cx (%)	5.00	3.68	2.63	1.81	1.19	0.74

Table 7A5: Example of Method for Determining Time to Corrosion Activation

Assuming n pairs of data measurements of the form $(x_1\ y_1)$.......................$(x_n\ y_n)$ with mean values of (x y). A linear regression line may be determined relating the x and y values. The regression line may be described as follows

$$y_0 = a_0 + a_1 x_0$$

where a_0 is the y axis intercept, a_1 is the regression coefficient (slope), x_0 is the selected value of x and y_0 is the predicted value of y.

Confidence intervals represent the limits within which the true regression coefficient will be found for the given data and level of confidence. Confidence intervals therefore define the variability of the regression coefficient and may be determined from

$$\textbf{Confidence intervals} = a_0 + a_1 x_0 \pm t\, s_{x/y} \sqrt{1/n + \frac{(x_0 - x)^2}{\sum_{i=1}^{n} (x_i - x)^2}}$$

where t is the Student t-distribution value determined from appropriate tables and $s_{x/y}$ is the covariance of x and y values given by a spreadsheet regression analysis.

Prediction intervals represent the limits within which values of y may be predicted from values of x for the given data and level of confidence. Prediction intervals therefore define the spread of data and may be determined from

$$\textbf{Prediction intervals} = a_0 + a_1 x_0 \pm t\, s_{x/y} \sqrt{1 + 1/n + \frac{(x_0 - x)^2}{\sum_{i=1}^{n} (x_i - x)^2}}$$

From the different definitions it is clear that confidence and prediction intervals have different properties and should not be confused. Confidence intervals are used when the variability of the regression coefficient is being considered while prediction intervals are used when the spread of data around the regression line is being determined.

Appendix 7A6: Prediction and Confidence Intervals for Linear Regressions[28]

CHAPTER EIGHT

CONCLUSIONS

8.1 Potential Durability of Concrete

Near-surface transport properties of concrete which protect the reinforcement from external agencies causing deterioration may be used to define the potential durability of the material. A number of durability index tests which characterize the cover concrete with respect to transport mechanisms such as diffusion, permeation and absortion were investigated. The reliability of these tests as durability indicators was assessed by comparing concrete characterization results with the durability performance of concrete in the marine environment as measured by chloride ingress. The chloride conductivity test when modified to allow for long-term effects was found to be a reliable indicator of chloride resistance and the technique shows promise for determining the potential durability of reinforced concrete exposed to marine conditions.

Durability index tests should be used before exposure to the marine environment as the reliability of the tests diminishes with exposure due to interactions between the material and ingressing agents. A comparison of characterization results indicated that the use of cement extenders, increasing length of moist curing and increasing grade of concrete improved the potential durability of concrete. Traditional methods of specifying for durability using criteria such as strength and cement content do not necessarily produce satisfactory results and current specifications were found to be inadequate. Allowance needs to be made in durability specifications for the different chemical as well as physical characteristics of concrete materials.

Early-age tests of concrete durability should not be used indiscriminantly, particularly when assessing the comparative performance of different concrete types. Short-term comparisons of concrete can only be made once the relationship between early-age tests and long-term performance has been established. Similarly, accelerated tests for measuring potential durability need to be used with caution and should preferably be used to complement other durability studies. Short-term testing of concrete must be done with an understanding of the complexity of physical and chemical interactions involved and the limitations of the test techniques. It is essential that field testing of concrete specimens and case studies of concrete structures should not be neglected since this data is required to validate other durability studies.

8.2 Concrete Durability Performance

Observations from field exposure samples and marine structures showed that the durability performance of concrete was dependent on the material properties and severity of the marine environment. Field exposure samples had durability characteristics consistent with indications from early age characterization tests measured at 28 days. Cement extenders such as fly ash and slag had significantly lower rates of chloride ingress than similar OPC concrete while the effects of curing and concrete grade were of lesser importance. A reasonable correlation was found between modified chloride conductivity results and diffusion coefficients measured after two years marine exposure.

Concrete performance in the marine environment was significantly affected by the severity of exposure, particularly with regard to the rate of chloride penetration. Concrete in the tidal and splash zone was generally exposed to more severe exposure than spray zone concrete resulting in higher rates of chloride ingress. Chloride levels were significantly higher in concrete exposed to heavy wave action than similar concrete subjected to mild seawater contact. The marine spray zone had extremely variable conditions of exposure and insufficient guidance is currently given to designers to make a rational assessment of site exposure conditions for durability considerations. A more systematic approach of rating the severity of exposure was proposed for the marine spray zone. This marine exposure rating system should have broad application for South African conditions and provides better guidance to designers than current methods.

