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PROBABILITY-BASED DURABILITY ANALYSIS OF CONCRETE STRUCTURES IN MARINE ENVIRONMENT

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To my family and Riitta

ABSTRACT

A majority of concrete structures in marine environment shows signs of degradation due to the corrosion of the reinforcement in the presence of chlorides. In some cases, the degradation is visible within a few years of construction completion. Despite the fact that the marine environment is particularly sever, other factors affect the premature degradation due to corrosion, such as: poor construction quality as a result of poor workmanship; inadequate standards based on prescriptive measures; and poor design as a result of insufficient information with regards to the parameters that influence the degradation process.

While standards are gradually improving, with the tendency to become performance based in future, much information is needed to assist the designer in the decision making process. This research work is aimed at providing designers with adequate means for probability - based durability analysis of concrete structures in marine environment.

A durability model for the reinforcement corrosion, based on the second Law of Fick, is developed that takes into account the time dependant parameter for diffusion coefficient and the influence of temperature. The model has been developed primarily for providing a general basis for durability analysis of concrete structures in marine environment. Such a durability analysis can be applied for obtaining a more controlled durability and the long-term performance durability design of new concrete structures, as well as, an improved basis for condition assessment of existing concrete structures in marine environment. The model has been developed to satisfy a balanced compromise of practicality and robustness. The existing models are either difficult to use due to the complexity of calculations, and the fact that they require sophisticated tests for collecting the necessary data, or are too simplistic to provide reliable results. The proposed model requires few basic data, either available or easily obtainable, without compromising its scientific robustness and validity.

As an extension of the model a probability - based software is proposed for the durability analysis. The parameters of the model have been discussed and the sensitivity of the model towards these parameters is analysed. Results show that the proposed software provides valuable information indicating the effect of each parameter on the outcome and the extent in which these parameters should be stochastically defined.

Further, the proposed model was validated comparing the results from the assessments of the structures with the outcome of the probabilistic simulation of the service life of the structure.

The effect small changes in parameters have on the overall performance can be rapidly evaluated. Through the use of the proposed model, the designer can appreciate the consequences of design options on the concrete and the structure.

The importance of correct temperature use in the model is demonstrated with two examples, one at 10 ºC and another at 20 ºC. An increase (more than double) in the time to reach the serviceability limit was observed.

The results from this investigation demonstrate that the analysis of the durability performance of the structures can be expanded and improved by using the proposed model, in relation to the traditional condition assessment.

RESUMO

A maior parte das estruturas de betão armado em ambiente marítimo demonstram sinais de degradação resultante da corrosão das armaduras na presença de cloretos. Nalguns casos, a degradação é visível poucos anos após a construção da estrutura. Se bem que o ambiente marítimo é extremamente severo, existem outros factores que afectam a degradação prematura do betão, tais como: a fraca qualidade da construção resultante da falta de mão de obra especializada; normas baseadas em requisitos prescritivos; projectos desadequados devido à falta de informação em relação aos parâmetros que influenciam o processo da degradação.

Enquanto que as normas, de uma forma geral, estão a melhorar, com a tendência a basearem-se cada vez mais em requisitos de desempenho, ainda há necessidade em auxiliar o projectista na especificação da durabilidade da estrutura. Este trabalho de investigação está centrado em fornecer aos projectista meios adequados para uma análise probabilística da durabilidade da estruturas de betão armado em ambiente marítimo.

Um modelo de durabilidade para a corrosão da armadura baseado na segunda Lei de Fick é desenvolvido tendo em conta a dependência do coeficiente de difusão com o tempo e com a temperatura. O modelo foi desenvolvido essencialmente para servir como um suporte para a análise da durabilidade de estruturas de betão armado em ambiente marítimo. Uma análise desta natureza pode ser aplicada tanto ao controlo da durabilidade e do desempenho a longo prazo de novas estruturas de betão armado, como à avaliação do estado de degradação de estruturas existentes em ambiente marítimo.

O modelo proposto é desenvolvido por forma a conciliar a facilidade de uso com a robustez científica. Modelos existentes ou são demasiado complicados para serem utilizados de uma forma prática, i.e. requerem normalmente ensaios sofisticados para recolher os dados necessário, ou, são excessivamente simplificados para fornecerem resultados fiáveis. O modelo proposto requer poucos dados básicos que, estão facilmente disponíveis, ou poderão ser obtidos com relativa simplicidade, sem contudo comprometer a robustez e validade científica.

Como extensão natural do modelo, é proposto um software baseado na metodologia probabilística para a análise da durabilidade. Os parâmetros do modelo foram estudados, e a sensibilidade do modelo a variação destes é analisada. Os resultados demonstram que o software proposto fornece informações valiosas no que respeite à influência de cada parâmetro na análise da durabilidade, e na forma como estes devem ser quantificados estocasticamente.

O modelo proposto foi validado com resultados obtidos de estruturas existentes. Através das simulações probabilísticas o efeito de alterações dos parâmetros do modelo no desempenho da estrutura pode ser eficazmente avaliado. Recorrendo ao software, o projectista pode avaliar a consequência de opções de projecto no betão e na estrutura.

A importância de utilização de temperaturas prevalecentes no local da obra é demonstrada através de exemplos analisados a 10 ºC e 20 ºC.

Os resultados desta investigação realçam que a análise do desempenho de durabilidade das estruturas pode ser expandida e melhorada com recurso ao modelo proposto quando comparado com a avaliação da durabilidade tradicional.

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European Union
European Social Fund

CONTENTS IN BRIEF

CONTENTS

FIGURE INDEX

TABLE INDEX

NOTATIONS AND UNITS

a_1, a_2, a_3	Regression parameters
A_{CR}	Critical air content
$A_{Cs,cl}$	Regression parameter
b	Binder
B	Performance function
b^d	Design value of a parameter depending on the position of the bar
c	Cement content of the concrete mix
c	Chloride concentration in solution, kg/m³
C_0	Initial chloride content in (wt.% Cl^-/binder)
CA	Cumulative amount of dissolved calcium. [mol/(m^2s)]
C_{CR}	Critical chloride threshold content (wt.% Cl^-/binder)
$c^d{}_{CR}$	Design value of the critical chloride concentration
$c^d{}_{S,CL}$	Design value of the surface chloride concentration
c_{MIN}	Minimum cover
c_{NOM}	Nominal cover
C_S	Surface chloride content in (wt.% Cl^-/binder)
Cv	Protecting concrete thickness (mm)
d, D	Diameter of reinforcement (mm)
D	Diffusion coefficient, m²/s
D_0	Diffusion coefficient at the time t_0 (m^2/s);
Df	Durability factor
D_{MAX}	Maximum aggregate size
D_{RCM}	Diffusion coefficient for the Rapid Chloride Migration method
D_{ref}	Diffusion coefficient at some reference time t_{ref} and temperature T_{ref}
e_0	Elementary charge
E_A	Activation energy for chloride diffusion in (J/mol)
Ed_0	Dynamic modulus at the start of test
E_{dn}	Dynamic modulus at the end of test
erf^{-1}	Inverse of the error function
F	Faraday constant (96485 C/mol)
$f^d{}_{C,SP}$	Design value of tensile splitting strength in MPa
$F^C{}_{CL}$	Characteristic value of the chloride corrosion rate factor
f_R	Probability density function
f_S	Probability density function

f_t	Tensile strength of the concrete.
G	Limit state function
I	Rainfall intensity (mm/h)
I_{CORR}	Corrosion current density in μA/cm^2
J	Ionic flow, kg/m^{2}s;
J_r	Instantaneous corrosion rate,
k_1, k_2, k_3	Constant
$k_{c,cl}$	Curing factor;
$k^c_{c,res}$	Characteristic value of the curing factor for the resistivity;
$k^c_{RH,res}$	Characteristic value of the humidity factor for the resistivity
$k^c_{cl,res}$	Characteristic value of the factor accounting for the presence of chloride
$k^c_{T,res}$	Characteristic value of the temperature factor for the resistivity
$k_{e,cl}$	Environment factor
L	Lifetime of the structure
L	Limit
m	Water /binder ratio
m_0	Constant for corrosion rate versus resistivity
n	Number of cycles at the end of test
n	Total number of parameters
n_{cl}	Age factor
n^c_{res}	Age factor for the electrolytical resistivity
p	Perimeter of rebar
P	Total porosity
p	Parameter
P_{acc}	Accepted maximum value of the probability of failure
p_C	Probability corrosion
$p_{C,A}$	Reliability of the prediction. T
p^C_0	Characteristic value of the potential electrolytical resistivity;
p_{CR}	Concrete crack width
p^d	Design value of the corrosion penetration in microns;
p^d_0	Design value of corrosion penetration necessary to produce a crack in microns.
P_f	Probability of failure,
$P_{f,T}$	Probability of failure of the structure with the period T.
ppt	Parts per trillion
p_{rust}	Density of corrosion products
p_{st}	Density of the reinforcement
Q_{CR}	Amount of corrosion when concrete cracks (x10^{-4} gm/cm^2)

R	Gas constant
R	Strength
R	Function (model) that describes the bearing capacity or strength of the structure
$R_{CL,0}$	Resistance with respect to the chloride ingress determined on the basis of compliance tests
R^d_{CL}	Design value of the chloride resistance
RH	Relative humidity
s	Addition content of the concrete mix
S	Load
S	Function that describes the influence of the load on the structure
S_{CAP}	Degree of saturation
T	Absolute temperature (Kelvin)
t	Exposure period (s)
T	Intended service period
T, t	Lifetime of the structure
t_0	Reference period (s)
t_{hydr}	Age of the concrete, maximum value one year
T_C	Time to cracking
t_{CORR}	Time of propagation period
t_{CR}	Critical time,
t^d_i	Design value of the time to initiation of corrosion
t_P	Time of depassivation
U	Activation energy of the diffusion process (35000 J/mol)
u	Mobility
V^d	design value of the corrosion rate
V_{PORE}	total pore volume in the concrete, m^3
V_W	volume of water available in the pore system of the concrete, m^3
w	water content of the concrete mix
w	crack width at the concrete surface by the expansive corrosion products
w_0	width of the initial visible crack
w_{CR}	Limit crack width of concrete cover according to the building code.
w^d	Design value of the actual crack width
W_{rust}	Mass of corrosion products that generate the critical tensile stress
w_t	Relative time of wetness
wt	Weight
x	Distance
x_C	Concrete cover
x^d	Design value for cover thickness

X_i	i-th basic variable
z	Number of electrons

α	Factor that takes the influence of age on D_0 into account;
α	Volume of the air contained in the pore system
α^C	Characteristic value of the pitting factor
α_r	Coefficient related to the type of rust products
β	Effect of continued hydration of the cement
β	Reliability index
ΔD	Increase in diameter of reinforcement due to rust formation,
Δx	Δx is the margin for the cover thickness
Δc_{DEV}	Allowance in design for deviation of Xc
ε	Error-term
Φ	Probability distribution function of standard normal distribution
γ	Effect on the concrete skin by being in contact with the seawater
γ	Surface tension pore water
γ_{Ccr}	Partial factor of the critical chloride concentration
$\gamma_{Cs,cl}$	Partial factor for the surface concentration
γ_{Rcl}	Partial factor for the resistance with respect to chloride ingress
γ_V	Partial factor for the corrosion rate
λ	Conductivity
μ	Mean value
ρ^C	Characteristic value of the resistivity,
ρ_{CORR}	Combined density factor for steel and rust,
σ_C	Characteristic value of the stress
σ_T	Minimum stress required to cause the cracking of concrete cover.
σ	Deviation

CPM	Chloride profile method
GGBS	Ground granulated Blastfurnace slag
OPC	Ordinary Portland cement
PDF	Probability density function
PFA	Pulverised fuel ash
PMF	Probability mass function
RCM	Rapid chloride migration
SF	Silica fume

1. INTRODUCTION

1.1 Background

Concrete structures generally make up a significant and important part of the national infrastructure. It is estimated that within Europe structures represent approximately 50% of the national wealth of most countries (Long et al 2001). Both the condition and performance of all these structures are essential for the productivity of the society.

In most EU countries approximately 50% of the expenditures in the construction industry are spent on repair, maintenance and remediation of existing structures. In future, these expenses are expected to increase even more. A large proportion of these expenses are due to problems related to lack of durability of concrete structures (Lindvall 2001). The growing number of deteriorating concrete structures, not only affects the productivity of the society, but has also a great impact on resources, environment and human safety. The operation, maintenance and repair of concrete structures are consuming ever more energy and resources while heavily burdening the environment with the large quantities of waste produced. Thus, the poor durability and premature end of the service life of concrete structures do not only represent technical and economical problems, but also a poor utilization of natural resources and therefore an environmental and ecological problem (Gjørv 2001).

In recent years, an extensive amount of research work has been carried out in order to better understand and control several of the most important deteriorating mechanisms such as alkali aggregate reactions, freezing and thawing and corrosion of embedded steel. In particular, much work has been carried out on corrosion of embedded steel, which represents the greatest threat both to the safety and economy of the structures. Never before so much basic information and knowledge about concrete durability has been available. The great challenge to the professional society is, therefore, to utilize and transform more of this existing knowledge into good and appropriate engineering practice (Gjørv 2002).

The traditional methods used for durability design are usually based on deem-to-satisfy rules, where sufficient durability of a concrete structure is secured with prescriptive requirements. Examples of prescriptive requirements are the usage of specified water/binder ratios, minimum concrete covers and minimum cement content. With the deem-to-satisfy methods it is expected that the concrete structure will achieve a long but not specified service life.

The problems related to durability and execution of work has been underestimated for many years. Main emphasis has been given to mechanical properties and structural capacity, while durability design, construction quality and life cycle management have been neglected. Seldom do the owners of the concrete structures come up with special requirements for durability and long-term performance of their structures (Gjørv 2002).

There is a challenge to the professional society to utilize and transform more of this existing knowledge into good and appropriate engineering practice. So far, only after extensive durability problems have been experienced in the field, that new durability requirements have been introduced in new revised concrete codes. In most countries, the upgrading of the durability requirements in current concrete codes has been far behind the technical and scientific developments. Although most concrete codes have been upgraded several times in recent years, current code specifications for concrete durability are still almost exclusively based on the traditional requirements for concrete composition, construction procedures and curing conditions, the results of which have shown to yield insufficient and unsatisfactory results. However, by a proper utilization of existing knowledge from recent research on new procedures for durability design, improved construction quality and life cycle management, it should be possible to produce new concrete structures with a more controlled durability and long-term performance (Gjørv 2002).

To improve durability implies an extra cost during construction. However, with the deem-to-satisfy rules it is not possible to estimate how an alternative design can reduce costs for future maintenance and repair. This implies that it is necessary to use a design methodology, where it is possible to explicitly design and construct safe, economic and durable structures, and estimate their total lifetime costs. Such design methodology is called performance-based design methodology. The performance can for example be related to load carrying capacity, durability, stiffness, appearance, operability, inspectability and maintainability. A performance based design methodology is based on performance criteria defined for the considered structure. The performance criteria may be related to, for example, a required function or the appearance of the structure.

Usually the performance criteria are defined as limit states. The limit state is the border that separates desired states from the undesired or adverse states in situations, acceptable to the owner, which a structure may be subjected to during its lifetime (DuraCrete 1999). A difference is made between the ultimate limit state, where the safety of the structure is considered (for example the risk of collapse), and the serviceability limit state, where the functionality of the structure is considered (for example the limitation of crack widths, aesthetic appearance). The limit states can be defined by, for example, authorities or the owner of the structure. To determine the probability that the structure is able to meet the performance requirements, with a specified level of reliability, a probabilistic model is formulated.

The probabilistic model is based on sufficiently realistic mathematical models, where the different behaviour of the concrete structure is modelled, e.g. chloride penetration, causing reinforcement corrosion and lowering load-bearing capacity.

1.2 Research Objectives

The main objective of this research work is to develop a probabilistic-based model for the durability analysis of concrete structures in marine environment, and understand how the model parameters influence durability analysis.

A probabilistic-based model is chosen to take into account the inherent variation associated with measurements, with the heterogeneity of the material, and the variation of the environmental factors.

The goal is to provide engineers with an aid that helps to understand the impact their decisions have on the durability performance.

Another goal is the use of performance based criteria in the specification of parameters for the evaluation of concrete durability.

1.3 Research work program

The research project was undertaken at the NTNU - Norwegian University of Technology and Science in Trondheim, Norway. Most of the research performed was done in the concrete and chemical laboratories of the BML - Building Materials Department.

The research project was initiated with an extensive bibliography review into the main topics of service life design, concrete degradation, models for concrete degradation and reliability assessment. Figure 1.1 present a schematic representation of the work plan for the research project.

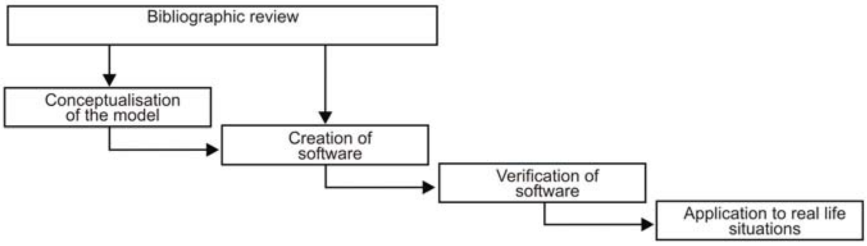


Figure 1.1 - Schematic representation of the work plan.

Based on the information obtained from the bibliography review, a model for the initiation of corrosion induced by chlorides in concrete was developed. The model is implemented in probabilistic version through the use of the Monte Carlo simulation technique.

The next phase is to transform the probabilistic model into a probabilistic analysis software. All the parameters of the software need to be verified and their sensitivity analysed. This is an essential part of the internal validation of the model. It determines the stability of the model and the possibility of error in the programming. Once the results are analysed and accepted, the model and the software need to be validated with real life data.

To obtain the necessary data, several reinforced concrete wharf structures were assessed. Several on site and laboratory test were necessary in order to determine the structures´ concrete properties. This is followed by a statistical quantification of the data collected so that it could be used in the developed methodology.

Finally, the software is applied to the real life structure data and the results are analysed. Conclusions are then drawn on the capabilities of the software.

1.4 Structure of the thesis

The thesis is divided into nine chapters, and includes four appendixes.

The first chapter, the introduction, gives a brief framing of the thesis in nowadays concrete durability research.

The next chapter is a literature review on the durability of reinforced concrete structures. A description of the main degradation processes of reinforced concrete structures is given with special attention to those caused by marine environment. Chloride penetration is detailed as this is the main cause of corrosion of reinforced concrete structures located in marine environment.

The design procedures are also described with special interest in the service life design. Current models for degradation of concrete due to corrosion are discussed. To end this chapter, the current codes of practice are reviewed concerning durability design.

In chapter three, a proposal for a probabilistic model is presented. Chloride induced corrosion of reinforced concrete structures is the mechanism modelled. This chapter is followed by a description of the development of software based on the proposed model. This includes a sensitivity analysis of the variables used in the model.

In chapter five, the applicability of proposed model for durability design of new concrete structures is presented. In addition, the basis for condition assessment of existing concrete structures in marine environment is also presented.

Chapter six presents the information from the in situ assessment of four reinforced concrete structures in marine environment. The test procedure used and the results are presented. These results will serve as the basis for the application of the proposed model, in an attempt to evaluate the service life of these structures. This is performed in chapter seven.

Finally, in chapter eight, the conclusion on this work are presented as well as some remarks on future developments.

The appendixes are made up of four parts: a stochastic background for the models used, given in some detail; the manual for the software developed; and a document with recommendations for reinforced concrete structure design in marine environment, and finally, the data files used in the software for the durability analysis performed in chapter seven.

CHAPTER 2
DURABILITY OF CONCRETE STRUCTURES IN MARINE ENVIRONMENT

2.1 Introduction

To discuss the durability of reinforced concrete structures is a huge undertaking. Even if only the structures in a marine environment and the effect the marine environment had on these structures was considered, the undertaking would still be daunting. The research presented in this thesis is directly related to the durability of reinforced concrete structures in marine environment. Therefore, some key topics that have directly relevance on the research presented are discussed in more detail.

Firstly, however, a brief appreciation of the general degradation processes of reinforced concrete structures is given. Current experience with these processes is discussed, with special attention being paid to the corrosion of concrete reinforcement.

