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Predictions of reinforced concrete durability in the marine environment

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Abstract

The marine environment provides a severe test of the durability of reinforced concrete. Predictions of durability are difficult to make given the complexity of deleterious physical and chemical interactions between seawater, materials and structure. In this monograph, the basic premise is that chloride-induced corrosion of reinforcement is the major form of deterioration affecting reinforced concrete structures in the marine environment. An empirical model to predict chloride ingress into concrete is presented. The model was formulated from the relationship between early-age chloride conductivity results and medium-term chloride contents measured in field exposure samples. Validation of the accuracy of the chloride prediction model was made using long-term chloride penetration data from marine concrete structures. The prediction model allows early-age properties of concrete to be related to the potential durability performance of the material under a range of marine conditions. This monograph shows that fly ash and slag concretes have significantly better chloride resistance than portland cement concretes.

This first revision of the monograph was undertaken to provide further data generated over the last four years. The basic approach remains the same but extensive new research data has allowed some refinement and a broader application. In particular, the performance of silica fume concrete is included in the prediction model for the first time. Preferred concrete mixes for marine applications are also given for practical guidance.

INTRODUCTION

The increasing number of concrete structures exhibiting unacceptable levels of deterioration, particularly in the marine environment, has attracted widespread attention in recent years. Information from research into concrete durability has increased the fundamental understanding of concrete as a material and the complex interactions between material, environment and structure that cause deterioration. Although durability specifications have become progressively more stringent in response to a perceived lack of durability of reinforced concrete, modern structures have not always shown a corresponding improvement in durability. This appears to be due to a lack of understanding of what is required to ensure durability and inadequate means of enforcing/guaranteeing compliance with specifications during construction.

Reinforced concrete structures in the marine environment are most susceptible to chloride-induced corrosion of reinforcement due to the presence of high chloride concentrations and humid or saturated conditions. Corrosion is manifested in two primary forms: cracking and spalling of cover concrete due to the formation of expansive corrosion products at the reinforcement, and local pitting at the anode which reduces the cross-sectional area of the bar. Many marine structures in South Africa exhibit severe corrosion damage. A survey of concrete structures along the Western Cape coast revealed that many structures will require major repairs to achieve their original design lives.[1] Existing durability specifications appear to be partly to blame, being ineffective or misleading. More reliable methods of predicting the risk of reinforcement corrosion need to be formulated so that appropriate durability specifications can be developed.

This monograph seeks to address these needs by providing a design approach that should assist designers of marine concrete structures in South Africa achieve greater durability. The process of reinforcement corrosion is described and important factors affecting durability are listed and examined. A framework for designing reinforced concrete structures for durability is given and methods of compliance testing of site concrete are proposed. The appendices contain additional information generated from laboratory, field exposure and long-term case studies.

CORROSION OF REINFORCEMENT

Corrosion of steel in concrete is a complex phenomenon, influenced by many internal and external factors. These include the pH of the concrete pore solution, temperature, internal stresses, stray currents and electrolytic potentials. Reinforcement cast into concrete is initially rendered passive by a protective layer of ferric oxide that forms under normal alkaline conditions (pH 12.5-13.5). The ferric oxide layer may be disrupted by the presence of sufficient chloride ions or by a reduction in pH of concrete below 10.5 due to carbonation or acidification. Once depassivation has occurred, corrosion of steel is possible provided oxygen and moisture are available as shown schematically in Figure 1.

Chloride ions act as catalysts in the disruption of the passive ferric oxide layer and are recycled for use elsewhere along the steel surface. A minimum concentration of chlorides at the steel, known as the corrosion threshold level, is required to depassivate reinforcement under normal alkaline conditions. Once the steel is effectively depassivated, the corrosion rate and subsequent damage will depend on micro-effects such as availability of oxygen and moisture and macro-effects such as structural geometry, anode / cathode ratios and general ambient conditions.