Current durability specifications were found to be inadequate for extreme and very severe exposure conditions and OPC concrete generally has insufficient chloride resistance to adequately protect reinforcement under such harsh exposure conditions. The current trend appears to be favouring higher grade OPC concrete despite evidence suggesting that these concretes are not sufficiently durable. Designers should rather consider using cement extender materials such as fly ash and slag to ensure adequate chloride resistance. New specifications need to be formulated which allow for the differing performance of concretes under marine conditions. This will ensure that material selection may be optimized in terms of long-term performance rather than short-term financial considerations only.

8.3 Predictions of Durability

The proposed prediction model was found to be reliable when validated independently on marine concrete structures. The model makes allowance for different materials, construction effects and severity of exposure. Design recommendations based on the model should help improve the durability of marine structures by increasing the accuracy of service life estimates but should be used within the limitations of the model. The basic premise inherent in the model is that the rate of chloride penetration through concrete determines the rate and amount of reinforcement corrosion. This assumption is supported by observations from marine concrete structures which show that there is a definite link between chloride ingress and corrosion damage. The prediction model is based on conditions prevalent along the Cape coast of South Africa and further validation and modification may be needed for broader application in other environments.

Reliable predictions of durability are a necessary precursor to implementing a system of performance-based durability specifications to ensure satisfactory serviceability of marine concrete structures. More data and further validation is required before the approach can be considered for use in durability specifications. The use of performance-based specifications is recommended as this is a more rational approach than presciptive specifications and performance specifications can be used to measure the quality of concrete during construction. Implementation of new specifications needs to be done incrementally to ensure suitable procedures for all parties and to prevent the approach being discredited during preliminary application. This process can only be started when researchers and specifiers agree on appropriate tests for use as durability specifications and develop standardized testing techniques.

8.4 Future Research Requirements

An empirical method for predicting the durability of reinforced concrete in the marine environment was formulated but some findings from this research were not conclusive and further investigation should be considered to establish the following:

a) More data is required from new and older marine structures to assess the durability performance of concrete under different exposure and service conditions. Particular attention should be given to characterize suitable new structures for later testing of durability performance.

b) Fundamental information about reinforcement corrosion is required to determine factors such as chloride activation levels for corrosion and corrosion rates in different concretes.

c) The severity of exposure needs to determined more scientifically particularly with regard to the marine spray zone.

d) Continued field exposure testing of concrete is essential to determine long-term characteristics of concrete and to assess new trends and materials in the construction industry. Testing should be done after three, five and ten years exposure in the marine environment to determine long-term trends. New materials which may benefit marine concrete need to be identified early so that the necessary field testing can be done before widespread application is considered.

e) An equation with a sound theoretical basis describing chloride penetration into concrete needs to be formulated. The assumptions made in using the solution of Fick's Law have been shown to be invalid in some circumstances and the parameters used to define the rate of chloride ingress (surface concentration, diffusion coefficient and diffusion coefficient reduction factor) introduce three sources of error into the calculation.

f) More fundamental information is required about the interaction of chlorides with the constituents of concrete. The effect of chloride binding on the concrete pore structure and diffusivity of the material is of particular interest when considering chloride ingress into concrete.

g) Repair techniques such as electrochemical chloride extraction require further study particularly with regard to possible negative side-effects. The long-term durability performance of concrete repaired with these techniques needs to be investigated.

8.5 Limitations of This Research

The complexity and variability of marine concrete deterioration precluded a definitive solution to the problem and recommendations were made to increase the service life of reinforced concrete structures in the marine environment. The reliability of the prediction model was shown to be poor when considering the statistical variability of the input data but independent validation showed a good correlation between predicted and actual chloride levels. Service life predictions of reinforced concrete are affected by a large number of variables and precise predictions are unlikely but general trends may be determined. The approach should be seen as an interim method which can be built on and improved as more data becomes available and a better understanding of the processes involved is gained.

Considerable improvements can be made by optimizing concrete materials and design but other factors such as construction and economic considerations may negate these material advantages. Durability of concrete structures is likely to remain a vexing issue for some time not merely due to the complexity of the problem but due to considerations outside the ambit of concrete technologists and academics. Hansson best summed up the existing situation as follows: 'until some system is devised to take into account the lifetime costs of a structure, and to educate the taxpayer of the importance of the infrastructure, there will be no incentive for improving long-term quality of concrete or providing for quality rehabilitation of the existing structures' [1].

The issue of concrete durability receives continued research attention but is often ignored by practising engineers in South Africa. It would appear that the problem will only be fully appreciated when the legacy of deteriorated structures is unavoidable, as happened in the Middle East. The lack of foresight in not providing adequate durability at construction will have to be paid for eventually and the humorous warning in Robert Burns' epigram on rough roads still rings true today.

"I am now arrived - thanks to the gods
Through pathways rough and muddy
A certain sign that making roads
Is not this people's study

Although I'm not with Scripture crammed
I'm sure the Bible says
That heedless sinners shall be damned
Unless they mend their ways"

8.6 References

1. Hansson, C.M., 'Concrete: The advanced industrial material of the 21st Century', Metallurgical and Materials Transactions, 26A, June 1995, pp 1321-1341.