The main degradation process for durability of reinforced concrete structures in marine environment is, without a doubt, the corrosion of the concrete reinforcement. In this specific environment, the corrosion occurs almost entirely due to the presence of chlorides in the sea water. Therefore, the presence of chlorides in concrete is discussed further. The main mechanisms responsible for the transport of chlorides into the concrete are well known and extensive literature exists on this topic alone. Of more interest to the research is the effect of the concentration of chlorides in the concrete has on the corrosion of the reinforcement and the binding of the chlorides by the concrete.

The research does not, however, deal only with corrosion, but also with the service life of reinforced concrete structures in marine environment. The inadequate durability is by far the most common cause of premature degradation of concrete structures, yet little attention is given to durability in the design process. Generally, durability is covered by prescriptive code requirements based on previous code clauses. Service life design and service life prediction are currently an area of significant international research. The service life concept is described in further detail in association with concrete durability design.

The bases for durability design are the models that simulate the phenomenon. These are discussed in some detail regarding the corrosion of reinforcement in concrete.

Finally, a brief overview of current codes for concrete durability is presented. In addition to the European codes, special attention is given to the Portuguese and Norwegian codes as well.

2.2 Degradation processes of reinforced concrete structures

Reinforced concrete is a versatile, economical and widely used construction material. It can be moulded to a variety of shapes and finishes. Usually it is durable and strong, performing well throughout its service life. However, it does not perform adequately as a result of poor design, poor construction, inadequate materials selection, exposed to a more severe environment than anticipated or a combination of these factors.

In most countries, concrete structures make up a very large and important part of the national infrastructure, and both the condition and performance of these structures are important for the productivity of the society (Grigg 1988). Since there is a growing amount of deteriorating concrete structures, however, not only the productivity of the society is affected, but it also has a great impact on resources, environment and human safety. The operation, maintenance and repair of concrete structures are consuming much energy and resources and are producing a heavy environmental burden and large quantities of waste. Thus, the poor durability and premature service life of many concrete structures do not only represent technical and economical problems. This is poor utilization of natural resources, and hence, also an environmental and ecological problem (Gjørv 2000).

In recent years, an extensive amount of research work has been carried out in order to better understand and control several of the most important deteriorating mechanisms such as alkali-aggregate reactions, freezing and thawing and corrosion of embedded steel. In particular, much work has been carried out on corrosion of embedded steel, which represents the greatest threat both to the safety and economy of the structures. Never before has so much basic information and knowledge about concrete durability been available. The great challenge to the professional society is, therefore, to utilize and transform more of this existing knowledge into good and appropriate engineering practice.

No single parameter controls the durability of concrete. Instead, there are a number of contributing factors which affect its durability. In practice, several degradation mechanisms can act simultaneously with possible synergistic effects. The schematic diagram below illustrates how different degradation mechanisms can act on concrete exposed to sea water.

Improper procedures or carelessness during any phase of the construction operation results in concrete of inferior quality. Poor transportation, placing, finishing techniques, and inadequate curing conditions are a few examples.

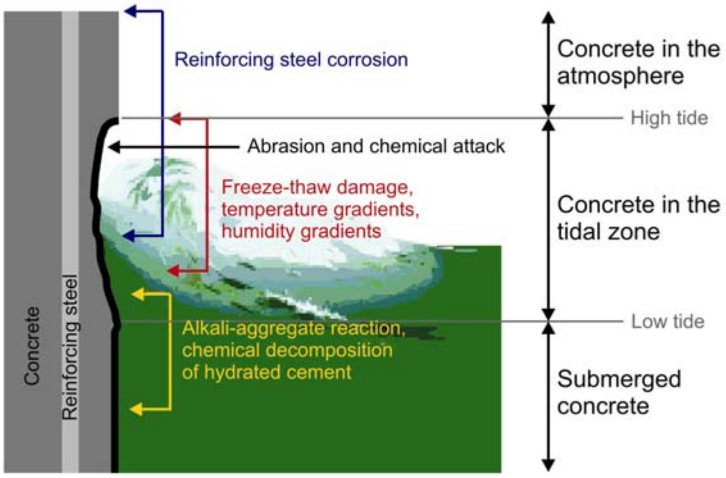


Figure 2.1 - Possible degradation mechanisms acting on concrete exposed to sea water (Malhorta 2000).

The design of concrete structures, while in full accordance with the requirement of the design specification, do not necessarily work well in practice. A survey of structures usually indicates that deterioration occurs repeatedly in connection with certain details, or that certain effects occur which were not anticipated in design. For example, inadequate design, which fails to allow creep of structural elements of a building (e.g. deflection of floors), may result in the load being transferred to non-structural elements, such as partition walls or cladding panels, and cracking and damage often result (Slater 1980).

In designing, there is a need to trace the anticipated water flow over the whole surface of both vertical and horizontal members. Design should include the provision of desired concrete quality to resist the adverse effects produced by in-situ exposure conditions. Thus, the designer needs to be better informed on the particular characteristics of the given environment so that a proper choice in matching the concrete selected with the characteristics of the environment can be made.

The quality of concrete with respect to durability can be measured in terms of various parameters. These properties of hardened concrete are governed by the micro and macro structure of concrete which is essential in its ability to resist chemical attack by external sources (e.g., acids, carbon dioxide, and sulphates), from within the concrete (e.g., alkali-aggregate reaction (AAR) and unsound cement), and from other environmentally induced distress related to moisture ingress (e.g., freeze-thaw cycling, leaching). These parameters regarding concrete quality are affected by the quality of cement and aggregate, the w/c ratio and degree of hydration, the effectiveness of compaction, the extent of curing, and the presence of cracks.

Exposure conditions vary considerably from location to location. Whatever the in-situ conditions, the designer must acquaint himself with the factors, results of a given environment, and only then design a concrete to meet those demands or take measures to protect the concrete from the aggressive conditions. Having designed the concrete, the specifier should then institute proper quality control procedures to ensure that the placed concrete will meet the specified features.

2.2.1 Modes of concrete deterioration

Deterioration is any adverse change of normal, mechanical, physical, and chemical properties either in the surface or in the body of concrete, generally due to the disintegration of its components (Anonymous 1987, Masters 1987, Cady 1990, Higgins 1981) The phenomenon which induces such distress may be associated with one of the phases (e.g. design, construction, or service). The effects of deterioration may or may not be manifested visually. There are three basic visual symptoms of distress in a concrete structure:

- cracking,
- spalling, and
- disintegration.

Although each of the basic symptoms is readily differentiated from the others, each occurs in several forms, each having a different significance. Furthermore, in a given structure, the three basic indicators of distress may occur not only in combination, but with several forms of each symptom being manifested simultaneously.

In addition to the deterioration of concrete, the breakdown of other auxiliary materials, such as sealants, coatings, membranes, which form the complex assembly of a structure, should also be considered. Polymeric products often interact with other materials with which they are in contact to form compounds which are devoid of the characteristics of the original

In figure 2.2 the different forms of deterioration of a reinforced concrete structure are presented. Degradation agents can he defined as any group of factors that can affect the performance of a building material, component, or system.

Climatic factors such as solar radiation, temperature, water, air contaminations, and wind are particularly important in the degradation of materials used in the exterior envelope of buildings. But the range and importance of these agents vary widely with type of climate, geographic location, time of the year, and even within a relatively small area (microenvironment). In actual service, degradation factors may interact to increase the rate of degradation, or less often, to decrease the rate by one factor cancelling the effect of others.

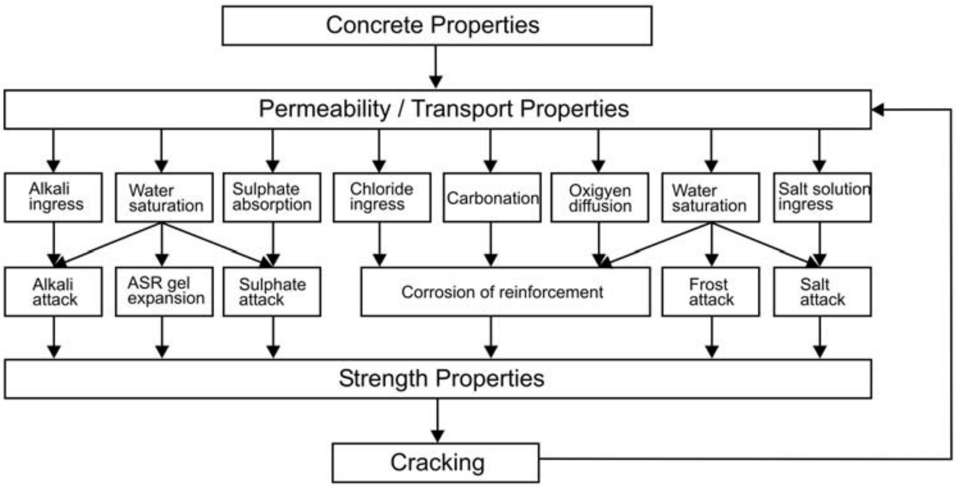


Figure 2.2 - The deterioration of concrete structures (Long et al 2000).

Degradation of material properties mostly sets in under combined action of internal and external factors. It is a complex process determined largely by the physiochemical properties of the material (internal) and the manner in which it is used (external). The processes or reactions lead to a chance in the ability of a material or component to perform as intended. Factors internal to a material are those which determine its quality (i.e., the way it is made, placed, and cured), and others such as shrinkage, creep, and thermal effects which are inherent in its nature. External causes of deterioration are broadly grouped as physical, chemical, or mechanical. Main physical factors are the fluctuations of moisture content, temperature, freezing and thawing (that occurs in natural weathering), and fire. The main chemical factors are aggressive gases and liquids, and the main mechanical factors are load, friction and vibration.

Basic environmental conditions have marked effects on the deterioration process. The effects of temperature on rate of reactions, solubility of salts, and the effect of relative humidity on carbonation and corrosion are good illustrations of how the effect of ambient conditions affects deterioration rates. As a rule of thumb, increases in the temperature range of 10°C will approximately double the reaction rate.

Following is a brief description of the main degradation mechanisms for reinforced concrete structures, including an analysis to the knowledge obtained from practical experience. Further descriptions of the deterioration of reinforced concrete can be found in the following publications: Frederiksen et al (1997), DuraCrete (1998a), Basheer et al (2001), Sandberg

(1998), Möller (1994), Neville (1997), Betonghandboken - Material (Concrete handbook - Material) (1994) etc.

2.2.2 Corrosion of reinforcing steel

The corrosion of reinforcing steel in concrete is a major problem facing civil engineers and surveyors today as they maintain an ever increasing ageing infrastructure. The economic loss and damage caused by the corrosion of steel in concrete makes it arguably the largest single infrastructure problem facing industrialized countries. Bridges, public utilities and buildings are ageing. Some can be replaced, others would cause great cost and inconvenience if they were taken out of commission. With major political arguments about how many more bridges and other structures we can build, it becomes crucial that the existing structures perform to their design lives and limits and are maintained effectively.

Reinforcement corrosion has been widely reported in the literature over the last two to three decades. In general, there are two major factors, which cause corrosion of reinforcement in concrete to proceed to an unacceptable degree. They are carbonation and the presence of chloride ions, which may either have been present in the concrete constituent's right from the beginning or are introduced into the concrete through ingress from the surrounding environment during the service life.

Although carbonation-induced corrosion still represents a durability problem for many concrete structures, it is primarily an uncontrolled penetration of chlorides which represents the most technically difficult and serious problem to the durability and safety of concrete structures (Gjørv 1996). Already in 1917, Wig and Furguson pointed out the problem with steel corrosion in concrete structures exposed to marine environment after a comprehensive survey of concrete structures in U.S. waters.

Concrete provides a high degree of protection to the reinforcing steel against corrosion, due to the high alkalinity ($pH \approx 13$) of the pore solution. Under high alkalinity steel remains passivated. In addition, well-consolidated and properly cured concrete with a low w/c ratio has a low permeability, which minimizes penetration of corrosion inducing agents, such as chloride, carbon dioxide, moisture, etc. to the steel surface. Furthermore, the high electrical resistivity of concrete restricts the rate of corrosion by reducing the flow of electrical current from the anodic to the cathodic sites. In resume, if the concrete is properly designed, applied and maintained, there should be little problem of steel corrosion during the design life of the structures. Unfortunately, the durability requirements are not always achieved in practice due to which the corrosion of reinforcement in concrete has become a commonly encountered cause of deterioration.

Once reinforcement corrosion is initiated, it progresses almost at a certain rate (depending on availability of oxygen and humidity, etc.) and shortens the service life of the structure, by causing surface cracking and subsequently spalling of the cover concrete due to expansion of the corroding steel. The rate of corrosion directly affects the extent of the remaining service life of a corroding reinforced concrete structure.

The most exposed types of structures include bridges and parking garages, port and harbour structures as well as a number of other concrete structures in marine environments.

2.2.2.1 Mechanism of corrosion of steel in concrete

The strongly alkaline nature of concrete, due to $Ca(OH)_2$ with a pH of about 13, prevents the corrosion of the steel reinforcement by the formation of a thin protective film of iron oxide on the metal surface. This protection is known as passivity. However, if the concrete is permeable to the extent that carbonation reaches the concrete in contact with the steel or soluble chlorides can penetrate right up to the reinforcement, and water and oxygen are present, then corrosion of reinforcement will take place. The passive iron oxide layer is destroyed when the pH falls below about 11.0. Carbonation lowers the pH to about 9. The formation of rust results in an increase in volume compared with the original steel so that swelling pressures will cause cracking and spalling of the concrete.

Corrosion of steel occurs because of the electro-chemical action which is usually encountered when two dissimilar metals are in electrical contact in the presence of moisture and oxygen. However, the same process takes place in steel alone because of differences in the electro-chemical potential on the surface, which forms anodic and cathodic regions, connected by the electrolyte in the form of the salt solution in the hydrated cement. The positively charged ferrous ions Fe^{2+} at the anode pass into solution while the negatively charged free electrons e^- pass along the steel into the cathode, where they are absorbed by the constituents of the electrolyte and combine with water and oxygen to form hydroxyl ions $(OH)^-$. These then combine with the ferrous ions to form ferric hydroxide and this is converted by further oxidation to rust (see figure 2.3(a)). Thus, it can be written:

$$Fe \rightarrow Fe^{2+} + 2e^- \quad \text{(anodic reaction)} \tag{2.1}$$

$$4e^- + O_2 + 2H_2O \rightarrow 4(OH)^- \text{ (cathodic reaction)} \tag{2.2}$$

$$Fe^{2+} + 2(OH)^- \rightarrow Fe(OH)_2 \text{ (ferrous hydroxide)} \tag{2.3}$$

$$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3 \text{ (ferric hydroxide)} \tag{2.4}$$

These differences in potential are due to the inherent variation in structure and composition (e.g., porosity and the presence of a void under the rebar or difference in alkalinity due to carbonation) of the concrete cover, and differences in exposure conditions between adjacent parts of steel (e.g., concrete that is partly submerged in sea water and partly exposed in a tidal zone). The reactions involved in the process can be represented by the following schematic equations:

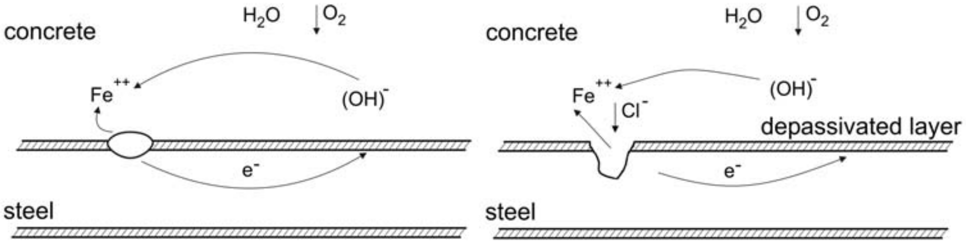
$2Fe$ (metal) —> $2Fe^{2+} + 4e^-$ (anodic reaction) (2.5)

$2H_2O + O_2 + 4e^- \rightarrow 4OH^-$ (cathodic reaction) (2.6)

$2Fe^{2+} + 4OH^- \rightarrow 2Fe(OH)_2$ (ferrous hydroxide) (2.7)

$2Fe^{3+} + 6OH^- \rightarrow 2Fe(OH)_3$ (ferric hydroxide) (2.8)

$2Fe(OH)_3 \rightarrow Fe_2O_3 + 3H_2O$ (ferric oxide) (2.9)



a) b)

Figure 2.3 - Schematic representation of electro-chemical corrosion: (a) electrochemical process, and (b) electro-chemical corrosion in the presence of chlorides (Neville 1997).

It is shown that oxygen is consumed, but water is regenerated and is needed only for the process to continue. Thus there is no corrosion in a completely dry atmosphere, probably below a relative humidity of 40 per cent. Nor is there much corrosion in concrete fully immersed in water, except when water can entrain air. It has been suggested that the optimum relative humidity for corrosion is 70 to 80 per cent. At higher relative humidities, the diffusion of oxygen is considerably reduced and also the environmental conditions are more uniform along the steel.

Chloride ions present in the cement paste surrounding the reinforcement react at anodic sites to form hydrochloric acid which destroys the passive protective film on the steel. The surface of the steel then becomes activated locally to form the anode, with the passive surface forming the cathode: the ensuing corrosion is in the form of localized pitting. In the presence of chlorides, the schematic reactions are (see figure 2.3(b)):

$Fe^{2+} + 2C1^- \rightarrow FeCl_2$ - (2.10)

$FeCl_2 + 2H_2O \rightarrow Fe(OH)_2 + 2HCl$ (2.11)

Thus, Cl^- is regenerated. The other reactions, and especially the cathodic reaction, are as in the absence of chlorides. It should be noted that the rust contains no chloride, although ferric chloride is formed at an intermediate stage.

Because of the acidic environment in the pit, once it has formed, the pit remains active and increases in depth. Pitting corrosion takes place at a certain potential, called the pitting potential. This potential is higher in dry concrete than at high humidities. As soon as a pit has started to form, the potential of the steel in the neighbourhood drops, so that no new pit is formed for some time. Eventually, there may be a large-scale spread of corrosion, and it is possible that overall and general corrosion takes place in the presence of large amounts of chloride.

2.2.2.2 Factors affecting corrosion of steel in concrete

The main causes of corrosion of steel in concrete are chloride attack and carbonation. These two mechanisms are unusual in that they do not attack the integrity of the concrete. Instead, aggressive chemical species pass through the pores in the concrete and attack the steel. This is unlike normal deterioration processes due to chemical attack on concrete. Other acids and aggressive ions such as sulphate destroy the integrity of the concrete before the steel is affected. Most forms of chemical attack are therefore concrete problems before they are corrosion problems. Carbon dioxide and the chloride ion are unusual as they penetrate the concrete without significantly damaging it.

The factors affecting corrosion of steel in concrete may be classified into two major categories: external factors and internal factors.

External factors affecting corrosion of steel in concrete - they include mostly environmental parameters, such as:

- *Availability of oxygen and moisture at reinforcement level:* presence of moisture and oxygen supports the corrosion. Moisture fulfills the electrolytic requirement of the corrosion cell, and moisture and oxygen together help in the formation of more OH^- thereby producing more rust component, i.e., $Fe(OH)_2$. Oxygen also affects the progress of cathodic reactions. In the absence of oxygen, even in a situation of depassivation, corrosion will not progress due to cathodic polarization.

In structures which are submerged or exposed to long-term or cyclic water application that causes water saturation of the concrete for periods of several weeks, the availability of oxygen is the only limiting factor for the corrosion rate of the reinforcement when the concrete surrounding the steel is water saturated and most of the oxygen within the concrete near the reinforcement surface has been consumed by the cathodic reaction of the corrosion process. Therefore, in the case of common outdoor structures being exposed to rain and not submerged or constantly water saturated due to other reasons, no reduction of the corrosion rate induced by limited oxygen diffusion is to be expected (Raupach 1996).

• *Relative humidity:* The moisture condition is one of the parameters that governs the hardening of concrete, especially close to the surface, and consequently governs the permeability to gases, water and ions. Moisture variations cause shrinkage and shrinkage cracking. Moisture plays a significant role in chemical reactions in concrete and in physical and chemical processes in various deterioration phenomena.