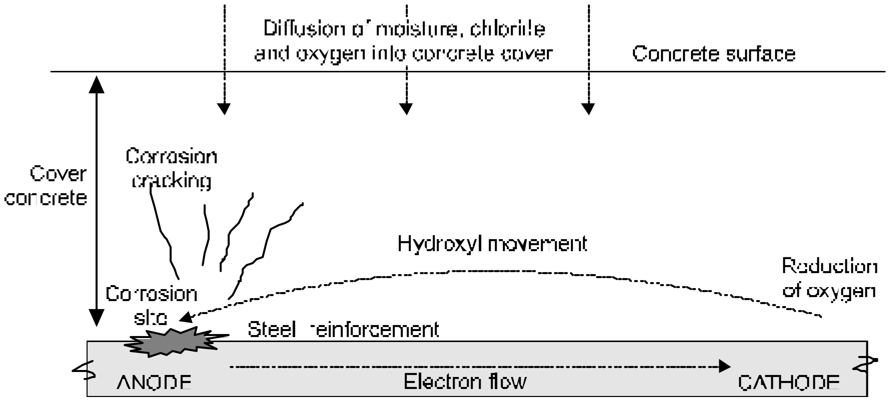

Figure 1: Schematic diagram of corrosion of reinforcement in concrete

Corrosion of reinforcement in concrete structures may be divided into two separate phases; an initiation period during which little damage occurs as chlorides diffuse towards the reinforcement, and a propagation phase in which damage occurs progressively once the corrosion threshold had been exceeded and corrosion has been initiated. The insidious nature of steel corrosion makes effective repairs expensive if remedial work is only contemplated once significant damage occurs. Repair principles for corrosion-damaged structures are dealt with in Monograph No. 5.

FACTORS AFFECTING MARINE CONCRETE DURABILITY

Deterioration of reinforced concrete structures in marine environments is generally associated with external agents such as chlorides that penetrate into concrete causing damage. Using the premise that the potential durability of reinforced concrete is determined by the protection provided by the cover concrete, a number of factors affecting durability may be defined. These include concrete type, cover depth to reinforcement, site practice and severity of exposure.

1. Concrete type

The type of concrete used to protect reinforcement has a major influence on durability since the material controls the rate at which aggressive agents move through the cover concrete. Current codes of practice make allowance for the improved chloride resistance of higher grade concrete but largely ignore the influence of binder type. This approach recognizes that physical properties of concrete control transport properties of the material, while largely ignoring chemical effects. Chloride ingress into concrete is not only determined by the permeability of the pore system but also by interactions between the material and the diffusant that depletes the concentration and constricts the pore structure. Concrete containing fly ash and slag have been shown to have exceptional chloride binding characteristics and produce material of high chloride resistance.[2]

2. Cover to reinforcement

The potential durability of reinforced concrete is greatly enhanced if adequate cover to reinforcement is specified and achieved on site. For sufficient protection of reinforcement under marine conditions, covers should be in the region of 50 to 75 mm. Reduced cover is risky even when using high quality concrete since defects such as cracks and voids may provide a low resistance path to the reinforcement. Increasing cover to depths beyond 75 mm may however result in excessive surface crack widths and is often not practically possible.

3. Site practice

Poor site practice, particularly with regard to placing, compaction and curing of concrete may negate the benefits of good design and material selection. Research has established the value of good site practice such as active moist curing in improving the near-surface properties of concrete.[3] Specifications have been proposed to control these site activities, but unfortunately adequate supervision and suitable methods to monitor compliance have not been implemented on site. The inability to ensure consistent quality of concrete on site is a major reason for the continued prevalence of concrete durability problems.

4. Severity of exposure

The severity of marine exposure varies considerably depending on factors such as climate, location relative to the sea and structural considerations. Current codes of practice provide limited guidance about exposure conditions and generally define only two marine categories: extreme exposure for concrete subjected to full abrasive action of the sea, and very severe exposure for concrete subjected to spray or mild abrasive / wave action. The wide variations of exposure in the marine spray zone are not adequately defined by these categories. This is particularly problematic since most marine concrete structures are located in the spray zone. Given the range of marine conditions, a more comprehensive and rationally-structured system for defining the severity of exposure needs to be formulated.

DESIGNING FOR DURABILITY

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

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

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

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

1. Marine exposure categories

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

Table 1: Classification of marine exposure categories

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

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

2. Service requirements

Factors relating to the service function and structural requirements need to be defined before detailed design can commence. Service functions include the service life required for the structure, likely future maintenance and logistics of repairs. Structural considerations include minimum concrete grade, cover depth and crack width limitations, dimensional stability (i.e. shrinkage and creep potential) and long-term risk of cracking.