The initiation time for reinforcement corrosion is highly influenced by the moisture content since high moisture delays the intrusion of carbon dioxide (diffusion of carbon dioxide is far slower than in the gas phase), however a certain moisture content is necessary for carbon dioxide to react with the portlandite ($Ca(OH)_2$). Chlorides need moisture to penetrate the concrete. In the splash zone of a marine structure moisture plays a more active role when salts penetrate due to convection, moving with the water and depositing where and when the moisture evaporates. When the corrosion starts the rate of corrosion is influenced by moisture to a great extent. In dry conditions (electrolytic resistance of the concrete) or very wet conditions (O_2 diffusion becomes the controlling factor) the rate is slow but intermediate moisture conditions give an electrolyte and permits the intrusion of oxygen to the corrosion process. Therefore, knowledge of the amount of pores described by the porosity, the pore size distribution and the water saturation degree of the pores are vital parameters to understand the electrochemical processes in concrete.

• *Temperature:* Temperature has a large influence on the corrosion process of steel in concrete, especially, on the corrosion potential, the corrosion rate, concrete resistivity and transport processes in concrete.

A rise in temperature may result in a two fold effect: the electrode reaction rates are generally increased, and the oxygen solubility is increased resulting in a increase in the rate of corrosion (Mazer 1965). If the situation is conducive for corrosion to take place, the corrosion rate is increased by high temperature and high humidity (Uhlig 1983).

- *Carbonation and penetration of acidic gaseous pollutants to the reinforcement level:*

Carbonation is the result of the interaction of carbon dioxide gas in the atmosphere with the alkaline hydroxides in the concrete. Like many other gases carbon dioxide dissolves in water to form an acid. Unlike most other acids the carbonic acid does not attack the cement paste, but just neutralizes the alkalis in the pore water, mainly forming calcium carbonate that lines the pores:

$$CO_2 + H_2O \rightarrow H_2CO_3 \tag{2.12}$$

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O \tag{2.13}$$

There is a lot more calcium hydroxide in the concrete pores than can be dissolved in the pore water. This helps to maintain the pH at its usual level of around 13 as the carbonation reaction occurs. However, eventually as the locally available calcium hydroxide reacts, precipitating the calcium carbonate and allowing the pH to fall to a level which may cause initiation of reinforcement corrosion, loss of passivity of concrete against reinforcement corrosion, and reinforcement corrosion (Berkeley 1990), as indicated in Table 2.1.

Table 2.1 – State of reinforcement corrosion at various pH levels (Berkeley 1990)

pH of Concrete	State of reinforcement corrosion
Below 9.5	Commencement of steel corrosion
At 8.0	Passive film on the steel surface disappears
Below 7	Catastrophic corrosion occurs

Carbonation damage occurs most rapidly when there is little concrete cover over the reinforcing steel. Carbonation can occur even when the concrete cover depth to the reinforcing steel is high. This may be due to a very open pore structure where pores are well connected together and allow rapid CO_2 ingress. It may also happen when alkaline reserves in the pores are low. These problems occur when there is a low cement content, high water cement ratio and poor curing of the concrete.

The carbonation rate reaches a maximum at a RH value range of 50 to 70%, while above and below this range, the rate is significantly slowed down. Corrosion may commence once a critical RH of about 50% has been reached and gradually increase up to a RH around 100%, at which point, it decreases due to lack of oxygen.

- *Aggressive anions reaching the reinforcement level:*

The corrosion of the reinforcement due to chloride ions from deicing salts or seawater is the main cause of damage and early failure of reinforced concrete structures. A high chloride concentration in the concrete cover results in depassivation. The depassivation mechanism for chloride attack is somewhat different. The chloride ion attacks the passive layer but, unlike carbonation, there is no overall drop in pH. Chlorides act as catalysts to corrosion when there is sufficient concentration at the reinforcement surface to break down the passive layer. They are not consumed in the process but help to break down the passive layer of oxide on the steel and allow the corrosion process to proceed quickly. Obviously a few chloride ions in the pore water will not break down the passive layer, especially if it is effectively re-establishing itself when damaged.

Severe corrosion attack usually occurs when alternating drying/wetting cycles take place, typically in the splash zone of a structure. It is generally recognized that only the ‘‘free chloride’’ ions influence the corrosion process (Arya 1987). It is reported (Hope 1987) that the resistivity decreases and corrosion rate increases with an increase in the chloride content. However, the change in pH is found to be insignificant due to a change in the chloride content of concrete (Hope 1987). The risk of reinforcement corrosion associated with the levels of chloride content in both uncarbonated and carbonated concrete is presented in Table 2 (Pullar-Strecker 1987).

Table 2.2 - Corrosion risk in concrete containing chlorides (Pullar-Strecker 1987)

Total chloride (wt.% of cement)	Condition of concrete adjacent to reinforcement	Corrosion risk
Less than 0.4%	Carbonated	High
	Uncarbonated, made with cement containing less than 8% C_3A	Moderate
	Uncarbonated, made with cement containing 8% or more C_3A	Low
0.4%-1.0%	Carbonated	High
More than 1.0%	Uncarbonated, made with cement containing less than 8% C_3A	High
	Uncarbonated, made with cement containing 8% or more C_3A	High
	All cases (carbonated)	High

Chloride penetration into concrete determines the time to depassivation initiation of localised corrosion and is thus one of the most decisive processes for durability and service life of reinforced concrete structures. Water and chlorides are transported rapidly into concrete by capillary suction. Besides the dominating influence of concrete porosity and pore size distribution the rate and amount of chloride ingress are related to the humidity gradients present in the concrete matrix as well as to the chemical properties of the hardened cement. Usage of cements with high C_3A ($3CaO{\cdot}Al_2O_3$) content is considered to be conducive to good resistance to corrosion because of its ability to bind chlorides chemically by forming calcium chloro-aluminate, $3CaO{\cdot}Al_2O_3{\cdot}CaCl_2{\cdot}10H_2O$, sometimes referred to as Friedel's salt. However, sulphate attack from seawater results in a decomposition of calcium chloro-aluminate, thus setting chlorides free by formation of calcium sulpho-aluminate (Neville 1995). Chlorides become therefore again available for the corrosion process.

Carbonation of hardened cement paste in which bound chlorides are present has a similar effect on setting free the bound chlorides and thus increasing the risk of corrosion. It has been found that the presence of even a small amount of chloride ions in carbonated concrete enhances the rate of corrosion induced by low alkalinity of carbonated concrete (Glass et al 1991). Whereas active corrosion of the reinforcing steel bars can be detected in the laboratory and onsite by non-destructive electrochemical techniques as potential measurements, no such possibility exists for the detection and quantification of chloride ions in concrete. Only qualitative information on chloride distribution can be obtained from potential mapping.

- *Stray currents:* Stray currents from the various sources, e.g., building power supply systems, cathodic protection systems, locomotive power supply systems, etc. cause electrolytic corrosion (Uhlig 1971).

- *Bacterial action:* Bacterial action is found to be effective in three ways:
  - The bacteria decrease the amount of cover by disintegration of the cementitious materials (ASTM STP818 1983);
  - The anaerobic bacteria produce iron sulfides in the oxygen deficit condition, such as concrete sewers, which enables the corrosion reaction to proceed even in the absence of oxygen (Berkeley 1990), and
  - aerobic bacteria may also aid in the formation of differential aeration cells, which can lead to corrosion.

Internal factors affecting reinforcement corrosion - they include concrete and steel quality parameters, as discussed below:

- *Cement composition:* The cement in the concrete provides protection to the reinforcing steel against corrosion by maintaining a high pH in the order of 12.5-13 owing to the presence of $Ca(OH)_2$ and other alkaline materials in the hydration product of cement, and by binding a significant amount of total chlorides as a result of chemical reaction between C_3A and C_4AF content of cement in concrete. Thus the threshold chloride value shifts to higher side with an increase in the C_3A content of cement (Rasheeduzzafar 1992). The use of blended cement, such as microsilica-blended high-C_3A cement, is found to be concomitantly resistant to sulfate attack and chloride corrosion of reinforcement (Rasheeduzzafar et al 1990).

• *Impurities in aggregates:* Aggregates containing chloride salts cause serious corrosion problems, particularly those associated with sea-water and those whose natural sites are in ground water containing high concentration of chloride ions (ACI C201 1991).

• *Impurities in mixing and curing water:* Mixing and curing water, either contaminated with sufficient quantity of chloride or being highly acidified due to any undesirable substance present in water, may prove to be detrimental as far as corrosion of reinforcement is concerned.

• *Admixtures:* Addition of calcium chloride in concrete, as a common admixture for accelerating the hydration of cement is perhaps the most significant reason for the presence of chloride in reinforced concrete structures exposed to normal weather conditions. Some water reducing admixtures also contain chlorides (ACI C201 1991).

• *w/c ratio:* Low w/c ratio decreases the concrete permeability, which in turn reduces the chloride penetration, carbonation penetration, and oxygen diffusion in concrete. However, a low w/c ratio does not by itself assure concrete of low permeability. For example, 'no fines' concrete can have a low w/c ratio and yet be highly permeable. Thus, in addition to the low w/c ratio, the concrete must be properly proportioned and well consolidated to produce a concrete of low permeability.

When reinforced concrete structures are immersed in some aggressive solution, it is the permeability of concrete, which is a function of w/c ratio, affects the corrosion of rebar. The depth of penetration of a particular chloride threshold value increases with an increase in the w/c ratio (Jaegermann 1990). Carbonation depth has been found to be linearly increasing with an increase in the w/c ratio (Ho et al 1987). The oxygen diffusion coefficient is also found to be increasing with an increase in the w/c ratio (Kobayashi 1991).

Goto and Roy (1981) have found a 100-fold increase in the permeability of hardened cement paste when the w/c ratio was increased from 0.35 to 0.45. Ferreira (2000) found a 50-fold increase in the permeability of hardened cement paste when the w/c ratio was increased from

0.40 to 0.65. Al-Saadoun (1992) has observed that the time to initiation of reinforcement corrosion in a sample with w/c ratio of 0.4 is 2.15 to 1.77 times more than that in a sample with a w/c ratio of 0.55, under the accelerated corrosion testing. Al-Amoudi (1985) has reported that permeability is significantly reduced for a w/c ratio below 0.45. From these observations it is to be noted that the w/c ratio should be less than 0.45 and preferably around 0.40 to obtain good durability of concrete. Suitable dosage of admixtures may be added to obtain the desired workability at low w/c ratio.

• *Cement content:* The cement content in concrete does not only affects the strength but it also has a significant effect on durability. Due to reduced amount of cement in mix the concrete is not consolidated properly leading to the formation of honeycombs and other surface defects. These honeycombs and surface defects help in the penetration and diffusion of corrosion causing agents, such as Cl^-, H_2O, O_2, CO_2, etc., in concrete. This results in the initiation of reinforcement corrosion due to the formation of differential cells. Furthermore, concrete with low cement content has a lack of plastic consistency due to which it does not form a uniform passive layer on the surface of the steel bars. Therefore, it is important to maintain minimum cement content from the durability point of view.

• *Aggregate size and grading:* Since the size of aggregates has a bearing upon the consistency of concrete, it may have an effect upon reinforcement corrosion. Aggregate grading is another factor, which should be considered for high quality impermeable concrete. It has been observed that for a given w/c ratio, the coefficient of permeability of concrete increases considerably with increasing size of aggregates (Verbeck 1968). Keeping this in view, Cordon and Gillespie (1963) have recommended maximum size of aggregate as: 25-50 mm for 35 MPa concrete, 18.5 mm for 40 MPa concrete and 9 to 12.5 mm for concrete with a compressive strength of more than 40 MPa.

The proportioning of coarse and fine aggregates is important for the production of a workable and durable concrete. The aggregate proportioning for this purpose consists of fixing the optimum volume fraction of sand in the total aggregate content. Studies conducted by Maslehuddin (1981) reveal that both 'too much' and 'too less' sand reduces the workability of concrete. The optimum proportioning for a maximum workability has been reported to be corresponding to a specific surface area of the combined aggregates in the range of 70-75 cm^2/cm^3 for concrete with a cement content in the range of 300-390 kg/m^3 (Maslehuddin 1981). This enhanced workability on account of an optimum aggregate grading allows a reduction in the w/c ratio resulting in increased strength and durability of concrete.

- *Construction practices:* Serious corrosion problems may occur if enough care, such as listed below, is not taken at the construction stage (Rasheeduzzafar 1989): (i) aggregate washing for deleterious materials, if any; (ii) control of chloride in almost all ingredients of concrete, i.e., water, cement, aggregate, and admixtures; (iii) strict enforcement of designed and recommended levels of w/c ratio, cement content, cover thickness, etc.; (iv) proper consolidation of freshly placed concrete; and (v) proper curing of concrete.

- *Cover over reinforcing steel:* Cover depth has a significant effect in case of corrosion due to penetration of either chloride or carbonation (Beeby 1978). This effect of cover is limited within the time of casting to the time at which the reinforcement is depassivated and corrosion is initiated. The rate of corrosion, once it has started, is independent, among other things, on the cover thickness as well (Schiessel 1975).

Risk of reinforcement corrosion with low cover thicknesses, has been reported by various researchers (Rasheeduzzafar et al 1985, Dakhil et al 1975, Tyler 1960). Rasheeduzzafar et al. (1986), based on their field and laboratory studies, have recommended the safe cover thicknesses for reinforced concrete structures exposed to the various aggressive environments of the Arabian Gulf. Codes of practices on design of reinforced concrete structures also specify cover thicknesses for various types of exposures. Cover thickness is one of the factors, which affects the cracking and spalling of the concrete due to the reinforcement corrosion.

- *Chemical composition and structure of the reinforcing steel:* The differences in the chemical composition and structure of reinforcing steel and presence of stress in the reinforcement, either static or cyclic, create different potentials at different locations on the surface of reinforcement, causing the formation of differential corrosion cells, which leads to its corrosion (Mozer 1965).

- *pH of the concrete porewater:* In reinforced concrete structures steel is protected against corrosion by the high alkalinity of the concrete porewater resulting in a passive film on steel. Concrete, with its continuous pore system and tendency to form surface cracks, is far away from being a perfect barrier. The real importance of the concrete cover is related mainly to its ability to preserve the conditions of high pH needed to maintain the reinforcement in a passive condition by preventing the rate of ingress of "acidic substances" from the external environment causing a lowering in pH. Generally those are atmospheric carbon dioxide and, in polluted locations, other gases such as sulphur dioxide. CO_2, reacts with the alkaline constituents of the cement paste to form a carbonated zone, which gradually penetrates into exposed concrete, reducing the pH of the affected region to a value where corrosion will occur.

This carbonation of concrete is characterised by a pH shift of about 4 units to lower values. Carbonation monitoring is therefore possible by pH measurements. As the carbonation front is generally well defined, requirements on the accuracy of pH sensors are not necessarily high, but are required for long term performance.

In combination with chloride measurements the ratio of $[Cl^-]/[OH^-]$ describing the real corrosiveness of the concrete pore solution could be assessed. The so-called critical chloride concentrations do not usually consider the role of pH.

2.2.2.3 Effect of reinforcement corrosion on structural behaviour (Corrosion Damage)

In most industries corrosion is a concern because of wastage of metal leading to structural damage such as a collapse, perforation of containers and pipes, etc. Most problems with corrosion of steel in concrete are not only due to loss of steel but also the growth of the oxide. This leads to cracking and spalling of the concrete cover.

The important factors in corrosion of steel in concrete compared with most other corrosion problems are the volume of oxide and where it is formed. A dense oxide formed at high temperatures (such as in a power station boiler) usually has twice the volume of the steel consumed. In most aqueous environments the excess volume of oxide is transported away and deposits on open surfaces within the structure. For steel in concrete two factors predominate. The main problem is that the pore water is static and there is no transport mechanism to move the oxide away from the steel surface. This means that all the oxide is deposited at the metal/oxide interface. The second problem is that the oxide is not dense. It is very porous and takes up a very large volume up to ten times that of the steel consumed (Neville 1995). Other authors suggest values between 2-7 times (Neville 1995). These depend on the composition of the concrete and the environmental conditions. The thermodynamics of corrosion, coupled with the low tensile strength of concrete, means that the formation of oxide breaks up the concrete. It has been suggested that less than 100 μm of steel section loss are needed to start cracking and spalling the concrete. The actual amount needed will depend upon the geometry in terms of cover, proximity to corners; reinforcement spacing, bar diameter, rate of corrosion, and the deformability and tensile strength of concrete.

The corrosion of reinforcing steel has the following major detrimental effects on the durability of reinforced concrete structures. The voluminous corrosion product formed during corrosion of the steel exerts a tensile stress on the concrete cover. As the corrosion product grows, the tensile stresses increase until they become high enough to crack the concrete cover. The effects of corrosion are usually threefold: (1) cracking of the concrete along the line of the reinforcement,

(2) rust staining of the concrete surface, and (3) spalling of the concrete away from the rebar, leaving it exposed to the environment and to further corrosion.

Due to the loss of cover concrete there may be significant reduction in the load bearing capacity of the structure and the bond between concrete and steel. The reinforcement may be more accessible to the aggressive agents leading towards further corrosion at an accelerated rate, and secondly, corrosion reduces the cross-sections of the steel and thereby the load carrying capacity of the structure. Pitting (i.e. localized) corrosion of the reinforcement is more dangerous than uniform corrosion because it progressively reduces the cross-sectional area of reinforcement to a point where the reinforcement can no longer withstand the applied load leading to failure of the structure.

A visual survey of a corroding structure provides valuable information as to, whether the corrosion of reinforcement is really a cause of distress or there is some other cause of distress. This survey consists of a careful investigation of the structure for any sign of distress, such as cracking, spalling, and rust staining. If visual inspection of the structure suggests that the cause of distress is the corrosion of reinforcement only, the next step is to make a careful examination of the structure and carry out detailed tests which will positively identify the cause and extent of the distress, and allow prediction to be made about the remaining service life of the structure.

2.2.2.4 Experience

The corrosion rate is dependent on if the reinforcement corrosion has been initiated by chloride ingress or carbonation. In extreme cases the corrosion rate can be up to 5 mm/year for corrosion initiated by chloride ingress and 0.05 mm/year for carbonation-initiated corrosion. There are several methods available to prevent corrosion and reduce the corrosion rate. The general principles of the methods are to make the concrete less permeable and protect the steel. Examples of methods are, Taylor (1997):

- Cathodic protection of the reinforcement. A voltage is applied between the reinforcement and a conducting paint applied to the outer surface of the concrete.
- Sacrificial anodes. An electropositive metal or alloy, e.g. zinc, is embedded in the concrete and connected electrically to the reinforcement.
- Corrosion inhibitors. A chemical compound, e.g. $Ca(NO_2)_2$, is used in the concrete mix to stabilize and/or alter the oxide film on the steel. A division is made in three groups, depending on their primarily action, Sharobim (1990): (1) Anodic, (2) Cathodic (3) mixed.
- Coating of the steel or concrete surfaces. The surface of the steel is coated, with e.g. epoxy, to prevent corrosion to initiate. The surface of the concrete is painted to decrease the

permeability of the concrete cover and also influence the moisture conditions in the concrete in a positive way.

- Alternative reinforcements (stainless steel, galvanized, epoxy coated, etc.)

These methods should be used with care since the result is, to large extent, dependent on the way they are applied. If, for example, cathodic protection and/or sacrificial anodes are used, it is important that the reinforcement bars have electrical contact with one another, otherwise the protective effect will be partly lost. If the reinforcement is coated with e.g. epoxy, a crack in the coating may destroy its protective function, and even make the problem worse due to pitting effects.

In many countries, deicing salt has created serious problems due to corrosion in concrete bridges and parking garages. Only in the US, it was estimated in 1986 that the cost of correcting all the corroding bridges was $ 24 billion, with an annual increase of $ 500 million (Gjørv 2002).

Marine environments may represent even more severe conditions (Gjørv 1996). Recent extensive field investigations in Norway have shown that more than 50% of all the larger concrete bridges along the Norwegian coastline either had a varying extent of steel corrosion or had been repaired due to steel corrosion (Gjørv 2002). Most of these bridges were built during the last 25 years, of which one was so heavily corroded that it had to be demolished already after a service period of 25 years (Gjørv 2002).