3. Selection of concrete materials

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

(a) Laboratory-based investigations

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

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

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

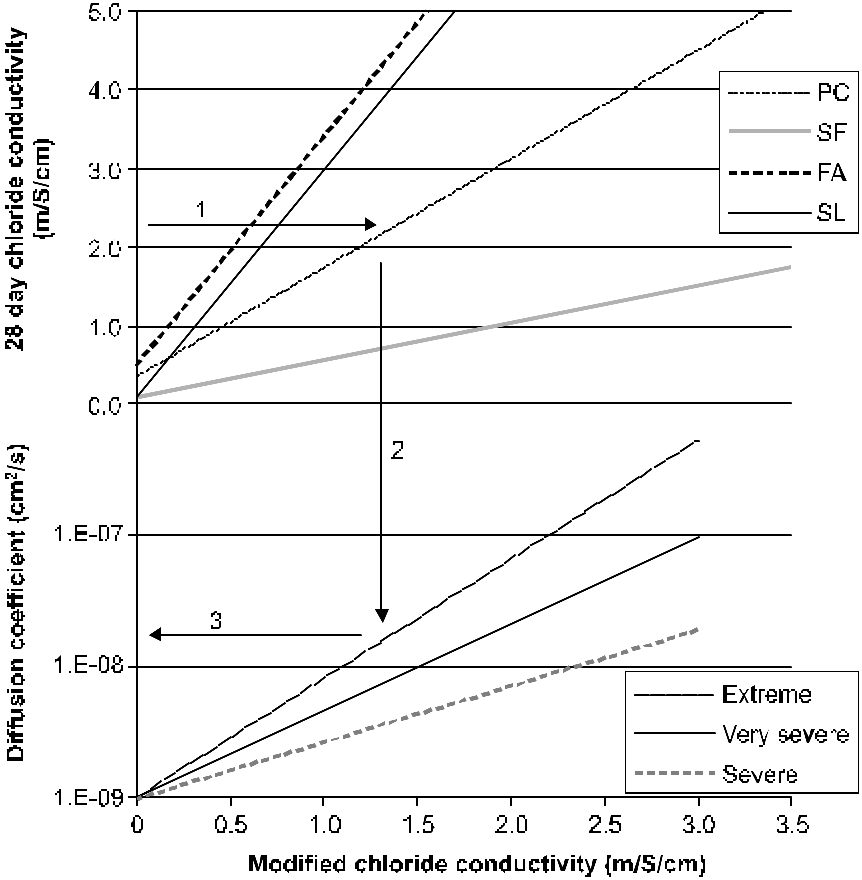

Figure 2: Predicting 50-year diffusion coefficients for marine concrete

(b) Desk-top study

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

Table 2: Predicted diffusion coefficients ($\times 10^{-8}$ cm^2/s)

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

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

4. Prediction of chloride levels in concrete

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

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

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

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

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

5. Estimation of time to corrosion activation

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

VALIDATION OF PREDICTION MODEL

Validation of the prediction model was done using chloride content results from several marine concrete structures along the Cape coast of South Africa (Appendix 3). Initially the validation process was not truly independent since case study data was used in the formulation of the prediction model. Since 1996 several new structures have been investigated and the results used to validate the model. Correlations between actual and predicted chloride levels from marine structures in the Western Cape were found to be good. Predictions for marine structures further afield (i.e. West Africa and Kwazulu Natal) were found to be less accurate, but still reasonable correlations were achieved.

It is accepted however that predictions of durability involve consid-

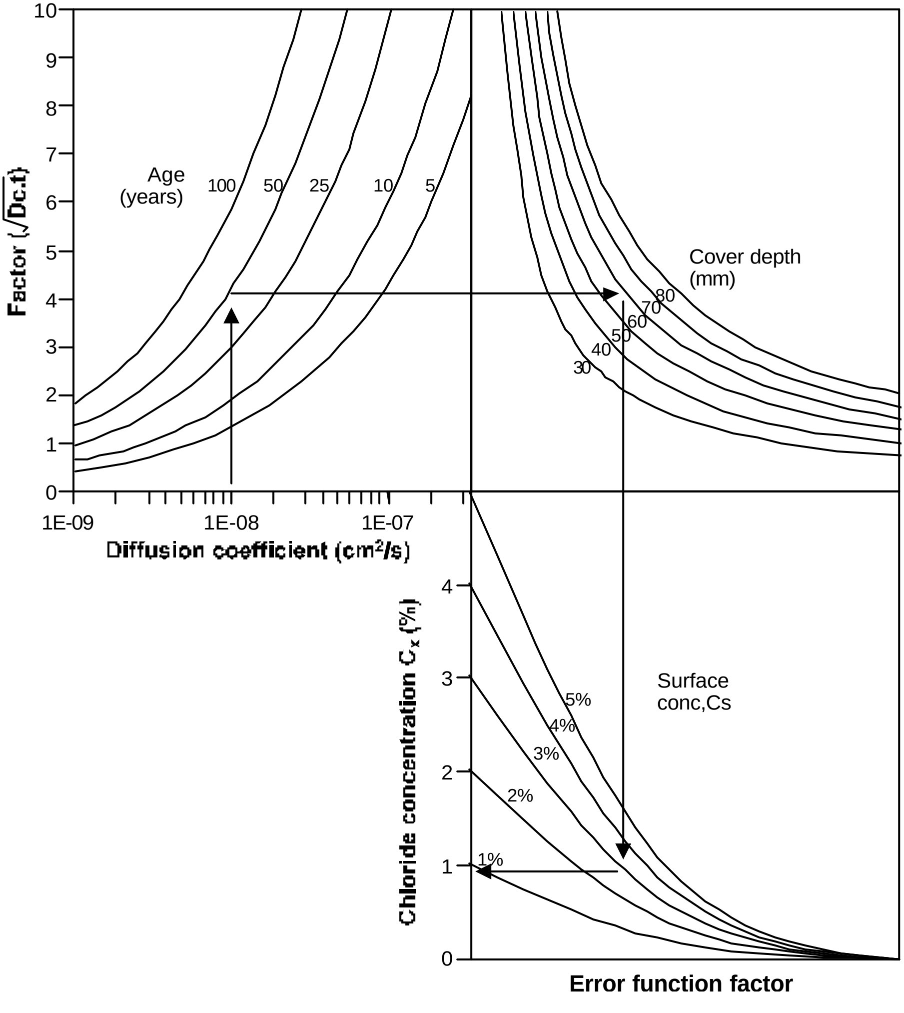

Figure 3: Nomogram showing a graphical solution of Fick's law of diffusion

erable variability and uncertainty and prediction model results should be considered critically.

COMPLIANCE TESTING OF SITE CONCRETE

Good design and material selection may be compromised by poor construction practices. Controls need to be established and implemented to ensure satisfactory execution of designs on site. Little progress in this

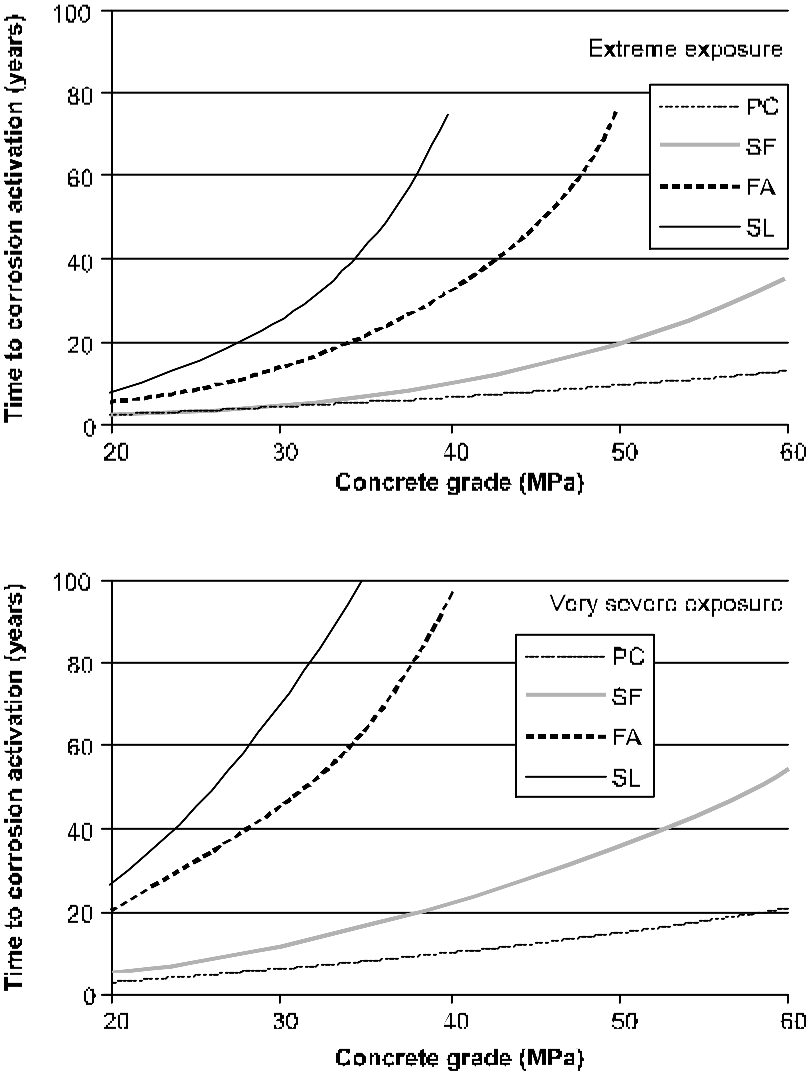

Figure 4: Time to corrosion activation for different concretes

area is likely until owners of structures are made aware of the implications of poor construction to their investments and parties are held accountable for the long-term integrity of their work.