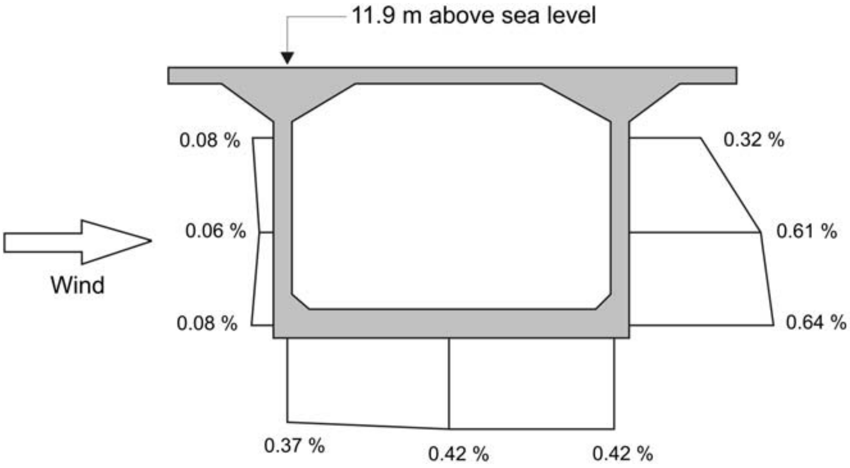


Figure 2.4 - Typical distribution of surface chloride distribution on a bridge in marine environment (Fluge 1997).

For all the corroding bridges along the Norwegian coastline, large amounts of chlorides had penetrated the concrete. A typical pattern for the chloride penetration was that those parts of the bridges that were the most exposed to prevailing winds and salt spray had the least chloride penetration compared to that of the more protected parts (see Figure 2.4). This is due to the rain intermittently washing off the salt from the most exposed surfaces, while on the more protected surfaces, the salt accumulates. Another pattern observed was that older bridges built before approximately 1970 performed better than bridges built later on. This happens generally as a result of lack on knowledge which resulted in use of large quantities of cement (for example) to compensate. With the developments of technologies and cements etc., more confidence has resulted in lower cement contents in order to obtain the same strengths. Also in a number of other countries, extensive investigations have revealed serious problems due to steel corrosion in concrete bridges exposed to marine environment (Nilsson 1991, Stoltzner et al 1994, Beslac et al 1997, Wood et al 1997).

From 1962 to 1968, a comprehensive investigation on the durability and long-term performance of 219 concrete harbour structures along the Norwegian coastline was carried out (Gjørv 1968). Almost all of these structures had steel corrosion to a varying extent or had been repaired due to steel corrosion, and the first visible corrosion damage typically appeared after a service period of 5 to 10 years. Extensive repairs had also been carried out, but new corrosion damage typically appeared within a period of less than 10 years.

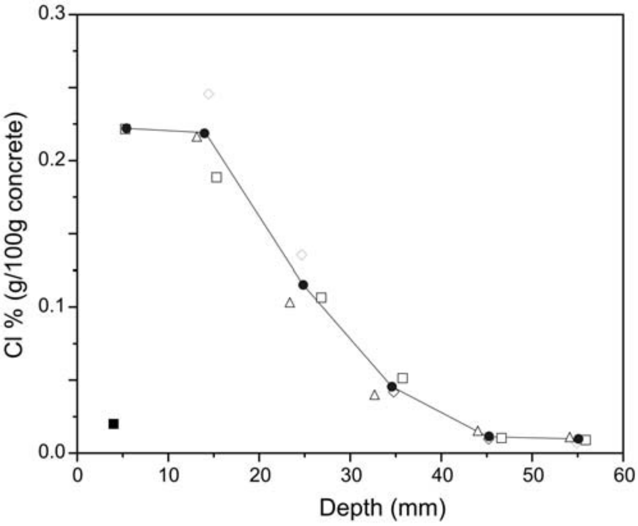


Figure 2.5 - Penetration of chlorides into a concrete harbour structure (1990) after 4.5 years of exposure (Lahus 1999).

During the last 25 to 30 years, both design and execution of concrete work for harbour structures and bridges have improved. However, new and recent field investigations in Norwegian harbours have revealed that a rapid and uncontrolled chloride penetration still represents a serious problem, and the steel corrosion may still occur after a service period of 5 to 10 years (Figures 2.5 and 2.6). For most of these structures, the conventional durability requirements according to current concrete codes both with respect to concrete quality and cover thickness have been fulfilled.

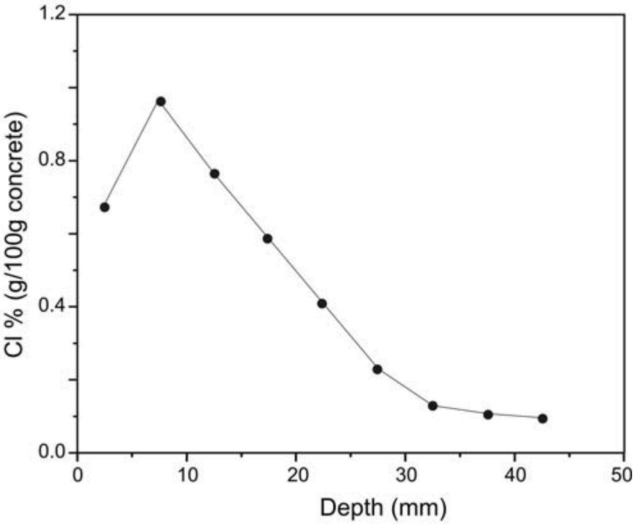


Fig. 2.6 - Penetration of chlorides into a concrete harbour structure (1993) after 8 years of exposure (NTNU 1999).

For construction work in marine environment, recent experience has also shown that a high chloride penetration may take place already during the construction period before the concrete has reached sufficient maturity and density (see Figure 2.7). At early age, most types of concrete are very sensitive to chloride penetration, and this may represent a special problem when the construction work is carried out during rough weather conditions.

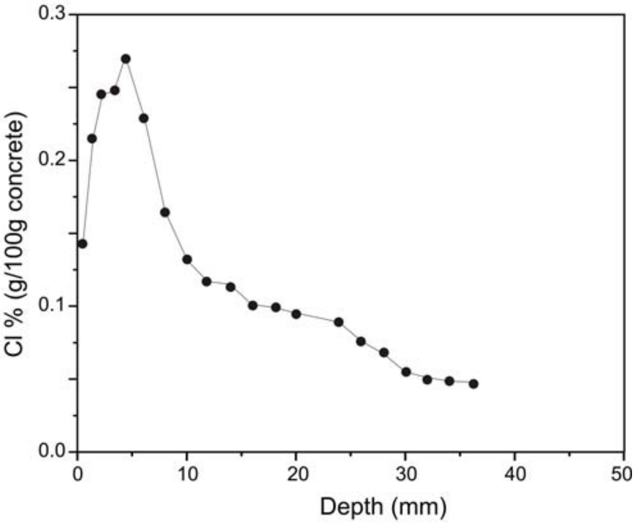


Figure 2. 7 - Penetration of chlorides into a concrete harbour structure (2001) during construction (NTNU 1999).

Also for the high-performance concrete applied to offshore platforms in the North Sea, it appears that chlorides penetrate the concrete, only at a slower rate. Figures 2.8 and 2.9 show the chloride penetration into the Statfjord A Platform (1977) and the Ekofisk Tank (1973) after 8 and 17 years at exposure, respectively, while Fiures 2.7 and 2.8 show the chloride penetration into the Brent B Platform (1975) and Brent C Platform (1976), both after 20 years of exposure. For the Oseberg A Platform (1988), where the obtained concrete thickness was partly less than prescribed, corrosion damage has already occurred and very expensive repairs are under evaluation (Gjørv 2002).

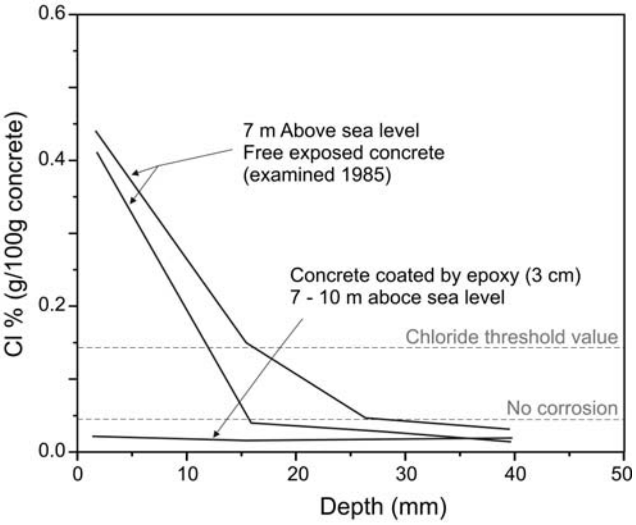


Figure 2.8 - Chloride penetration into the Statfjord A Platform (1977) after 8 years of exposure (Sandvik 1993).

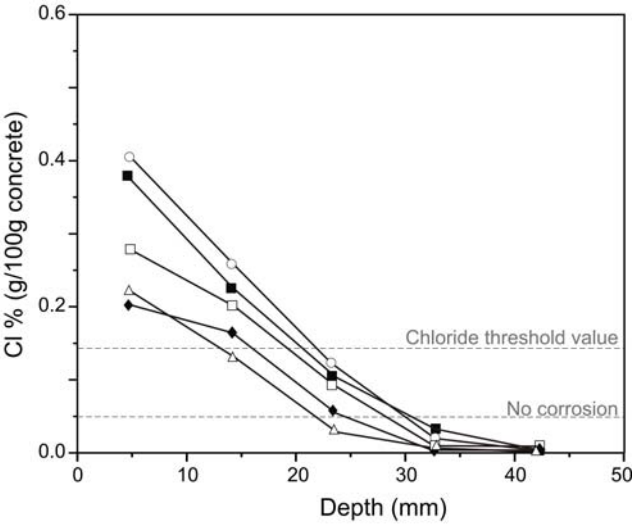


Figure 2.9 - Chloride penetration into the Ekofisk Tank (1973) after 17 years of exposure (Sandvik 1994).

From Figure 2.8 it should be noted that a protection by a solid epoxy coating on the concrete surface in the splash zone had very efficiently prevented the chlorides from penetrating the concrete. For the Heidrun Platform (1995), however, Figure 2.12 demonstrates that the poorer surface coating applied here had not been able to prevent chloride penetration after 2 years of exposure.

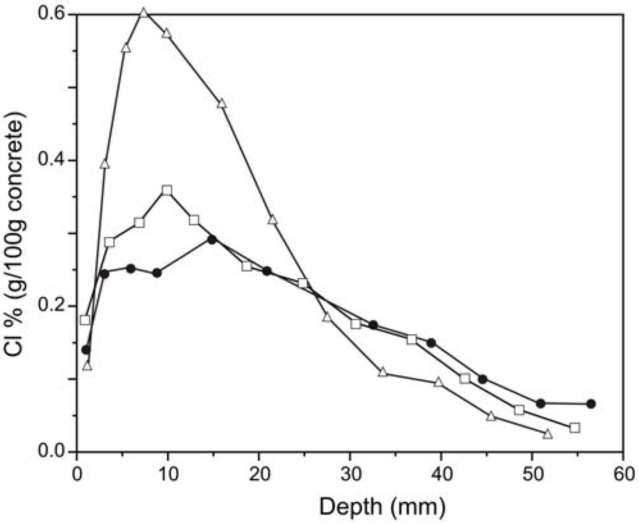


Figure 2.10 - Chloride penetration into the Brent B Platform (1975) after 20 years of exposure, 14.4 m above sea level (NTNU 1999).

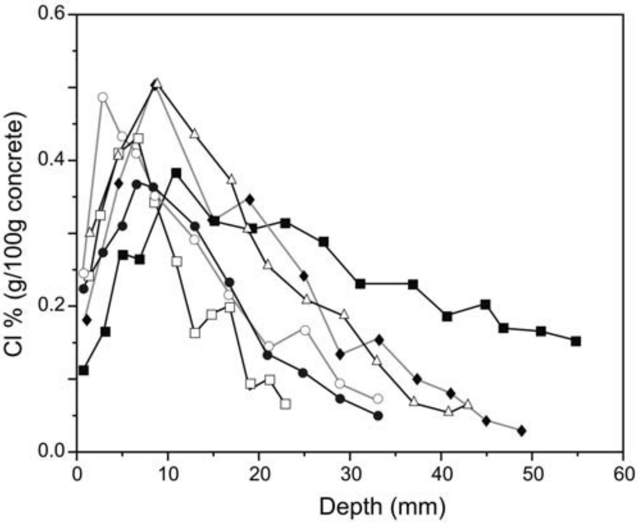


Figure 2.11 - Chloride penetration into the Brent C Platform (1976) after 20 years of exposure, 11 to 18 m below sea level (NTNU 1999).

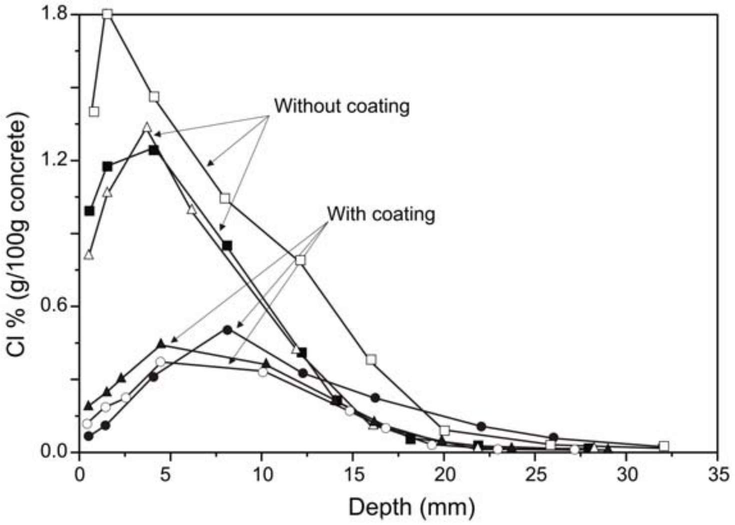


Figure 2.12 - Effect of concrete coating on the chloride penetration into the Heidrun Platform (1995) after 2 years of exposure (NTNU 1999).

2.2.3 Carbonation

Carbonation is a process where CO_2 present in the atmosphere reacts in the presence of moisture with the hydrated cement minerals. CO_2 mainly reacts with $Ca(OH)_2$ to form calcium carbonate, while other cement compounds are carbonated to hemi-carboaluminate ($4CaO \cdot Al_2O_3 \cdot \frac{1}{2}CO_2 \cdot 8H_2O$).

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \tag{2.14}$$

Calcium hydroxide + carbon dioxide → calcite + water

The reaction may occur in three phases. In the first phase, CO_2 diffuses inwards. In the second phase, CO_2 reacts with H_2O molecules, and in the third phase, the resultant carbonic acids react with the alkaline components of the concrete. Carbonation penetrates beyond the exposed surface of concrete very slowly as moisture released by the above reaction diffuses out to preserve hygral equilibrium between the atmosphere and the interior of the concrete. The rate of carbonation is defined by the formula:

$$c = k\sqrt{t} \tag{2.15}$$

where

c - depth of carbonation;

k - constant;

t - time.

The rate at which carbonation develops is dependent on the moisture content of the structure and relative humidity in the vicinity of the concrete structure. Carbonation is also greatly influenced by the CO_2 concentration in the air, type of cement, and w/c ratio of the concrete mix. Humidity must be combined with CO_2, for carbonation to progress.

Consequently, relative air humidity will determine the rate of carbonation. Rapid penetration occurs at a relative humidity ranging between 40 to 75%. While it is less at lower and at very high humidities as shown in the following (Henderson 1988).

Table 2.3 – Rates of carbonation function of RH.

Relative air humidity (%)	Rate of carbonation
Below 30	low
40 to 70	high
Above 75	low

Actual field concrete measurements taken in a variety of concretes in different exposure conditions have shown that, given favourable conditions, concrete may have only carbonated a few millimetres after 30 years. Whereas under adverse conditions, it may have advanced by more than 20 mm in less than 10 years (Henderson 1988, Curie 1987).

Carbonation of Portland cement concrete results in increased strength and reduced permeability of the skin because $CaCO_3$ reduces the voids within the concrete. The calcite layer formed on the surface may therefore protect the concrete from further attack. By contrast, carbonation neutralizes the protection of steel from corrosion provided by the alkaline conditions of hydrated cement paste. The reaction progressively lowers the saturation level of the alkaline solution which initially has a pH value between 12 and 13. When the pH of the pore water decreases below 9, the alkalinity is no longer capable of maintaining the passive oxide film and subsequent access of the electrolyte causes electrochemical processes in the steel to begin. Concrete which has a pH value below 9 is usually categorized as carbonated concrete.

Carbonation of highly permeable concrete may produce further leaching due to the action of the bicarbonate ion on any free lime still present in the concrete, and similarly, carbonation of the chloroaluminate reaction product may release chloride ions previously removed from pore fluid solution by reaction of the calcium aluminate hydrate with sodium chloride. Therefore, although chloride may be removed from solution by the calcium aluminate hydrate, it may subsequently be released if carbonation of this phase occurs. This process exacerbates reinforcement corrosion in heavily carbonated concrete.

A significant side effect of carbonation is the attendant shrinkage that occurs. When concrete is subjected to wetting and drying cycles in air, shrinkage due to carbonation becomes progressively worse and irreversible, and this ultimately leads to surface crazing and cracking of exposed concrete.

2.2.3.1 Experience with carbonation

Carbonation can be prevented or greatly reduced by producing concrete with a denser microstructure. This can be achieved by producing a quality concrete consisting of the right blend of cementitious materials, at the lowest workable w/c ratio. For example a silica fume concrete produces such an effect by a unique double action, (1) by blocking the concrete capillaries, and (2) by converting the by-product calcium hydroxide into additional calcium silicate hydrates. This densification of the concrete results in a dramatic reduction in CO_2 diffusion (TWRL 1980).

A report published by the Building Research Establishment, U.K. (Curie 1987) states that a concrete should have the potential to achieve 65 MPa to render it safe from carbonation attack. One method of attaining this strength is the use of both silica fume and a superplasticizer in the concrete mix.

2.2.4 Freeze-thaw damage

Although the freeze-thaw resistance of concrete has been the subject for extensive research throughout a major part of the last century, this type of deterioration still represents a durability problem in many countries. This is primarily due to the widespread use of deicing salt on concrete pavements and highway bridges (Gjørv 2002).

While pure water in the open freezes at 0°C, in concrete the 'water' is really a solution of various salts so that its freezing point is lower. Moreover, the temperature at which water freezes is lower the smaller the size of the pores full of water. Specifically, the gel pores are too small to permit the formation of ice, and the greater part of freezing takes place in the capillary pores. It is noted that larger voids, arising from incomplete compaction, are usually air-filled and are not appreciably subjected to the initial action of frost. Therefore, frost attack in concrete structures can be divided into two different types of attacks depending on the mechanism:

- Freezing with fresh water - pure frost attack. The attack causes normally internal damages, while the surfaces are undamaged, i.e. deterioration of the physical properties of the bulk concrete. The damages can be avoided if air is entrained in the concrete and a waterproof concrete is used.
- Freezing with salt water or water that is polluted - salt-frost attack. The attack causes external damages, i.e. scaling of the surface. The damage can be avoided if air is entrained and the w/c is kept low.

When water freezes there is an increase in volume of approximately 9 %. As the temperature of concrete drops, freezing occurs gradually so that the still unfrozen water in the capillary pores is subjected to hydraulic pressure by the expanding volume of ice. Such pressure, if not relieved, can result in internal tensile stresses of sufficient magnitude to cause local failure of the concrete. This would occur, for example, in porous, saturated concrete containing no empty voids into which the liquid water can move. On subsequent thawing, the expansion caused by ice is maintained so that there is now new space for additional water which may be subsequently imbibed. On re-freezing further expansion occurs. Thus repeated cycles of freezing and thawing have a cumulative effect, and it is the *repeated* freezing and thawing, rather than a single occurrence of frost, which causes damage. However, the presence of adjacent air voids and empty capillaries allows a relief of hydraulic pressure (caused by the formation of ice) by the flow of water into these spaces; this is the basis of deliberate air entrainment.

There are two other processes which are thought to contribute to the increase of hydraulic pressure of the unfrozen water in the capillaries. Firstly, since there is a thermodynamic imbalance between the gel water and the ice, diffusion of gel water into capillaries leads to a growth in the ice body and thus to an increase of hydraulic pressure. Secondly, the hydraulic pressure is increased by the pressure of *osmosis* brought about by local increases in solute concentration due to the removal of frozen (pure) water from the original solution.