Probably the greatest area for improvement of concrete durability is that of ensuring adequate cover to reinforcement. On marine structures where cover control is critical, it is essential that some form of cover survey is undertaken after construction. Areas of low cover can be quickly identified and appropriate protection provided to the reinforcement.

Durability index tests that are sensitive to changes in concrete pore structure that affect durability may be used to assess the quality of site concrete. These techniques include oxygen permeability, water sorptivity and chloride conductivity and are fully documented in Monographs 2, 3 and 4. Durability index testing should ideally be done at early ages (preferably 28 days) before significant deterioration has occurred.

PREFERRED MIXES

The choice of concrete materials used in the marine environment will be largely dictated by economic, logistic and technical factors. Preferred concrete mixes for reinforced concrete structures exposed directly to wave action (i.e. extreme exposure) in the Western Cape are given below. It is assumed that grade 40 concrete is sufficient for structural purposes and a standard service life of 50 years is required.

(a) PC concrete

Portland cement concretes have poor chloride resistance and should only be used with high cover depths to reinforcement. A cover depth of 75 mm was therefore selected whilst a w/c ratio of 0.39 was required. Chloride conductivity values of 1.6 mS/cm at 28 days should be achievable with this concrete. Due to the high cement content, granite coarse aggregate was selected together with a high quality pit sand.

Cement (CEM I)	460 kg/m^3	Binder content – 460 kg/m^3
Stone	1100 kg/m^3	
Sand	700 kg/m^3	
Water	180 L/m^3	

(b) CSF concrete

Replacement of cement with 9% condensed silica fume will produce higher strength material with a refined pore structure. The resulting improved chloride resistance of CSF concrete allows reasonable cover depths to be specified, in this case 60 mm. The required w/b ratio was 0.47 to achieve a 28-day chloride conductivity limit of 0.5 mS/cm. Aggregates selected were greywacke stone and blended pit and dune sand.

Cement (CEM I)	370 kg/m^3	Binder content – 405 kg/m^3
CSF (9%)	35 kg/m^3	
Stone	1100 kg/m^3	
Sand	715 kg/m^3	
Water	190 L/m^3	

(c) Fly ash concrete

Only moderate cover to reinforcement of 50 mm is required for fly ash concrete due to the high chloride resistance of the material. Effective curing is however essential to produce dense near-surface properties in the concrete. The required w/b ratio for the concrete is 0.45 in order to achieve a 28-day chloride conductivity of 1.5 mS/cm. Aggregates selected were greywacke stone and blended pit and dune sand.

Cement (CEM I)	265 kg/m^3	Binder content – 380 kg/m^3
Fly ash (30%)	115 kg/m^3	
Stone	1150 kg/m^3	
Sand	685 kg/m^3	
Water	170 L/m^3	

(d) Slag concrete

Slag concrete has good chloride resistance and 50 mm cover to reinforcement should be sufficient to protect the reinforcement. Special care with regard to curing is required to achieve a dense and durable concrete surface. In order to achieve a chloride conductivity value of less than 0.8 mS/cm at 28 days, a w/b ratio of 0.50 was selected. Aggregates selected were greywacke stone and blended pit and dune sand.

Cement (CEM I)	175 kg/m^3	Binder content – 350 kg/m^3
Slag (50%)	175 kg/m^3	
Stone	1150 kg/m^3	
Sand	715 kg/m^3	
Water	175 L/m^3	

CONCLUSIONS

The durability performance of reinforced concrete structures in the marine environment is generally unsatisfactory and needs to be improved. Current approaches of specifying concrete durability are not always satisfactory and may be misleading. The continued reliance on concrete strength as an indicator of potential durability is irrational since other factors such as binder type and construction practice have a greater influence on durability performance. A new approach is required to ensure the durability of marine concrete structures; one that is able to quantify the resistance of concrete using specifications that can be easily implemented on site.

A design technique is proposed for predicting chloride levels in marine concrete using a modified solution of Fick's law of diffusion. The prediction model is based on the relationship between early-age concrete characterization testing and medium-term diffusion coefficients of concrete, validated with long-term chloride contents from marine concrete structures. The technique allows long-term characteristics of concrete to be assessed rather than early-age properties that may have little bearing on durability. The durability of marine concrete structures should be improved by the use of sound design techniques and ensuring the requisite concrete quality is achieved on site. The proposed approach should go some way towards achieving these objectives.

APPENDIX I
Concrete characterization testing

Using the premise that the potential durability of concrete is determined by the protection provided by the cover concrete to the reinforcement, the resistance of this layer may be defined in terms of transport properties such as absorption, diffusion and permeation.[9] Measuring transport properties of concrete intrinsically is difficult to achieve due to lengthy test periods and the need for sophisticated equipment. A suite of durability index tests has therefore been developed that produces reliable results using relatively simple and inexpensive test methods.

Durability index tests may be defined as techniques that measure the early-age resistance of concrete to transport of fluids and ions, that affect deterioration, through the cover concrete.[10] Durability index tests are able to characterize the material properties of concrete at 28 days. These properties can be used for comparative purposes or related to long-term durability performance (see Monograph 2 for further details).

The potential chloride resistance of concrete may be assessed using the chloride conductivity test. The technique has the advantage of being extremely rapid and simple to perform whilst conductivity values determined have a sound theoretical basis. The chloride conductivity apparatus is shown in Figure A1.

The chloride conductivity apparatus consists of a two-cell conduction rig in which concrete core samples (68 mm diameter and 25 mm thick) are exposed on either face to 5M NaCl solution. The core samples are initially preconditioned at 28 days in an attempt to standardize the pore water solution (oven-dried at 50 ^{0}C and 15% R.H. for seven days followed by 24 hours vacuum saturation in 5M NaCl solution). The movement of chlorides is accelerated by applying a 10V potential difference across the concrete sample, and the chloride conductivity determined by measuring the current flowing through the concrete.

The chloride conductivity of concrete may be defined as follows:

$$? = \frac{i\,.\,t}{VA} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where ? is the chloride conductivity, ***i*** is the current, ***V*** is the voltage, ***t*** is the sample thickness and ***A*** is the cross-sectional area.

Increased chloride resistance as measured by the chloride conductivity test is generally found for concrete with increasing grade, extent of

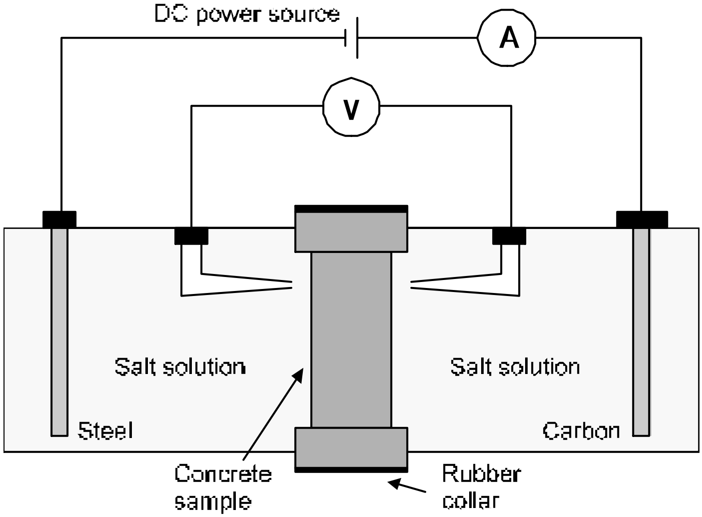

Figure A1: Chloride conductivity apparatus

initial moist curing and replacement of portland cement with silica fume, fly ash and slag. Typical chloride conductivity measured at 28 days for a range of different concretes is shown in Figure A2. All concrete was fully wet cured for 28 days and the following notation applies:- PC denotes 100% portland cement, SF denotes 10% condensed silica fume, FA denotes 30% fly ash and SL denotes 50% slag.

Increasing the grade of concrete improves the durability potential although portland cement concretes have limited improvement. Concrete exposed to highly aggressive marine conditions should therefore contain cement extenders such as fly ash, silica fume and slag. If concrete is poorly cured, the use of cement extenders may however produce poorer quality cover concrete.

Corex slag has recently been launched in the Western Cape to replace blast-furnace slag from Gauteng. Concrete containing Corex slag has been found to exhibit excellent hardened properties and durability potential. Preliminary laboratory tests indicate that the material should have similar chloride resistance to that of concrete made with blast-furnace slag.

Early-age testing of concrete tends to be comparative in nature and may be misleading when comparing different concrete types without