The extent of damage caused by repeated cycles of freezing and thawing varies from *surface scaling* to complete disintegration as layers of ice are formed, starting at the exposed surface of the concrete and progressing through its depth. Concrete which remains wet for long periods is more vulnerable to frost than any other concrete. Salts used in deicing make concrete vulnerable because, as they become absorbed by the top surface of the concrete, the resulting high osmotic pressures force the water towards the coldest zone where freezing takes place.

The main factors in determining the resistance of concrete to freezing and thawing are the degree of saturation and the pore structure of the cement paste; other factors are the strength, elasticity and creep of concrete. Below some critical value of saturation (80 to 90 per cent), concrete is highly resistant to frost, while dry concrete is, of course, totally unaffected.

The risk for frost damage in a concrete can be determined from the critical air content, which follows from the properties of the air-pore system and if the freezing occurs with fresh water or in the presence of salt. However, usually the risk for frost damage is determined from the degree of saturation, S_{CAP}, see (2.16), Lindmark (1998).

$$S_{CAP} = \frac{V_W}{V_{PORE}} = \frac{V_W}{V_W + \alpha} \tag{2.16}$$

where:

V_W - the volume of water available in the pore system of the concrete, m^3;

V_{PORE} - the total pore volume in the concrete, m^3;

α - the volume of the air contained in the pore system, m^3;

It is possible to determine a critical degree of saturation, S_{CR}, with knowledge of the critical air content, Acr, and the total porosity in the concrete, including air pores, P

$$S_{CR} = \frac{P - A_{CR}}{P} = 1 - \frac{A_{CR}}{P} \quad \text{where} \quad P = \frac{V_{TOTAL}}{V_{PORE}} \tag{2.17}$$

To assess the risk of frost damage the critical degree of saturation is compared with the degree of saturation in the concrete. If $S_{CAP} > S_{CR}$ there is large risk of extensive frost damage even after few freeze/thaw-cycles and if $S_{CAP} < S_{CR}$ the risk of frost damage is low even after a large number of freeze/thaw-cycles.

The risk of frost damage in concrete is influenced by the following factors:

Air content. The risk of frost damage of a concrete can be determined by studying the air-pore system of the concrete. It is possible to theoretically determine critical air content, with knowledge about the amount of cement paste, the diameter of the air pores and the critical spacing factor. The critical spacing factor is the maximum distance for water transport between the air-filled pores. For pure frost attack the critical air content has been found to be some 2-3 % and with salt-frost the critical air content has been found to be over 4.5 %. The natural air content in a concrete is some 2 %. To increase the air content in the concrete Air Entraining Agents are used. The effect of the AEA depends on the properties of the concrete, the mixing procedure and transports.

Degree of capillary saturation. An alternative way to determine the risk for frost damage in a concrete is to study the degree of capillary saturation, S_{CAP}. The degree of capillary saturation is then compared with a critical degree of capillary saturation, S_{CR}, to assess the risk for frost damage to occur. When $S_{CAP} > S_{CR}$ the risk of frost damage becomes substantial while if $S_{CAP} < S_{CR}$ the concrete is resistant against frost action.

Environmental actions. The formation of ice within the concrete is influenced by the temperature conditions, where the minimum temperature and the number of passages of 0°C are the most important factors. The cooling rate seems to have a minor influence on the occurrence of frost damage. Furthermore the moisture conditions, both inside and outside the concrete, have an influence on the frost damage.

Presence of salt. In the presence of salt, from seawater or deicing salt, the damages due to frost has shown to increase. Principally freezing with salt present reduces the critical spacing factor and thus the critical degree of saturation is decreased. The worst damages occurred when the concentration of NaCl was 2-4 %.

Concrete properties. Generally concretes with high permeability, i.e. high w/c, have shown low resistance against frost damage. By decreasing the w/c, and thus the permeability, the resistance against frost damage has been improved. Furthermore a concrete with low w/c contains less freezable water, due to self-desiccation effects.

The adequacy of resistance of a given concrete to frost attack can be determined by *freezing and thawing tests.* Two methods are prescribed by ASTM C 666-92. In both of these, rapid freezing is applied, but in one freezing and thawing take place in water, while in the other freezing takes place in air and thawing in water. These conditions are meant to duplicate possible practical conditions of exposure. BS 5075: Part 2:1982 also prescribes freezing in water. Frost damage is assessed after a number of cycles of freezing and thawing by measuring the loss in mass of the specimen, the increase in its length, decrease in strength or decrease in the dynamic modulus of elasticity, the latter being the most common. With the ASTM methods, freezing and thawing are continued for 300 cycles or until the dynamic modulus is reduced to 60 per cent of its original value, whichever occurs first. The *durability factor,* D_f is then given by

$$D_f = \frac{n}{3}\left[\frac{E_{dn}}{E_{d0}}\right] \tag{2.18}$$

where

n - number of cycles at the end of test,

E_{dn} - dynamic modulus at the end of test, and

Ed_0 - dynamic modulus at the start of test.

The value of D_f is of interest primarily in a comparison of different concretes, preferably when only one variable (e.g. the aggregate) is changed. Generally, a value smaller than 40 means that the concrete is probably unsatisfactory, values between 40 and 60 are regarded as doubtful, while values over 60 indicate that the concrete is probably satisfactory.

The test conditions of ASTM C 666-92 are more severe than those occurring in practice since the prescribed heating and cooling cycle is between 4.4 and -17.8 °C (40 and 0 °F) at a rate of cooling of up to 14 °C per hour (26 °F per hour). In most parts of the world, a rate of 3°C per hour (5°F per hour) is rarely exceeded. However, in another test method, ASTMC 671-86 prescribes this slower rate of freezing with one cycle every two weeks. The test continues until the specimen has undergone either the desired number of cycles or the number of cycles after which the critical dilation has occurred. The critical dilation is defined as a dilation which is at least twice that in the preceding cycle.

2.2.4.1 Experience with freeze-thaw

Although not many field studies on the freeze-thaw resistance of concrete structures have been reported in the literature, a Swedish report of 1995 (Petersson 1995) which included field tests both in marine and highway environment, states that the highway environment was by far the most aggressive to the concrete. For many concrete pavements and highway bridges, the deicing salt may cause a rapid degradation of the concrete due to corrosion of the reinforcement.

A general problem both for specifying and assessing the frost resistance of concrete is the lack of correlation between existing test methods and field performance. Also, the different existing test methods appear to give very different and conflicting results.

Another problem is the production of a frost resistant concrete which has a good and stable air void system during transportation and handling of the fresh concrete. Extensive investigations of existing concrete structures with intentionally entrained air have revealed that no air was present or was inadequate by current standards (Klieger 1980, Manning 1989, Gjørv 1987). Even for a normal-strength concrete without the use of any superplasticizer, the production of a good and stable air void system may represent a problem, but in the presence of a superplasticizer, however, this may be an even more severe problem (Okkenhaug et al 1992). Therefore, much attention has been given in recent years to find out whether a frost resistant high-performance concrete can be produced without any air entrainment.

Both Okada et al. (1981), Foy et al. (1988) and Gagne et al. (1990) have reported a good frost resistance of non-air-entrained concrete with w/c ratios in the range of 0.25 to 0.35. Malhotra et al. (1987) who also tested a number of concretes with different types of cement and w/c ratios in the same range, concluded, however, that air entrainment was necessary for these concretes to be frost resistant.

Also for salt scaling, there are some conflicting results in the literature. Petersson (1984) reported that deterioration of high-strength concrete was small for the first 50 to 100 freeze–thaw cycles, while in the following 10 to 20 cycles, a very rapid deterioration with total destruction took place. However, both Foy et al. (1988), Gagne et al. (1990) and Hammer and Sellevold (1990) have shown that it is possible to produce high-strength concrete without any air entrainment which is resistant to salt scaling. These investigations included concretes with w/c ratios of up to 0.37 and testing of up to 150 freeze–thaw cycles.

2.2.5 Alkali-aggregate reactions

Although the first durability problems caused by alkali-aggregate reaction (AAR) were observed on several Californian concrete structures and reported by Thomas Stanton already in 1940, it has taken a long time to recognize AAR being a general durability problem. Already in 1947 Mielenz et al. published a ”black list” of alkali-reactive aggregates and minerals. This form of degradation occurs when alkalis released from the hydrating cement react with aggregate containing reactive constituents. Gilliot (1975) suggested that AAR should be subdivided into the following three groups of reaction: alkali-silica reactions with siliceous aggregates (some cherts, opal, and siliceous limestones), alkali-carbonate reactions with carbonate aggregates (some argillaceous dolomites) and alkali-silicate reactions (same as alkali-silica reaction except that the reactive constituent in the aggregate is silica present in the form of phyllosilicates).

Later on, several international publications have shown that even more stable silicious rocks can be alkali reactive, e.g. granite, quartzite, shist and sandstone. A list of reported alkali-reactive aggregates were published by Coull (1981) and by Dolar-Mantuani (1983). Since 1974, extensive international experience with AAR in concrete has been published in a large number of international conferences.

The AAR is influenced by a number of factors, DuraCrete (1998a):

- Constituent materials: This involves the properties of binders (equivalent alkali content, relative proportions of sodium and potassium and fineness), aggregates (reactivity, proportions and grading of the fine and coarse fractions, presence of salt impurities, alkali contributions from certain aggregates and porosity of coarse aggregates) and admixtures (AEAs, plasticizers and superplasticizers).
- Mix proportions: This involves the binder content, water content, w/c and the aggregate/binder ratio in the concrete.
- Exposure environment: This factor involves the availability of moisture and external chlorides, the temperature and alkali concentration effects.
- Structural loading: This factor consists of the magnitude and the nature of the structural loading.
- External and internal restraints: This factor is influence by the amount of reinforcement and the structural configuration.

It has been found that the occurrence of AAR to a large extent is caused by a choice of not suitable aggregates and advert moisture conditions. Procedures that aim to minimise the expansion due to AAR are usually based on fault tree diagrams, where all combinations of the following factors are analysed, DuraCrete (1998a):

- Experience with existing aggregates (types, proportions, combinations etc),
- Petrographic examination,
- Use of low alkali cements,
- Limitation of total alkali content and
- Accelerated mortar bar and concrete prism testing.

2.2.5.1 Alkali-Silica Reaction

Certain siliceous minerals from the aggregates react with the alkalis (Na_2O, K_2O) in cement forming a gel; an alkali-silica complex of variable composition. The clear alkali-silica complex may react with calcium ions originating from calcium hydroxide or other cement hydrates to form a white, opaque calcium silica or alkali-calcium-silica complex (Powers 1955). The product of the alkali-silica reaction absorbs water and increases in volume, and in this process changes from a hard solid to a softer gel and finally to a sol. As water is absorbed, the swelling reaction product (which usually appears as a while rim around the reactive aggregate) exerts a disruptive force on the surrounding cement paste and cracks it. As more water is absorbed, the crack is propagated into the mortar and becomes wider. The forces produced by many such swelling particles may interact and accentuate the propagation and widening of a crack (Vivian 1978). Cracking usually takes many years and is often preceded by pop outs and spalling on the concrete surface.

For this reason, it is believed that it is the swelling of the hard aggregate particles that is most harmful to concrete. The speed with which the reaction occurs is controlled by the size of the siliceous particles: fine particles (20 to 30 μm) lead to expansion within four to eight weeks while larger ones do so only after some years. It is generally the very late occurrence of damage due to the alkali-aggregate reaction, often after more than five years, which is a source of worry and uncertainty.

Other factors influencing the progress of the alkali-aggregate reaction are the porosity of the aggregate, the quantity of the alkalis in the cement, the availability of water in the paste and the permeability of the cement paste. The reaction takes place mainly in the exterior of the concrete under permanently wet conditions, or when there is alternating wetting and drying, and at higher temperatures (in the range of 10 to 38 °C); consequently, avoidance of these environments is recommended. Although it is known that certain types of aggregate tend to be reactive, there is no simple way of determining whether a given aggregate will cause excessive expansion due to reaction with the alkalis in cement.

2.2.5.2 Alkali-Carbonate Reaction

Carbonate reactivity involves carbonate rocks in a complex series of reactions the nature of which depends on the local aggregate, cement, and microenvironment. As in the previous reaction, the alkalis released from cement pastes or other sources react with carbonate aggregates. The presence of clay minerals in the aggregate and the crystalline texture of the rocks have been reported to influence the rate of these reactions (French 1974, Ewenson 1960). A typical feature of these reactions is the development of reaction zones up to *2* mm wide around the aggregate particles (French 1974). Cracking develops extensively within these rims both parallel to the interface and radial to it. The radial cracks gradually extend, linking together through the paste. Development of the crack network coupled with the loss of adhesion between the aggregate and paste will eventually cause a concrete to deteriorate.

Several distinct types of alkali-carbonate reactions are known but not all of them are expansive. The most common reaction, however, involves the removal of magnesium from the rim zone of dolomitic particles by the action of alkali to form calcium-alkali carbonates and magnesium hydroxide. The volume change that accompanies de-dolominization is thought to result from the increased porosity which promotes the entry of water and the subsequent swelling of the clay incorporated in the aggregate (Ewenson 1960).

2.2.5.3 Experience with alkali-aggregate reactions

In many countries, maintenance costs due to AAR make up a large proportion of the total maintenance costs of concrete structures. Thus, in Western Europe it is estimated that the damage due to AAR makes up approximately 10% of the total maintenance costs of all concrete structures, which annually amounts to about 5 billion € (Cost 521 1997). It is difficult to estimate the costs of the reduced service life of all these concrete structures, but it is probably significantly higher than that of the maintenance costs.

For concrete structures suffering from AAR, the first sign of cracking and deterioration may take a long time. For the more fast reacting types of aggregates such as porous flint, it may only take 2 to 5 years, but for the more slow reacting types of aggregate such as sandstone, it may take as much as 10 to 20 years. The rate of deterioration is determined both by type of aggregate and type of environment.

Inspections of concrete structures suffering from AAR are very expensive, as they require use of both expensive equipment and skilled personnel. Testing of aggregates for AAR may also be very difficult, because considerable experience is needed both for selection of proper test method for the given aggregate and for interpretation of the test results. Current knowledge has clearly shown that existing test methods for alkali reactivity in many cases are not suitable for certain types of aggregate. Thus, many examples are given in the literature demonstrating that two of the most world-widely used test methods such as the mortar bar test ASTM C 227 and the chemical method ASTM C 289 are not able to detect the slow/late expansive types of aggregate such as sandstone, greywacke and rhyolite. Both the selection of a reliable test method and a proper interpretation of the test results may have large economical consequences.
In general, the results of accelerated testing are an inferior substitute for existing experience on

the long-term behaviour of an aggregate in a relevant type of concrete and relevant type of environment. In order to provide experience with local aggregates, therefore, many countries have established national field exposure sites.

We should note that neither slag nor fly ash is assumed to contribute reactive alkalis to the concrete, although those materials have fairly high levels of alkalis. However, most of these alkalis are probably contained in the glassy structures of the slag or fuel ash and take no part in the reaction with aggregate. Moreover, the silica in fuel ash, paradoxically, attenuates the harmful effects of the alkali-silica reaction. The reaction still takes place, but the finely divided siliceous material in PFA forms, preferentially, an innocuous product. In other words, there is a minimum content of reactive silica in the concrete above which little damage occurs.

2.2.6 Sulphate attack

Sulphate attack is the most common form of salt attack on concrete. The concrete is exposed to sulphates usually due to environmental actions, but they can also be present in the binder or aggregates. Sulphates may follow moisture into the concrete and be concentrated locally. Damages caused by sulphate attack can for example be strength loss, expansion and spalling. Solid salts do not attack concrete but when present in solutions they can react with hydrated cement paste. The most common solutions are sulphates of sodium (Na), potassium (K), magnesium (Mg) and calcium (Ca). The natural origins of sulphates are in ground water but they may also come from fertilizers and industrial activities.

Concrete attacked by sulphates has a characteristic whitish appearance, damage usually starting at the edges and corners and followed by cracking and spalling of the concrete. The reason for this appearance is that the essence of sulphate attack is the formation of calcium sulphate (gypsum) and calcium sulphoaluminate (ettringite), both products occupying a greater volume than the compounds which they replace so that expansion and disruption of hardened concrete take place.

The effects of sulphate attacks in concrete can be reduced in two different ways, Neville (1997):

- Reduction of the C_3A in the binder. This is done by the usage of sulphate resistant cements in the concrete.
- Reduction of the quantities of $Ca(OH)_2$ in the cement paste. This is done by the usage of blended cements, containing GGBS or pozzolans, in the concrete. The effects of blended cements are: (1) Pozzolans react with the $Ca(OH)_2$ so that $Ca(OH)_2$ is not longer available for reaction and (2) Blended cements contain less $Ca(OH)_2$ than PC.

The consequences of sulphate attack include not only expansion, which disrupt the concrete, but also loss of strength of the concrete due to loss of cohesion in the hydrated cement paste and between the particles in the aggregate.

Gypsum is added to the cement clinker in order to prevent flash set by the hydration of the tricalcium aluminate (C_3A). Gypsum quickly reacts with C_3A to produce ettringite which is harmless because, at this stage, the concrete is still in a semi-plastic state so that expansion can be accommodated.

A similar reaction takes place when hardened concrete is exposed to sulphates from external sources. A typical sulphate solution is the groundwater of some clays which contain sodium, calcium or magnesium sulphates. The sulphates react with both $Ca(OH)_2$ and the hydrated C_3A to form gypsum and ettringite, respectively. Magnesium sulphate has a more damaging effect than other sulphates because it leads to the decomposition of the hydrated calcium silicates as well as of $Ca(OH)_2$ and of hydrated C_3A; hydrated magnesium silicate is eventually formed and it has no binding properties.

The extent of sulphate attack depends on its concentration and on the permeability of the concrete, i.e. on the ease with which sulphate can travel through the pore system. If the concrete is very permeable, so that water can percolate right through its thickness, $Ca(OH)_2$ will be leached out. Evaporation at the 'far' surface of the concrete leaves behind deposits of calcium carbonate, formed by the reaction of $Ca(OH)_2$ with carbon dioxide; this deposit, of whitish appearance, is known as *efflorescence.* Efflorescence is generally not harmful. However, extensive leaching of $Ca(OH)_2$ will increase porosity so that concrete becomes progressively weaker and more prone to chemical attack. Crystallization of other salts also causes efflorescence.

Salts attack concrete only when present in solution, and not in solid form. The strength of the solution is expressed as concentration, for instance, as the number of parts by mass of sulphur trioxide (SO_3) per million parts of water (ppm). A concentration of 1000 ppm is considered to be moderately severe and 2000 ppm very severe, especially if magnesium sulphate is the predominant constituent. Since it is C_3A that is attacked by sulphates, the vulnerability of concrete to sulphate attack can be reduced by the use of cement low in C_3A, i.e. sulphate-resisting (CEM V) cement. Improved resistance is obtained also by the use of Portland blast-furnace (CEM III) cement and of Portland-pozzolan (CEM II) cement; the exact mechanism by which these cements are beneficial is uncertain. However, it must be emphasized that the type of cement is of secondary importance, or even of none, if the concrete is dense and has a low permeability, i.e. a low water/cement ratio. The water cement ratio is the vital factor but high cement content facilitates full compaction at low water/cement ratios.

2.2.6.1 Experience with sulphate attack

A number of conditions must be fulfilled for delayed ettringite formation to occur, Taylor (1997):

Exposure environment. The temperature inside the concrete must be high enough, at least 65 to 70 °C. During its life the concrete structure must be in an environment, which is saturated or nearly so, either intermittently or continuously. If the concrete has been subjected to an elevated curing temperature and exposed to water or saturated air ettringite is formed a couple of years after casting.

Binder properties. A binder with high contents of SO_3, alkalis and MgO improves the expansion due to delayed ettringite formation. The content of C_3A and in some degree C_4AF in the binder is important. However there are no single components that are predominant over the others.