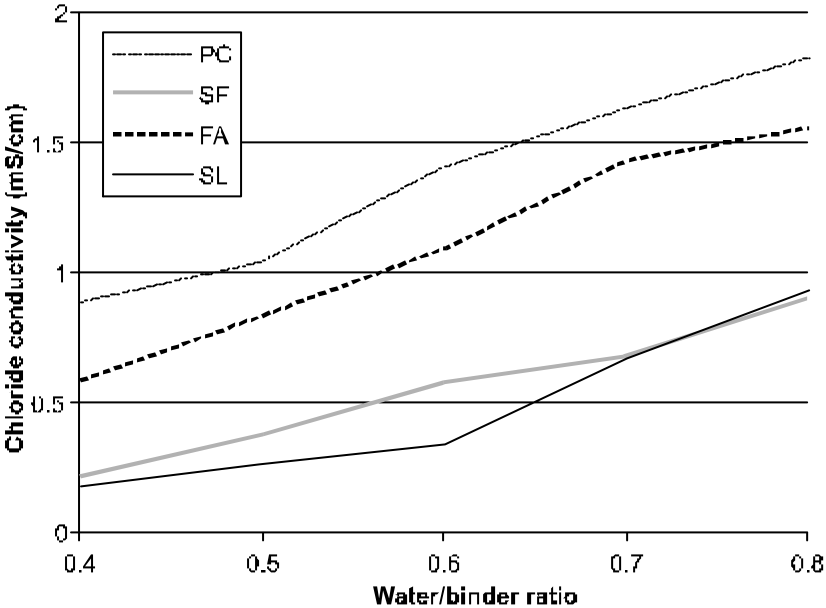

Figure A2: Typical chloride conductivity results (wet cured concrete)

considering long-term factors. Characterization testing at 28 days will naturally favour cementitious materials that mature rapidly, such as silica fume concrete, in comparison to slower reacting materials such as fly ash concrete. Some allowance must therefore be made for this phenomenon as was done in the chloride prediction model.

APPENDIX 2
Marine exposure testing

The complex chemical and physical interactions between seawater and the constituents of concrete make laboratory simulations of marine exposure 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; this is not possible with accelerated laboratory tests. The disadvantage of field exposure testing is that deterioration may take many years to proceed to a measurable extent and extrapolations may therefore be necessary to estimate long-term trends. These extrapolations may be misleading if the results are not independently validated with long-term data.

Six separate marine exposure projects have been undertaken around the Cape Peninsula over the last eight years. Concrete blocks were exposed to the marine environment at 28 days and monitored at six or twelve month intervals. The most important indicator of durability was deemed to be chloride ingress into the concrete. Drilled powder and core samples were extracted from blocks for chloride analysis. Chloride contents with depth were analysed in accordance with BS 1881 Part 124 but using a potentiometric titration.[11] Diffusion coefficients and surface concentrations were determined from chloride profiles obtained using the standard solution of Fick's Law (equation 1) and a curve fitting program.

Typical chloride profiles determined from Simonstown tidal zone exposure are shown in Figure A3. Concrete shown had a water/binder ratio of 0.66 and was exposed to very severe marine conditions for two years (mild wave action and sea temperatures ranging from 13-20^{0} C). PC denotes 100% portland cement concrete, SL denotes 50% blast-furnace slag concrete and SF denotes 10% condensed silica fume.

In all marine exposure trials, concrete containing blast-furnace slag was found to have good chloride resistance. Preliminary marine exposure findings suggest that concrete made with Corex slag has similar chloride resistance to that achieved with blast-furnace slag concrete.

Chloride ingress into concrete exposed to the marine environment was found to be dependent on the severity of exposure. Concrete exposed to heavy wave action at Granger Bay had significantly higher chloride contents than concrete exposed at Simonstown despite cooler

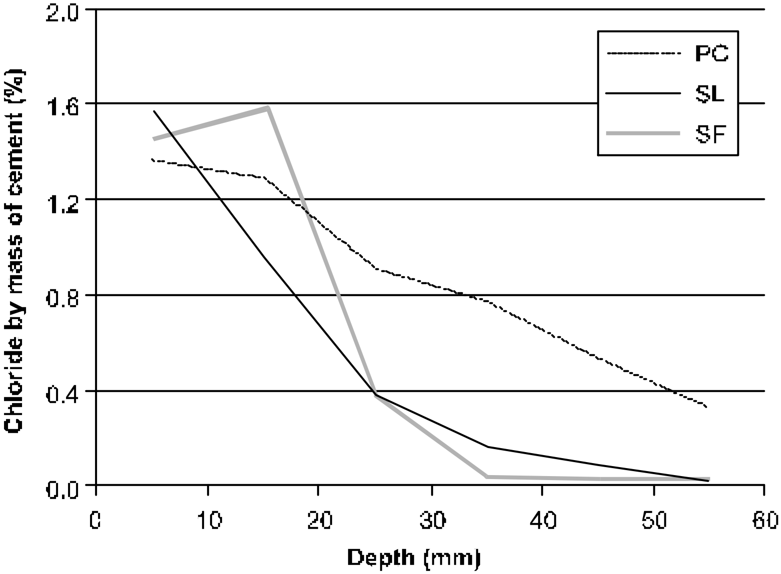

Figure A3: Chloride profiles after two years exposure at Simonstown tidal zone

sea temperatures (i.e. 12-15 ^{0}C). Figure A4 shows chloride profiles measured after two years for grade 40 PC concrete (SMT and SMS denotes Simonstown tidal and spray zones while GBT and GBS denotes Granger Bay tidal and spray zones respectively).