Sulphate attacks are mainly influenced by two factors, Neville (1997):

- The surrounding environment. The concentration of sulphates at the surface of the concrete has a large influence on the sulphate attack. A classification of the severity of different environments, depending on the concentration of sulphate and if the sulphates appear in soil or dissolved in water, can be made. For example if sulphates in sea water are considered they vary in the following way: Baltic sea - 1.25 ppt, Atlantic ocean - 2.54 ppt, North sea - 2.78 ppt, Mediterranean - 3.06 and Persian Gulf-2.72 ppt.
- The concrete - Chemical composition and permeability. To avoid sulphate attack the binder used in the concrete should have low content of C_3A (sulphate resistant cement) or low content of $Ca(OH)_2$ (by using pozzolanic additives). By using a sulphate resistant cement reactions between C_3A and calcium sulphates can be avoided. However, under severe conditions also other forms of sulphates may penetrate into the concrete, which means that this is not always enough to protect the concrete against sulphate attacks. A low permeability of the concrete, achieved with low w/c or additives, slows down the rate of penetration of sulphates and hence the consequences of sulphate attack are reduced.

2.2.7 Attack by sea water

Sea water contains sulphates and could be expected to attack concrete in a similar manner to that described in the previous section but, because chlorides are also present, sea-water attack does not generally cause expansion of the concrete. This is because the expansive products (ettringite and gypsum) formed due to the reaction of C_3A with sulphates in sea water, are not accompanied by the extent of swelling seen in pure solutions of sodium and magnesium sulphate, because ettringite and gypsum are more soluble in solutions containing chlorides such as sea water (Verbeck 1975, Mehta 1975).

In sea water containing a high degree of dissolved carbon dioxide, the degradation process can gradually progress inwards (Biczock 1964). The calcium and hydroxyl ions transported to the surface react with carbon dioxide to form aragonite. However, due to the high carbon dioxide levels, aragonite is converted to calcium bicarbonate which is leached away.

$$CaCO_3 + CO_2 + H_2O \rightarrow Ca(HCO_3)_2 \qquad (2.19)$$

Magnesium ions now cause the precipitation of magnesium hydroxide in the new surface, while chloride and sulphate ions move in and form chloroaluminates and sulfoaluminates. When the hydroxyl ions are depleted at an advanced state of deterioration, magnesium ion penetration into the paste can occur, thus exposing the calcium silicate hydrate to magnesium ion attack (Gutt 1977, Regourd 1978).

In the interior layers, the formation of gypsum and ettringite is responsible for removal of sulphate ions from sea water. The chlorides can now continue to penetrate into the interior. The observed penetration by chloride ions even at low w/c ratios of 0.3 is thus explained by the fact that, except for chloride ions, the components of sea water (such as carbon dioxide and magnesium sulphate) are readily removed in the outer zones as a result of the chemical reactions with the cement pastes (Gutt 1977).

Although C_3A and C_4AH_{19} are capable of removing chlorides as chloroaluminate hydrate, the compound is not stable in a sulphate environment. Thus, in sea water, chloroaluminate enlivens to ettringite releasing chloride ions. The C_3A content also plays a role in the minimum Cl^-/OH^- ratio needed to initiate corrosion. The threshold value for corrosion initiation (usually 0.63) is significantly influenced by cements with high C_3A content because of chloride removal. Such cements may then tolerate a higher than normal Cl^-/OH^- ratio (Mehta 1980).

The pH of the concrete is the controlling factor in the corrosion process. Of the various salts present in sea water, magnesium chloride is the most destructive because it replaces OH^- ions, which are then available to reduce the pH to threshold levels which initiate corrosion. In consequence, there is no disruption but only a very slow increase in porosity and, hence a decrease in strength.

Concrete between tide marks, subjected to alternating wetting and drying, is severely attacked, while permanently immersed concrete is attacked least. However, the attack by sea water is slowed down by the blocking of pores in the concrete due to the deposition of magnesium hydroxide which is formed, together with gypsum, by the reaction of magnesium sulphate with $Ca(OH)_2$.

In some cases, the action of sea water on concrete is accompanied by the destructive action of frost, of wave impact and of abrasion. Additional damage can be caused by rupture of concrete surrounding reinforcement which has corroded due to electro-chemical action set up by absorption of salts by the concrete.

In-place concrete density, cement type and content play a pivotal role in the resistance of concrete to sea water. Concrete made with calcium aluminate, supersulfated cements, and also those containing supplementary cementing materials, resists sea water fairly well. Such improved resistance as compared to 100% Type I ordinary Portland cement (OPC) stems from reduced free lime content in such concretes. Regardless of the cement type used, deteriorated concrete generally contains the same constituents - calcium carbonate as calcite or aragonite, brucite and magnesium silicate hydrate (Mehta 1980, Regourd 1978, Kalousek 1970).

2.2.7.1 Experience with sea water attack

Sea-water attack can be prevented by the same measures which are used to prevent sulphate attack but, here, the type of cement is of little importance compared to the requirement of low permeability. In reinforced concrete, adequate cover to reinforcement is essential at least 50 to 75 mm. A cement content of 350 kg/m^3 above the water mark and 300 kg/m^3 below it, and a w/c ratio of not more than 0.40 to 0.45 are recommended. A well-compacted concrete and good workmanship, especially in the construction joints, are of vital importance.

2.2.8 Acid attack

Acid attack generally occurs where the calcium hydroxide is attacked vigorously, although all the Portland cement compounds are susceptible to degradation. Acidic solutions both mineral (such as sulphuric, hydrochloric, nitric, and phosphoric acids) and organic (such as lactic, acetic, formic, tannic, and other acids produced in decomposing silage) are about the most aggressive agents to concrete. Depending on the type of acid, the attack can be mainly an acid attack, or a combination of acid followed by a salt attack.

Owing to the highly basic character of Portland cement, an acid cannot penetrate dense concrete without being neutralized as it travels inwards. Therefore, it cannot cause deterioration in the interior of the specimen without the cement paste on the outer portion being completely destroyed. The rate of penetration is thus inversely proportional to the quantity of acid neutralizing material, such as the calcium hydroxide, C-S-H gel, and limestone aggregates.

No Portland cement is resistant to attack by acids. In damp conditions, sulphur dioxide (SO_2) and carbon dioxide (CO_2), as well as some other fumes present in the atmosphere, form acids which attack concrete by dissolving and removing a part, of the hydrated cement paste and leave a soft and very weak mass. This form of attack is encountered in various industrial conditions, such as chimneys, and in some agricultural conditions, such as floors of dairies.

In practice, the degree of attack increases as acidity increases; attack occurs at values of pH below about 6.5, a pH of less than 4.5 leading to severe attack. The rate of attack also depends on the ability of hydrogen ions to be diffused through the cement gel (C-S-H) after $Ca(OH)_2$ has been dissolved and leached out.

Concrete is also attacked by water containing free carbon dioxide in concentrations of at least 15 to 60 ppm; such acidic waters are moorland water and flowing pure water formed by melting ice or by condensation. Peaty water with carbon dioxide levels in excess of 60 ppm is particularly aggressive - it can have a pH as low as 4.4.

Although alkaline in nature, domestic sewage causes deterioration of sewers, especially at fairly high temperatures, when sulphur compounds in the sewage are reduced by anaerobic bacteria to H_2S. This is not a destructive agent in itself, but it is dissolved in moisture films on the exposed surface of the concrete and undergoes oxidation by anaerobic bacteria, finally producing sulphuric acid. The attack occurs, therefore, above the level of flow of the sewage. The cement is gradually dissolved and progressive deterioration of concrete takes place.

2.2.8.1 Experience with acid attack

Attack of $Ca(OH)_2$ can be prevented or reduced by fixing it. This is achieved by treatment with diluted water glass (sodium silicate) to form calcium silicates in the pores. Surface treatment with coal-tar pitch, rubber or bituminous paints, epoxy resins, and other agents has also been used successfully. The degree of protection achieved by the different treatments varies, but in all cases it is essential that the protective coat adheres well to the concrete and remains undamaged by mechanical agencies, so that access for inspection and renewal of the coating is generally necessary.

The leaching-rate is increased by acid attack. Acid attack on concrete may have two different mechanisms: dry and wet deposition. It is known that the CSH gel is decomposed and silica gel is deposited when the pH is below 10, Okochi et al (2000).

The rate of dissolution of calcium hydrates is dependant on the pH of the exposure solution, the rate at which the exposure solution is applied and the temperature. Furthermore the rate of dissolution is dependent if the concrete is carbonated or not - the solubility of $CaCO_3$ is about 1/100 of the solubility of $Ca(OH)_2$. In Okocchi et al (2000), a relationship is established between the rainfall intensity and the cumulative amount of dissolved calcium.

$$CA = a \cdot I^b \tag{2.20}$$

where:

CA - the cumulative amount of dissolved calcium. [mol/(m^2s)]

I - the rainfall intensity. [mm/h]

a - constant, determined from linear regression analysis (a=0.457).

b - constant, determined from linear regression analysis (b=0.556).

The cumulative amount of dissolved calcium increased when the rainfall intensity decreased. Thus with the same amount of rainfall the dissolution of calcium hydrates in concrete increases with decrease in the rainfall intensity, because the exposure time between rainwater and the concrete is longer.

2.3 Chloride Penetration

Corrosion due to chloride penetration is the main cause of concrete degradation in marine environment. It is therefore important that this phenomenon be understood.

The transport and distribution of chlorides in a concrete structure is very much a function of the environmental conditions, mainly the concentration and duration of the solutions in contact with the concrete surface. The conditions are quite different in different exposure situations.

Below the level of sea water, the concentration of chlorides is more or less constant with time. The penetration is a pure diffusion process at constant boundary conditions. At early ages, however, a negative pressure (suction) is produced as the hydration of cement takes place in the interior parts of the concrete (Gonzalez, Jalali 2004).

Most other cases of exposure situations have boundary conditions that change according to a more or less rapid and complicated relationship with time. In the splash zone above sea water level, salt water is sucked into the concrete surface. Rain water washes the surface free from chlorides and may remove some of them. Evaporation increases the concentration. Chlorides move inwards and outwards due to moisture flow and ion diffusion.

The conditions are different at different heights from the average sea water level. Maximum chloride content may be found at a height where salt water is frequently supplied to the surface but where the surface intermittently dries out. Closer to the sea water level the concrete is almost continuously saturated by water and no additional salt water can be "added". In a tidal zone the conditions are somewhat similar to a splash zone. The time of wetness and the intermittent drying time, however, are different.

The distribution of chlorides, i.e. concentrations at different depths, is a time-dependent function of the environmental conditions, the design of the structure and the material properties. The mechanisms of chloride transport and binding involved are complex and usually combined in a complex way. Chloride binding in concrete is important since the binding process removes chloride ions from the concrete pore solution into the solid hydrated binder phase. The processes are not always understood and still not easy to quantify.

2.3.1 Chloride ingress into concrete.

It is known that the degradation process is strongly influenced by the environmental conditions, which in turn influence the transportation processes. The rates of most deterioration processes that affect concrete structures are controlled by the transport of particular aggressive species through the concrete. It is, therefore, necessary to understand the various transport mechanisms that may take place in the deterioration of a concrete structure.

There are a number of different transport processes that are listed below (Glanville 1997).

- Pressure-induced water flow of water due to the application of a hydrostatic head. Characterized by a water permeability coefficient, this transport process has been researched more thoroughly than any other. However, it is rare for this to be the predominant transport process; exceptions include water-retaining structures and deeply submerged concrete.
- Water absorption is the uptake of water resulting from capillary forces, characterized by a sorptivity coefficient. In environments where significant concrete drying is possible, water absorption may lead to very rapid penetration of species dissolved in the water. For example, concrete in the tidal zone of an offshore structure, or bridge decks subjected to regular applications of deicing salts, may suffer from rapid chloride ion ingress due to water absorption.

- Water vapour diffusion is the diffusion of water as a vapour, characterized by a water vapour diffusivity coefficient. This is the process by which concrete dries and it controls the moisture distribution in a concrete element after a period of drying. In turn, moisture distribution influences properties such as water absorption during wet and dry cycles, electrical resistivity, gas permeability and gas diffusivity.
- Wick action is a combination of water absorption and water vapour diffusion, characterized by sorptivity and water vapour diffusivity coefficients respectively. Wick action is the transport of water from the wetted face of a concrete element to a drying face. Species dissolved in the water are transported to the zone of the element where drying occurs and may result in salt crystallization.
- Ion diffusion is the movement of ions as a result of a concentration gradient, characterized by an ion diffusion coefficient. Ion diffusion is only significant where the concrete is nearly or completely water saturated. The ions that are usually of most interest are chlorides and sulphates. Compared with most other transport processes, ion diffusion is very slow, but is often rapid enough to cause deterioration well within the design life of a concrete structure.
- Gas diffusion is the movement of gas molecules as a result of a concentration gradient, characterized by a gas diffusion coefficient. Diffusion of carbon dioxide into concrete results in carbonation. In some situations, diffusion of oxygen controls the rate of corrosion of steel in concrete.
- Pressure-induced gas flow is the flow of gas due to a pressure gradient, characterized by a gas permeability coefficient. Gas permeability is relatively easy to measure, but is not relevant to the behaviour of the vast majority of concrete structures. It is relevant to the behaviour of certain structures used in the nuclear industry.

2.3.2 Quantifying chloride contents

Chlorides in concrete can be present as a result of various sources. They can be cast into the concrete or they can diffuse in from the outside. Chlorides cast into concrete can be due to the deliberate addition of chloride set accelerators (calcium chloride, $CaCl_2$, was widely used until the mid-1970s); the use of sea water in the mix or of contaminated aggregates (usually sea dredged aggregates which were unwashed or inadequately washed). Chlorides from the environment can diffuse into concrete as a result of the sea salt spray and direct sea water wetting; the deicing salts or the use of chemicals (structures used for salt storage, brine tanks, aquaria, etc.)

In concrete, chloride may be present in any of the following forms:

- *Acid soluble chloride* which is equal to the total amount of chloride present in the concrete or that is soluble in nitric acid;

- *Bound chloride* which is the sum of chemically bound chloride with hydration products of the cement, such as the C_3A (tricalcium aluminate) or C_4AF (tetracalcium aluminoferrite) phases, and loosely bound chloride with C-S-H gel;
- *Free or water-soluble chloride* which is the concentration of free chloride ions (Cl^-) within the pore solution of concrete, and is extractable in water under defined conditions.

The following equation shows the relationship between the chloride concentrations present (HETEK-53 1996):

$$c_{TOTAL} = c_{FREE} + c_{BOUND} \tag{2.21}$$

where c is the concentration in [kg/m^3], [moles/volume] or [kg/kg]. The free chlorides are usually given per volume of solute, i.e. [kg/m^3 solution]. The bound, and total amount of chlorides, however, are often given per weight of binder or concrete.

The importance of chloride ions for reinforcement corrosion in concrete has lead to the concept of the chloride threshold level or critical chloride concentration, which may be defined as the minimum chloride level at the depth of the reinforcement which results in active pitting corrosion of the reinforcement.

Another possible definition of the chloride threshold level in concrete is the critical chloride content at the steel surface at a specific time, when deterioration or damage to the concrete structure starts (Schiessl 1990).

The first definition corresponds to the point in Tuutti's model (Tuutti 1982) where the initiation period stops and the propagation period starts (see Figure 3.1 in Chapter 3). The second definition corresponds to a less well-defined point in the propagation period.

The methods used today for predicting the service life of reinforced concrete structures exposed to chlorides require knowledge of the chloride threshold level for the structure in question. For that reason it is of great interest to determine the chloride threshold levels for a given situation, not only characterised by the concrete mix design, but also by curing conditions, age, temperature, etc. Environmental factors determining the moisture state of the concrete and the electrochemical potential of the reinforcing bars are also very important parameters for the chloride threshold level (Arup 1993).

Normally, steel in concrete is passivated in a moist, alkaline environment which is free of chlorides and other aggressive ions. The term passivity denotes that although ordinary steel reinforcement in concrete is thermo-dynamically not stable, the corrosion rate is depressed to an insignificantly low level by the formation of a barrier of iron oxides on the steel surface. The state of passivation is maintained until the concrete in contact with the reinforcement becomes carbonated, or until a sufficient concentration the *threshold level* of water soluble aggressive ions (normally chlorides) has reached the steel surface. The aggressive ions (e.g. chlorides) then locally penetrate the iron oxide layer and trigger the dissolution of the iron oxide layer resulting in the dissolution of steel.

The chloride threshold level may be easy to detect in well defined laboratory experiments, since the corrosion rate increases by orders of magnitude after the initiation of corrosion. In the field however, the threshold level is often not so clear due to several reasons (HETEK-53 1996):

- The passivity of steel in sound concrete is not perfect. The passive oxide layer protecting the steel is non-uniform and so is the concrete.
- Both the "passive" and the "active" corrosion rate may vary due to the changing local exposure conditions (temperature, RH, etc.)
- The corrosion rate is often difficult to measure accurately in field structures (the area of the corroding steel is not easy to define).

Chloride induced corrosion mainly develops in localized corrosion pits, with a very small area as compared with the surrounding passive area. The chloride threshold level can be presented in several ways:

- A total chloride content by weight of concrete;
- A total chloride content by weight of binder;
- A free (water soluble) chloride concentration in the pore solution;
- A ratio between the free chloride and the free hydroxide concentration.

The free concentrations are difficult to determine. The total chloride content measured parallel to the binder content is easy to measure, and reflects the ratio of aggressive substance relative to the protective binder. This approach also gives the least spread in the results (Glass 1995a). For practical reasons the chloride content of concrete is often defined as the total amount of acid soluble chloride in per cent by mass of the dry concrete, because this is normally measured when performing chloride analyses on concrete samples. However, this means that the chloride threshold level will be strongly related to the cement content of the concrete.

In the case of prestressed structures the depassivation of the steel surface must be prevented, because very low corrosion rates can cause stress corrosion cracking of the prestressed steel.

But with ordinary reinforced concrete structures the depassivation of the reinforcement may not always lead to any deterioration. For example, if the environment is extremely dry, the corrosion rates after depassivation of the steel are very small due to the high electrolytic resistance of the concrete. On the other hand, if a concrete structure is totally water saturated, depassivation of the reinforcement will not cause high corrosion rates, because oxygen cannot reach the steel surface. Therefore the chloride content of the concrete which leads to depassivation is not the only decisive parameter governing the durability of concrete structures or the subsequent corrosion rate of the steel.

The minimum total chloride content, or the chloride threshold level, required for corrosion to occur has been the subject of a considerable amount of debate over recent years. A value of 0.4 % by weight of cement is often used (Bamforth 1999). However, it is clear that it cannot be represented by a single value. It is dependent on a number of factors such as:

- the w/c ratio;
- cement type;
- pH of the pore solution;
- capacity of the cement paste to bind the chloride ions;
- whether the chloride ions are 'cast-in' or ingressing from an external source
- the exposure conditions;
- measurement difficulties (especially for free chloride and hydroxide near the reinforcement);
- local variations in chloride concentration at the reinforcement.

Literature threshold levels typically range from 0.17 to over 2 % by weight of cement, with the probability of corrosion increasing with the chloride concentration. Values for the threshold level for ingressing chlorides in uncarbonated concrete in marine environments (Browne 1996) are given in Table 2.4.

Table 2.4 - Chloride threshold levels from Browne (1984).

Chloride ion concentration (wt% cement)	Probability of corrosion
0.4 >	negligible
0.4 - 1.0	possible
1.0 - 2.0	probable
> 2.0	certain

Table 2.5 shows values given by Vassie (1984). These are consistent with those of Browne, but show that there is a significant risk of corrosion for chloride ion contents below 0.4%.

Table 2.5 - Risk of corrosion at different threshold levels (Vassie 1984).

Chloride ion range (wt% cement)	% analyses showing corrosion
0.2	2
0.2-0.3	22
0.3-0.5	23
0.5- 1.0	32
1.0-1.5	64
1.5	76

Table 2.6 - Values for macro crack free concrete (width < 0.1 mm) and minimum reinforcement concrete cover of 25 mm - % by weight of binder (Frederiksen *et al* 2002).

w/c	Exposure condition	PC	20% pfa	5% silica fume	10% silica fume
0.3	Below sea water	2.2	1.4	1.6	1.2
	Marine splash zone	1.0	0.5	0.6	0.4
	Deicing salt splash zone	0.8	0.4	0.5	0.3
	Above splash zone	1.2	0.5	0.7	0.5
0.4	Below sea water	2.0	1.2	1.5	1.0
	Marine splash zone	0.8	0.4	0.5	0.3
	Deicing salt splash zone	0.6	0.3	0.4	0.2
	Above splash zone	1.0	0.4	0.5	0.3
0.5	Below sea water	1.5	0.7	1.0	0.6
	Marine splash zone	0.6	0.3	0.4	0.2
	Deicing salt splash zone	0.4	0.2	0.3	0.2
	Above splash zone	0.6	0.3	0.4	0.2

Frederiksen *et al* (2002) have given design threshold levels for various exposure conditions. This data is similar to Tuutti's (1999) and is presumably taken from the same source. These are summarised in Table 2.6. The chloride threshold levels are only valid for "macro crack free" concrete with a maximum crack width of 0.1 mm and a minimum cover of 25 mm. The data are not valid for calculations of the initiation time in cracked concrete with crack widths > 0.1 mm.