Marine exposure data was also analysed to assess longer-term trends with regard to chloride penetration into concrete. The reduction of diffusion coefficients with time was particularly significant and formed the basis for the empirical chloride prediction model. Diffusion coefficients determined for portland cement concrete were found to reduce only moderately between one and eight years whereas values measured for fly ash and slag concrete reduced by more than one order of magnitude. In contrast, all concrete types were found to have increasing surface concentrations with time.

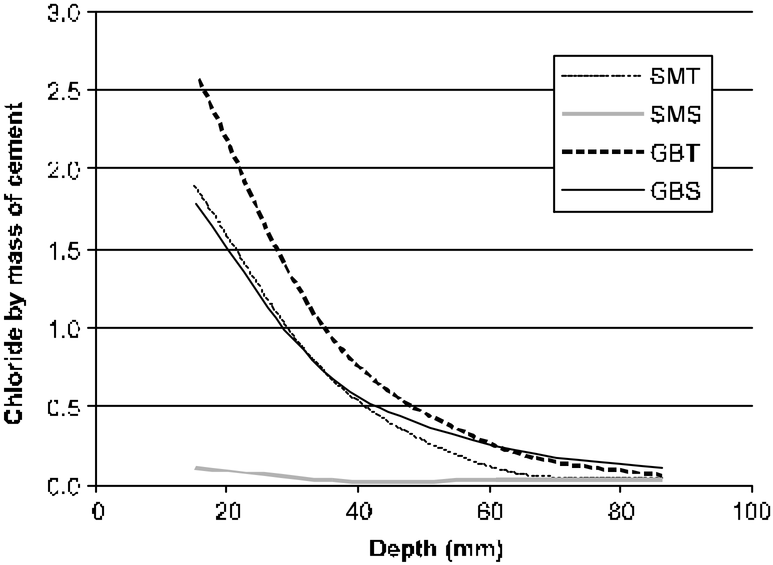

Figure A4: Chloride profiles for grade 40 PC concrete – two years exposure

APPENDIX 3
Case studies of reinforced concrete structures

Durability studies of concrete structures must ultimately relate to the behaviour of structures in service, yet most research does not directly consider site conditions. There are several reasons for favouring a more analytical laboratory approach: site concrete is exposed to a multitude of influences that are difficult to quantify, design and construction data is often unreliable, and assessing the durability performance of a structure is relatively subjective. Despite these practical limitations, useful information may be obtained about the long-term behaviour of concrete under real service conditions. This information is essential for confirming trends established from early-age laboratory and field exposure studies. Case studies of concrete structures should therefore be regarded as the ultimate benchmark for concrete durability studies.

Over the past eight years more than twenty marine concrete structures have been investigated along the South African coast (shown in Table A1).[12] Direct comparisons between structures are complicated by different materials, exposure conditions and service functions. Chloride penetration into concrete was found to be dependent on the type of concrete and the severity of exposure, but was less affected by the grade of the concrete. Fly ash and slag concretes in particular were found to have improved chloride resistance compared with PC concrete.

Chloride levels were found to increase with age and most structures older than 30 years exhibited severe corrosion damage. Virtually all structures older than 25 years were in poor condition and in need of repairs. Of greater concern was the number of structures less than ten years old that exhibited premature deterioration. Contributing factors to this early damage include bad designs, inadequate specifications and poor construction practice.

Table A1: Details of marine structures investigated

Age (years)	*Structure name*	*Present condition*
1-5	Hermanus abalone tanks	Poor – fair
3	Simonstown precast masts	Fair
3	Kogel Bay tidal pool	Good
6	Hout Bay marina	Repaired
6	Strand hotel	Repaired
6	Table Bay breakwater	Fair
7	Waterfront aquarium	Fair
13	Strandfontein tidal pool	Good
18	Camps Bay pumpstation	Good
19	Oudekraal retaining wall	Poor
20	Koeberg power station	Fair
25	Humewood bridge	Poor
28	East London breakwater	Fair
30	Swakopmond bridge	Repaired
35	Baden-Powell Road bridges	Poor
38	Muizenberg bridges	Repaired
50	Wilderness bridge	Repaired
55	Sea Point aquarium	Repaired
65	Steenbras River bridge	Repaired
70	Table Mountain cable stations	Repaired
75	Simonstown Jetty	Repaired

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