According to Nilsson *et a1* (1997) chloride threshold levels vary extensively in field exposed concrete exposed to the air, as a consequence of the varying microclimate at the steel surface. As a consequence the chloride threshold level depends on the cover thickness and on the physical bonding between concrete and reinforcement.

Table 2.7 - Some critical chloride threshold concentrations as reported in the literature. (Pettersson 1992; Pettersson 1993; Glass 1995b)

c_{CR} % Cl /wt. Binder	Remarks	Reference
0.06-1.0	steel in aqueous solutions pH = 12.5 - 13.2 (Cl/OH = 0.6)	(Hausmann 1967)
0.17-1.4	Field	(Stratful et al. 1975)
0.2-0.4	Varied with pH	(Cady 1990)
0.2-1.5	Field	(Vassie 1984)
0.25	Field	(West 1985)
0.25-0.5	Laboratory	(Elsener 1986)
0.3-0.7	Field	(Henriksen 1993)
0.4	Varied with cement type	(Brown 1982)
0.4	Outdoors	(Bamforth 1994)
0.4-1.6	Laboratory	(Hansson 1990)
0.48-2.02	Varied with cement type and admixtures	(Schiessl 1990)
0.5	Outdoors	(Thomas 1990)
0.5-1.4	Laboratory	(Tuutti 1993)
0.5-2.0	Laboratory	(Schiessl 1990)
0.6	Laboratory	(Locke, Siman 1980)
0.6-1.4	Varied with cement type, curing, water to binder ratio, admixtures, etc.	(Hansson 1990)
0.9-1.8	W/b 0.30-0.40, pH13.0-13.8, CI/OH = 3-6, varied with pH, cement pozzolan	(Pettersson 1993)
1.6-2.5	Laboratory	(Lambert 1991)
1.8-2.2	Field	(Lukas 1985)

Carbonation can reduce the threshold level by reducing the pH and the chloride binding capacity of the cement paste. When the carbonation front reaches the reinforcement, corrosion can commence in the absence of chlorides as the protective oxide layer breaks down. It is not clear what account the above threshold values take of the potential influence of cracking upon corrosion risk. Pitting corrosion is of particular concern because of the associated localized loss in cross-sectional area of reinforcement, which can be very severe in some circumstances.

Haussmann (1967) has shown that the probability of anodic reaction is dependent on the Cl^-/OH^- ratio. This has been reported to have a value of about 0.61 (this is reported to be valid for steel exposed to solution - the interface between reinforcement and concrete may have an extra protective effect). Tang and Nilsson (1996) report a value of about 1.l. The w/c ratio is also a factor. The threshold level raises as the w/c ratio increases (this is probably due to an increase in pH at lower w/c ratios and may not be a factor in carbonated concrete, although the lower permeability will inhibit the corrosion process). Leaching can reduce the OH^- concentration.

The very limited data on free chloride levels required to initiate corrosion are in the range 0.14-1.8 mol/litre. However, according to Tang and Nilsson (1996) a chloride content of 0.4 % by weight of cement corresponds to a free chloride concentration of about 0.9-1.0 g/l (0.025-0.028 mol/l). This corresponds to a Cl^-/OH^- ratio of about 0.6 in saturated lime. The threshold limit of 0.4 may only hold therefore at pH's of about 12.4 (i.e. when the alkali hydroxides have been removed).

Thomas and Matthew (1994) have suggested (Table 2.8) the following dependence of the threshold level on the PFA content of the concrete based on an analysis of BRE Report 216 (1994) data.

Table 2.8 - Dependence of the threshold level on the pfa content (BRE Report 216 1994).

pfa content (%)	Threshold Cl^- / weight % total cement
0	0.7
15	0.65
30	0.5
50	0.2

Dhir *et al* (1995) have suggested, based on experiments using cast-in chlorides, that there is no specific chloride level at which corrosion is initiated. Rather the corrosion rate increases gradually as the chloride level increases and that some corrosion activity will occur if free chlorides are present at the steel interface. They state that there is very little difference in the water-soluble chloride content required to initiate corrosion in equivalent strength Portland cement and fly ash concrete. However, the rate at which the chloride ions are transported to the site once corrosion has been initiated is important. The diffusion coefficient is therefore an important parameter.

The probabilistic nature of the chloride threshold level is reflected in Figure 2.13 which is adapted from BRE Digest 444 Part 2 (1999). The figure shows the probability that corrosion initiation will have occurred due to ingressing chlorides in uncarbonated concrete. The graph is based on available data from a range of sources and includes values taken from laboratory tests and actual structures. The central line represents the average.

The outer areas are intended to represent a normal distribution reflecting the fact that measured values from different sources show a large degree of scatter. The nature of the materials used and the exposure environment affect the risk of corrosion.

Values for a 1% chloride ion concentration at the reinforcement are shown as an example. Figure 2.13 shows that corrosion is likely to have commenced in about 60% of cases. However, the actual figure may vary between 30% and 90% depending on other factors such as the environment and the nature of the materials.

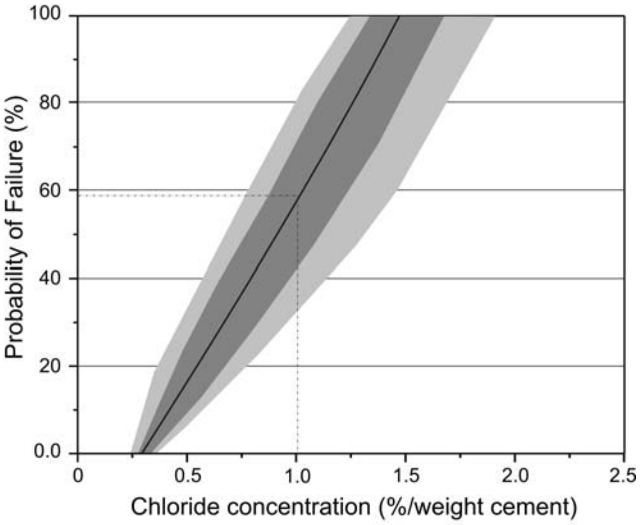


Figure 2.13 – Estimated risk of corrosion associated with ingressed chloride in the absence of carbonation (BRE Digest 444 – 1999).

As Figure 2.13 is based only on the limited available data it may need to be modified as knowledge and experience increase.

Chloride thresholds reported in the literature are confusing as they vary extensively depending on whether they refer to total or free chlorides. Chloride thresholds also vary depending on the experimental methods, the exposure conditions, the concrete quality and the cement type used.

The following is a list of some factors contributing to the large spread in critical chloride concentrations found in the literature:

- Some results are derived from steel exposed to saline solutions, while others are derived from steel embedded in concrete, mortar or cement paste. Several researchers (Sorensen 1990, Pettersson 1992, Yonezawa 1988, Chen 1988) have reported significantly higher chloride threshold levels for embedded steel as compared to that of naked steel exposed to a chloride solution. Yonezawa (1988) attributed the improved chloride resistance of embedded steel to (i) the interfacial interaction cement paste - steel, as visualised by the thick oxide layer, (ii) the formation of calcium hydroxide in the interfacial zone. The acid action of pitting corrosion may be counteracted by dissolving calcium hydroxide, thus buffering the pH in a local pit.

- Differences in chloride binding (Byfors 1990) and the formation of iron oxides (Yonezawa 1988) occur when samples are made with cast in chlorides as compared to samples being originally chloride free, everything else being equal. As a consequence, the critical total chloride content found for a given concrete quality varies with the experimental procedure.
- The passive steel potential and the alkalinity of the pore solution in contact with the steel most probably have a dramatic influence on the critical chloride concentration measured. However, most of the literature data is presented without this most important information. In some cases measurements or estimations of the concrete alkalinity were made, but the effect of passive steel potential is seldom taken into account. The present situation may be seen as remarkable, since the corrosion behaviour for all metals are more or less related to the potential as expressed in potential - pH diagrams (Porbaix 1966)

A review of chloride thresholds in the literature performed by Glass and Buenfeld (1995) indicates free chloride thresholds expressed as the ratio $[C1^-]/[OH^-]$ in the range of 0.26 to 40. The authors suggested that a major reason for the inconsistency in reported results is the difficulties associated with methods for extracting and analyzing the free ions in the concrete pore solution close to an emerging corrosion pit. No accurate method exists for studies of the pore solution composition in concrete with w/c ratio less than approximately 0.45.

The literature on chloride threshold levels for the initiation of reinforcement corrosion in concrete is very conflicting. If the most practical representation of the threshold level is used, e.g. total chloride by weight of binder, threshold levels have been reported in an interval of 0.17 to 2.5 % chloride by weight of binder, (Pettersson 1992, Glass 1995b).

Because of the uncertainties connected with the threshold level, national codes and recommendations, if they recognise the threshold level at all, tend to be conservative. They do not recognise the effect of exposure conditions, cover size and w/c ratio. In Great Britain, BS 8110 and in Norway, NS 3420, a maximum of 0.4 % total chloride by weight of cement is allowed in normal concrete. In America the ACI recommends a maximum of 0.2 % total chloride by weight of cement (Glass 1995b).

A wide range of parameters is likely to affect the chloride threshold level in concrete. Only very recently systematic approaches have been adopted in order to identify their relative importance. The following is a review of parameters which have some evidence of being important in controlling the threshold level. More extensive information can be found in an excellent review of the literature on chloride thresholds by (Glass 1995b).

There is a *chloride threshold* for corrosion given in terms of the chloride/hydroxyl ratio. It has been measured in laboratory tests with calcium hydroxide solutions. When the chloride concentration exceeds 0.6 of the hydroxyl concentration, corrosion is observed (Hausmann 1967). This approximates to a concentration of 0.4 % chloride by weight of cement if chlorides are cast into concrete and 0.2 % if they diffuse in. In the USA a commonly quoted threshold is 0.59 kg chloride/m^3 (1.0lb chloride/yd^3) of concrete. Although these figures are based on experimental evidence, the actual values are a function of practical observations of real structures.

All these thresholds are approximations because:

- Concrete pH varies with the type of cement and the concrete mix. A tiny pH change represents a massive change in hydroxyl ion (OH^-) concentration and therefore (theoretically) the threshold moves radically with pH.
- Chlorides can be bound chemically (by aluminates in the concrete) and physically (by adsorption on the pore walls). This removes them (temporarily or permanently) from the corrosion reaction. Sulphate resisting cements have low aluminate (C_3A) content which leads to more rapid diffusion and lower chloride thresholds.
- In very dry concrete corrosion may not occur even at very high Cl^- concentration as the water is missing from the corrosion reaction.
- In sealed or polymer impregnated concrete, corrosion may not occur even at a very high Cl^- concentration if no oxygen or moisture is present to fuel the corrosion reaction.
- Corrosion can be suppressed when there is total water saturation due to oxygen starvation, but if some oxygen gets in, then the pitting corrosion can occur.

Therefore corrosion can be observed at a threshold level of 0.2 % chloride by weight of cement if the concrete quality is poor and there are filler and oxygen available, in different circumstances no corrosion may be seen at 1.0 % chloride or more if oxygen and water are excluded. If the concrete is very dry or totally saturated then a change in conditions may lead to rapid corrosion.

On the other hand chloride thresholds based on the total chloride content seems more consistent, in the range of 0.17 to 2.2 % total Cl by weight of binder. It can be argued that the total chloride concentration by weight of binder reflects the ratio of [C1$^-$]/[OH$^-$] in the concrete pore solution for a given binder type, provided that the hydroxide concentration at the depth of the reinforcement is relatively unaffected by leaching. This assumption is based experimental data indicating a similar transport rate for hydroxide ions as for chloride ions. If the assumption is correct, the chloride threshold expressed as total chloride content by weight of binder reflects the probability of corrosion initiation for a given concrete in a given exposure condition.

2.3.2.1 Influence of cracks on corrosion activity

The influence of cracks on corrosion activity has been reviewed by Frederiksen *et al* (1992). The main conclusions were:

- Cross sectional cracks in the splash zone: Only a low corrosion activity can be observed in cracks with crack widths up to 0.4 mm, if the concrete cover thickness and/or permeability is sufficient to protect the corresponding non-cracked structure against chloride initiated corrosion during a long period of time. On the contrary, if the cover do not have a sufficient thickness or permeability, damage due to corrosion will be inevitable in the short or long term, even for the non-cracked structure. Cracks in the concrete leads to an earlier initiation of corrosion, but not to a higher corrosion intensity in the long term.
- Longitudinal cracks in the splash zone: The conditions for longitudinally cracks are basically the same as for cross sectional cracks. However, the maximum crack width, under which one would not expect pitting corrosion is somewhat lower, than in the case of cross sectional cracks.
- Cracks in the submerged zone: cracks from the surface to the reinforcement in submerged marine concrete will lead the chlorides rapidly into the reinforcement. But other conditions can prevent corrosion from taking place at a significant rate, e.g. self-healing.

2.3.3 Chloride Binding

Chloride binding in concrete is important since the bonding process removes chloride ions from the concrete pore solution into the solid hydrated binder phase. By analogy with general sorption processes in a porous solid phase - liquid phase system, equilibrium conditions are normally assumed between chlorides in the pore solution and chlorides bound to the solid hydrates. Equilibrium implies that under a given set of conditions, a certain concentration of dissolved, free chlorides corresponds to a concentration of chlorides bound to the solid phase. In practice, chloride binding may help to prevent or to delay the chloride activation of reinforcement corrosion in two different ways (HETEK-53 1996):

- The rate of chloride ingress into concrete is decreased, since the chloride binding process reduces the concentration of chloride ions in the pore solution available for transportation. At low concentrations the binding process is very effective, removing nearly all free chlorides from the pore solution.

- Only mobile chloride ions, i.e. ions dissolved in the pore solution, are believed to initiate reinforcement corrosion. The total amount of chlorides (free and bound) which can be tolerated in concrete at the depth of the reinforcement is therefore assumed to be a function of the chloride binding capacity. If this assumption is right, a higher chloride binding capacity implies that a higher total amount of chlorides (by weight of binder) corresponds to a given critical concentration of free chlorides in the pore solution.

The definition of what are free and what are bound chlorides is not clear. Chloride binding seems to vary in strength over a wide range, from chlorides which are not even dissolvable in boiling nitric acid, to chlorides which are released by minor changes in temperature, pressure, relative humidity, pH, etc. It is not known to what extent such loosely bound chlorides actually take part in diffusion processes, thus affecting both the chloride transport rate and the chloride threshold value.

The mechanisms of chloride binding are not quite clear. Both chemical and physical binding are believed to occur. *Friedel's salt*, $C_3A\cdot CaCl_2\cdot 10H_2O$, is commonly mentioned as a major chemical binding product. Some resaechers even claim that Friedel's salt is all of the chloride binding (Nagataki 1993). It seems, however, as if most of the bound chlorides are physically bound to a limited number of binding sites in the pore walls. The amount of physically (and chemically) bound chlorides depends on the content of chlorides. A linear relationship between the amount of Friedel's salt and the total amount of bound chlorides, i.e. the chemical binding is also concentration dependent, has been suggested (Nagataki 1993).

Although proper theoretical models for the mechanisms of chloride binding are not yet fully developed to model the process of chloride binding in concrete, several factors have been found to affect the chloride binding capacity of concrete.

Without going into details, it is important to note that the chloride binding capacity is largely affected by how the chlorides enter the concrete. If chlorides are added to the fresh concrete mix, the chlorides will alter the hydration procedure as compared to a chloride free mix. The formation of chloro-aluminate sulphates may become significant if chlorides are present in the early hydration of aluminate and ferrite in the cement. Chlorides also alter the pore size distribution of the hydrated binder, which is likely to affect the absorption capacity of the hydrated binder.

The following is a list of factors being known to, or being suggested to, affect the chloride binding capacity of a given concrete quality (Sandberg 1993, Byfors 1990, Larsson 1995, Sandberg 1995):

- The chloride binding properties are different for different binders, depending on what kind of Portland cement is used, especially the C_3A content is important but also other factors, and the amount and type of pozzolans have large effects (Tang 1993).
- The composition of the pore solution is decisive for chloride binding. It is important to consider the concentration of other ions present in the solution. Especially the hydroxides, i.e. the pH, have a large effect since the hydroxide ions "compete" for the binding sites with the chloride ions. (Tang 1993). The chloride binding capacity has been experimentally shown (Sandberg 1993, Byfors 1990) to be affected by the alkalinity and the sulphate ion concentration of the pore solution. Since hydroxide ions are removed from concrete in a similar way as chloride and sulphate ions are transported into it, the chloride binding capacity is affected by the pore solution chemistry, a property which is time dependent.
- As the binder hydration proceeds for a very long time in concrete with a low w/c ratio, an increased amount of hydrated binder will be available for chloride binding as the exposure time increases. Any self desiccation developed will reduce the amount of free water in the pore system, therefore probably affecting the concentration of chlorides and hydroxides among other ions.
- The physical absorption part of the chloride binding process is assumed to be a fairly rapid process (Byfors 1990). If the unhydrated clinker phases are hydrated in the presence of chlorides, the aluminate and ferrite phases are known to chemically bind chlorides by the formation of sulfonated aluminate hydrates related to Friedel's salt (Byfors 1990), a process which is also fairly rapid.
- Carbonation dramatically changes the chloride binding capacity to almost zero (Tuutti 1982). The specific surface area drops and the pore walls become more or less inert. Consequently, the chloride content in the surface near regions of a concrete exposed to air will be very low due to carbonation capacity (Tuutti 1982).
- The w/c ratio and the degree of hydration for hardening concrete should then be an important parameter for quantifying the binding of chlorides (Tang 1993).
- Little has been done regarding the effect of temperature level on chloride binding: this is a remarkable lack of knowledge since most concrete structures have temperature conditions far from the ones in laboratory conditions around +20°C, where most tests are performed. This is certainly a limiting factor for predictions of chloride penetration in structures. An increasing temperature reduces chloride binding and liberates bound chlorides into the pore solution. This effect is so large that it causes unexpectedly high free chloride concentrations in a sample from structures or field exposed specimens when taken into the

laboratory (Nagataki 1993, Mangat 1995).

- The type of aggregates in the concrete may influence chloride binding. Alkalis released from the aggregate may have such an effect (Larsen 1995), i.e. decreased binding.

In resume, we still have to rely upon measured data for every individual binder for quantification of the binding isotherms. Of course this is also applies to aggregates that are not inert.

2.3.3.1 Binding isotherms

The higher the total content of chlorides the more chlorides are bound (Akita 1995). Some results give a linear relationship but it is more probable that the relationship between the free and bound amount of chlorides is non-linear, i.e. the binding capacity is concentration dependent

The effect of chloride content or chloride concentration may be shown in different ways. A binding isotherm gives the bound and total amount of chlorides as a function of the free chloride concentration at a constant temperature.

The chloride binding isotherms have been mathematically described in a number of ways:

- a straight line with an intercept on the axis for bound or total chlorides,
- a Langmuir isotherm,
- a Freundlich isotherm and
- even as a modified BET isotherm.

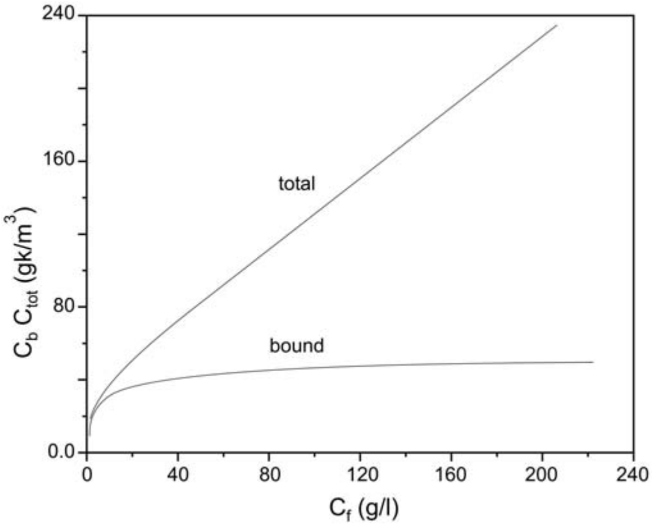


Figure 2.14 – Chloride binding shawn as a binding isotherm, i.e. with the bound chlorides as a function of the free chlorides. Two alternatives are shown at zero concentration: $c_b=0$ and $c_b=0.6\%$ by weight of binder.

The straight line is an oversimplification, at least when chloride concentrations increase (Tang 1993). All chlorides are not bound at low chloride contents. For *desorption* of chlorides, concentration decreases with time, the binding isotherm could be different at low chloride contents. Some of the bound chlorides could remain bound also at zero concentration in the pore solution. That would give a binding isotherm with an intercept on the axis for bound, or total, chloride content.

A number of more or less linear binding isotherms, with no intercept at the y-axis, i.e. straight linearity, have been presented (Tuutti 1982, Mangat 1995). Mangat (1985) even found negative binding capacities. Those binding isotherms usually originate from field samples where the free chlorides were measured by pore water extraction. A probable explanation for these discrepancies is the temperature effect on binding, already mentioned, and minor experimental errors in the extraction technique. The theoretical justification of the other descriptions is not yet clear and must be regarded as pure mathematic suggestions for the time being. More and more, accurate binding data and physical and chemical models are needed to obtain better descriptions.

It should be noticed, however, that a lot of binding data is derived from specimens where the chlorides were mixed into the original mix. The binding in such cases is not necessarily equal to the binding when chlorides penetrate into the hardening or hardened concrete. Additionally, when chlorides are removed from concrete the content of free and bound chlorides drops with another equilibrium binding isotherm (Tang 1993). A large hysteresis seems to exist between the amount of bound chlorides when the concentration increases or decreases.

Several factors can affect the chloride binding in concrete. The chloride binding in carbonated concrete is virtually zero (Tuutti 1982). As the carbonation of the binder increases the effect on the chloride binding is drastically reduced (HETEK 53 1996).

An increase in the temperature triggers a decrease in the binding capacity of the concrete (Larsson 1995). A decrease in the relative humidity decreases the binding capacity of the concrete. A decrease in the specific surface implies a decrease in the physical adsorption (HETEK 53 1996).

2.3.4 Chloride profiles

A measured or predicted distribution of chlorides at a certain time in a part of a structure or in a specimen is usually shown as a "chloride profile", a curve showing the total amount of chlorides as a function of depth (see Figure 2.15).

The penetration of chlorides by diffusion means that the content of chlorides is largest at the exposed surface and the content decreases with increasing depth. The slope of the profile is smaller and smaller at larger depths. At large depths a certain amount of chlorides may be present. That is the initial chloride content in the concrete which is mixed into the concrete.

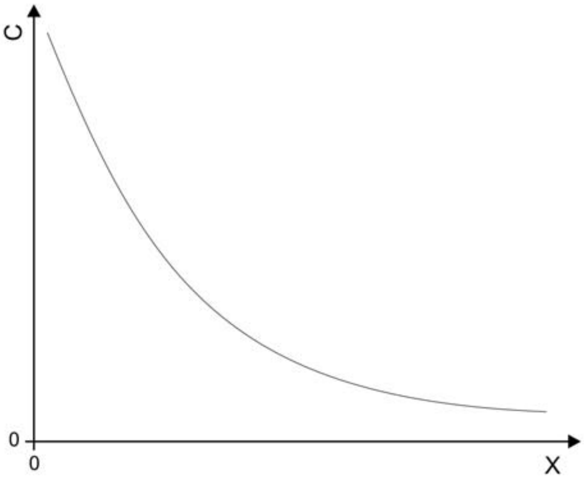


Figure 2.15 – Typical chloride profile in concrete.

The concentration of free chlorides in the pores of the concrete surface is usually assumed to be equal to the concentration of chlorides in the surrounding environment. The amount of bound chlorides is related to the concentration of free chlorides in the pore liquid. Consequently the profile of bound chlorides and the profile of the total amount of chlorides follow the profile for the free chlorides.

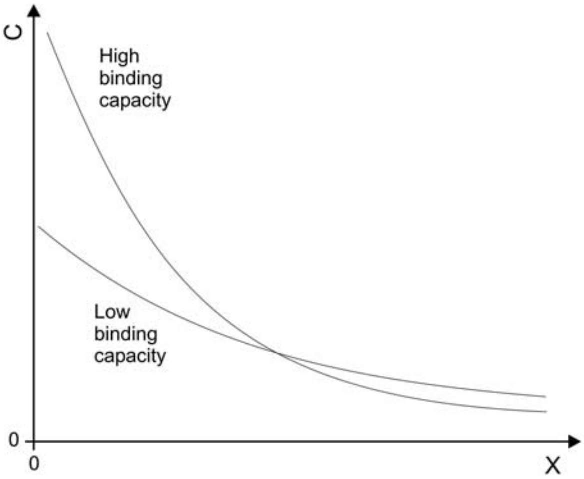


Figure 2.16 – Chloride profiles for two concretes with different chloride binding properties at the same time of exposure.

The binding of chlorides is different for different concretes depending on the concrete composition and the type of binder. The binding properties will influence the shape of the chloride profiles. In Figure 2.16 the effect of binding is shown as a comparison of chloride profiles for two concretes which differ only with respect to chloride binding properties.

The concrete with a higher binding capacity will have higher total chloride content at the concrete surface, since the amount of bound chlorides is larger. On the other hand, the penetration depth in that concrete will be smaller because more of the chlorides that penetrate are bound, which will delay the penetration.

However, measured chloride profiles do not always have the simple theoretical shape as expected from pure diffusion. Frequently lower chloride content is found close to the surface and a maximum in chloride content is found at some depth (Figure 2.17).

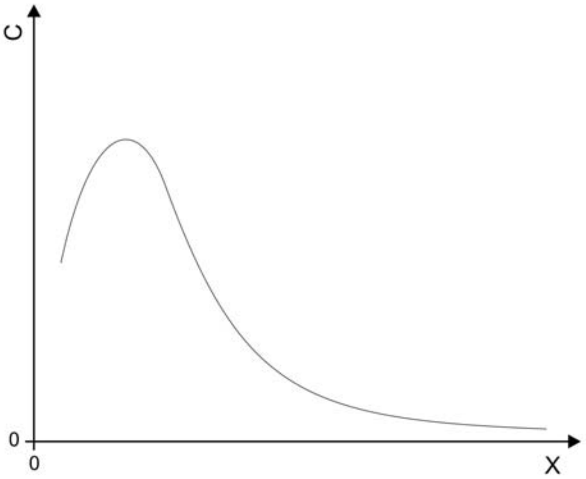


Figure 2.17 – A more realistic chloride profile with a lower content at the surface and a maximum at a certain depth.

The shape of the chloride profile in Figure 2.17 may have several explanations. Variations in the environmental conditions could cause such a shape, with water movement and evaporation concentrating chlorides at some depth and washing out chlorides from the surface near region. Other explanations include carbonation and leaching of substances other than chlorides, both of which influence the binding properties.

2.4 Service life design

Inadequate durability is by far the most common cause of premature failure of concrete structures, yet very little attention is devoted to durability in the design process. Generally, durability is covered by prescriptive code requirements based on previous code clauses, arbitrarily tightened where case histories have shown problems. However, this approach is suspect when applied to new materials or to design lives longer than relevant experience. Service lives of 120 year and longer are regularly being specified, even though reinforced concrete structures did not exist 100 years ago. Largely in recognition of these shortcomings, service life prediction is currently an area of significant international research.

The service life for a structure can be defined as (Eurocode 1, 1994):

> "A structure shall be designed and executed in such a way that it will, during its intended life with appropriate degrees of reliability and in an economic way:
>
> - remain fit for the use which it is required; and
> - sustain all actions and influences likely to occur during execution and use"

Specific reference is made to the inclusion of durability in these requirements. It continues:

> "The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing, and by specifying control procedures for design, production, execution and use relevant to the particular project."

Service life can also be defined as the time for which a structure is expected to be able to fulfil its requirements with sufficient reliability with or without periodic inspection and maintenance and without unexpected high costs for maintenance and repair (CEN 1999). Associated with the definition of service life is the concept of limit state based on performance criteria.

The performance of a structure is referred to its ability to fulfill demands and requirements set by the owner, authorities and users. Concrete structures are usually expected to have a long service life since they often require large investments. This means that it is necessary to have an understanding of the interrelations between design, material, deterioration and future maintenance. Furthermore it is necessary to have an understanding of how aging of concrete changes the design parameters.

Depending on the type of structure different definitions of the service life are used. In DuraCrete (1997) three different types of service lives are defined:

- Technical service life: The technical service life is the time in service until a defined unacceptable state of deterioration is reached;
- Functional service life: The functional service life is the time in service until the functional performance of the structure becomes obsolete, due to changed requirements from e.g. authorities or the owner;
- Economic service life: The economic service life is the time in service until it is

economically interesting to replace the structure than to maintain it.

The service life for a structure can be described as a combination of the service lives for the different components of the structure. The component with the shortest service life dictates the service life of the structure. With this approach it is possible to get information about that component that determines the service life for the structure.

Andrande (1995) proposes specifying design life in quantitative terms:

- required service life: the service life specified to meet user's requirements (e.g. as stated in the client's brief for a project or in a performance specification);
- design life: period of use intended by the designer (e.g. as stated by the designer to the client to support specification decisions).

The design life would not be less than the required service life but it may be longer when the designer wishes to introduce an extra allowance for uncertainties or to increase the probability of achieving at least the required service life. The required service life is defined corresponding to the requirements given in common regulations, codes and standards in addition to possible special requirements of the client.

Munch-Petersen (2001) propose an alternative definition of service life, a so-called residual service life. The residual service life is determined by subtracting the calculated service life from the time passed since the structure was constructed. Thus, when the residual service life is equal to zero, the service life of the structure is ended. However in reality, the residual service life will never reach zero, but the structure will undergo maintenance and/or repair before the critical time is reached.

Most of the degradation factors affecting concrete and its reinforcement are time dependent in their effect. It is somewhat surprising, therefore, that few standards or codes of practice for design of concrete structures currently contain any indication of the service life that can be expected from those structures. This omission is, however, rapidly being addressed within several international fora.

The ideal service life situation occurs when ageing is so slow, that the structure maintains satisfactory serviceability for the anticipated service life without unexpected high costs for maintenance. To cope with aggressive exposure conditions, care shall be taken during all phases of the design and execution of the structure to take advantage of available knowledge regarding exposure and deterioration of concrete structures. Already at these early stages the need for future inspection and maintenance shall be taken into account as a foreseen follow up on the as built information. This means that structures shall be designed to be accessible, inspectable and maintainable to the extent possible (Rostam 2001).

2.4.1 Design for durability and service life

The past has taught us that the classical procedures for design, construction and use of concrete structures have failed to provide reliable long-term performance in aggressive environments. To improve this situation a new concept for durability design needs to be established. Similarly to the current procedures for structural design, durability design shall be performance based taking into account the probabilistic nature of the environmental aggressivity, the degradation mechanisms and the material properties as achieved in the finished structure.

The process of design, construction and the use of a structure influence the service life. This process can be divided into the following stages, DuraCrete (1999):

- *Required performance*. The first step is to define the required functions and the quality level of these functions. The required functions must be translated into requirements for the performance. It must be possible to express the required performance in quantifiable terms, usually as limit states.
- *Defense Strategy*. A structure can fulfill the required performance in many different ways. The designer makes decisions about the materials to use, e.g. the composition and quality of the material, the structural system and the defense actions.
- *Maintenance Strategy*. The defense actions are determined from the maintenance is planned for the structure. The designer decides which kind of maintenance that should be applied, e.g. maintenance free or planned maintenance. Planned maintenance can be divided into periodic maintenance, used based maintenance or maintenance on complaints.
- *Design*. During the design phase the structure is designed in such way that the required performance is achieved with the chosen defense and maintenance strategies.
- *Construction*. During the construction phase the design concepts are executed. If deviations from the original design concept occur during the process of construction a redesign must be made. An important factor during the construction phase is how the structure is executed, e.g. size of concrete covers and compactation of concrete.
- *Maintenance Plan*. A maintenance plan is established when the construction phase is finished. This plan is derived from the maintenance strategy, the design and the actual construction.
- *Use and Maintenance*. During the use and operation of the structure the maintenance plan is used. If the structure is repaired or changed in some way a redesign may be

necessary and a new maintenance plan has to be established.

As mentioned before, the performance of a concrete structure will, due to environmental actions and age-dependant variations of the material, vary over time. When the concrete is young the positive effects are dominant and the performance is improved. After a certain time the degrading effects are growing dominant and the performance decreases.

Design strategies

In principle, two basically different design strategies for durability can be followed (Rostam et al 1994):

- Avoid the degradation threatening the structure due to the type and aggressivity of the environment.
- Select an optimal material composition and structural detailing to resist, for a specified period of use, the degradation threatening the structure.

CIB W94 (Croce 1993) identifies the need for a pre-design phase which is aimed at working out a performance programme, i.e. setting objectives at the engineering level, and defining design strategies for achieving performance objectives. Wilson and Wallbank (1993) suggest that the designer should prepare a clear statement summarising the strategy to be adopted. There are a number of such design strategies available (Beeby 1993):

a) avoid degradation by use of non-reactive materials
b) use concrete itself as a protection (e.g. provision of adequate cover)
c) block ingress of deleterious substances by a barrier
d) inhibit degradation reactions
e) arrange for planned replacement
f) combined solutions

Strategies a), c) and d) aim to avoid degradation and depend on the reliability of the system; b) and e) are design approaches, although reliability will be a major influence on the effectiveness of b) (e.g. achievement of cover).

The choice of strategy depends on the particular degradation mechanism, for example: carbonation or chloride induced corrosion use concrete as a protection i.e. adequate quality to resist for intended life or avoid through material selection (non-corroding reinforcement); inhibit freeze-thaw action by air entrainment (or adequate strength) abrasion use concrete as a protection i.e. adequate quality to resist for intended life or avoid by inhibiting action (surface protection) alkali-aggregate reaction avoid through material selection chemical attack use concrete as a protection i.e. adequate quality to resist for required life or avoid through blocking ingress (i.e. changing the environment by use of coatings, barriers, etc.)

It is an assumption in design that the durability of a structure, or part of it, in its environment is such that it remains fit for use during the design working life given appropriate maintenance. The structure should be designed in such a way that deterioration should not impair the durability and performance of the structure having due regard to the anticipated level of maintenance. The following interrelated factors shall be considered to ensure an adequately durable structure:

- the intended and possible future use of the structure;
- the required performance criteria;
- the expected environmental influences;
- the composition, properties and performance of the materials;
- the choice of the structural system;
- the shape of members and the structural detailing;
- the quality of workmanship, and level of control;
- the particular protective measures;
- the maintenance during the intended life.

The environmental conditions shall be appraised at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials and products. The degree of deterioration may be estimated on the basis of calculations, experimental investigation, experience from earlier constructions, or a combination of these considerations.

The development of the service life is influenced by a number of factors. Three main factors can be identified: (1) constituent materials, (2) execution during construction and (3) environmental actions during construction and service period.

Each of these three main factors is influenced by other parameters, e.g. design of structure, cement type, type of curing and temperature conditions. To make careful predictions of the service life, it is required to have mathematical models, in which the influencing parameters are included. The parameters can then be numerically quantified, where both mean values and scatters are described. The service life can be predicted with different principal methods, depending on how the scatters in the influencing parameters are considered and the physical background of the mathematical model, shown in table 2.9.

Table 2.9 - Levels used in a service life design. DuraCrete (1997)

Level	Description
0	No probabilistic and/or physical background. Deem-to-satisfy-rules (rules of thumb)
1	Partial Factor Method (Calibrated to meet explicit or implicit safety demands)
2	Probabilistic analysis. First Order Approximation.
3	Full Probabilistic Analysis
4	Explicit Cost-Benefit Optimization

The deem-to-satisfy rules are usually based on rules of thumb, where the durability of concrete structure is based on experiences drawn from the behavior of existing structures. For example, the durability is specified in terms of w/c ratio and concrete cover. The service life for a structure where the durability design has been made with deem-to-satisfy rules will be long but explicitly not specified. With deem-to-satisfy rules it is not possible to assess which effect different design alternatives, e.g. different concrete compositions or concrete covers, have on the service life. Furthermore it is not possible to plan maintenance and repair of the structure in advance for different design alternatives.

This implies a need to express the service life in a quantitative way, i.e. in years of satisfying condition for the structure. With this approach a performance is required for the structure and when the performance is not satisfying the end of the service life is reached. This methodology is called “Performance based design”.

Structural design versus durability design

When designing a structure today, the designer first defines the mechanical loads to be resisted. As these loads usually vary, some safety factors are applied to be on the safe side. These factored loads must then be resisted by the structure through selecting a combination of structural systems, element geometry, material types and material strengths. As it is difficult to predict the precise properties of the materials, the geometry and the qualities, which are achieved in the structure, safety factors are applied to account for this by limiting the maximum allowable stresses. Mathematical equations are then used to verify that the probabilities of the loads, which exceed the resistances, are maintained at an acceptable low level.

When it comes to durability design to verify that the intended life can be achieved with an acceptable level of reliability, the situation is entirely different. It seems to be acceptable without question to use a grossly over-simplistic approach. The codes provide only qualitative definitions of exposure and they fail to define the design life in relation to durability (Bamforth 1998). In particular, they fail to quantify the durability limit states that must be exceeded for the design life to be ended. Previous approaches fail to recognise that, in relation to durability, it is not the properties of the materials or components alone that define performance, but the condition of the structure in its environment as a whole, and its need for intervention. This performance can be defined by functional requirements such as fitness-for-purpose, which includes issues such as deflections, cracks and spalling, vibrations, aesthetics and structural integrity.

When it comes to residual service life assessment of existing structures the same situation of using over-simplistic procedures prevail as for service life design of new structures. This is in spite of the fact that once the structure is a reality a large part of the inherent uncertainties associated with new designs, can be eliminated or limited to required levels through appropriately planned inspections, measurements and testing. The longer the structure has served, and the longer it has interacted with the environment the more reliable could the residual service life forecast be. This is the condition, which shall be kept in mind when performing assessment and residual service life evaluation (FIB 1999).

Durability enhancing measures

For the very large majority of ordinary structures to be placed in aggressive environments the design approach for durability described in the foregoing sections will ensure a satisfactory service life. A conscious choice of cement type, concrete mix (especially w/c ratio), concrete cover, curing (moisture and temperature control) and geometry of the exposed parts of the structure, all adapted to the identified aggressivity of the environment, will normally result in a satisfactory service life of these structures.

However, it must be recognised up front that in some selected or special cases the usual choice of design parameters for durability will not provide adequate service life, often just because a very small or local part of the structure is particularly prone to premature deterioration. This would typically be in the following situations:

- The environment is particularly harsh and aggressive;
- The available ingredients of the concrete are unavoidably polluted, particularly if this is with chlorides in the fine aggregates and in the mixing water;
- The available workforce does not have adequate knowledge or understanding of the major importance of the execution process in providing a long term durable structure;
- The required service life is very long compared to normal structures, say 100 or 200 years, or more;

In all the above situations it should be recognised up front that maybe concrete alone should not be relied upon to provide the service life needed, and durability enhancement measures should be taken into consideration such as:

- Provide structural protection, by using:
    - smooth surfaces with rounded edges and minimise the area exposed to the environment
    - roof, eaves or similar to protect concrete surfaces against rain
- Provide special protection of the concrete, which usually also will protect the reinforcement, by using:
    - tanking of foundations (bituminous sheets)
    - surface protection, surface coating and surface insulation
    - lining of exposed surfaces (stainless steel lining)
    - permeability controlled formwork liners
    - self-compacting concrete
    - fibre reinforcement
- Provide special protection against corrosion of the reinforcement by using:
    - high performance concrete
    - increased concrete cover. Provide special skin reinforcement (e.g. stainless steel mesh) for very large covers, say > 100 mm
    - non-corrodable reinforcement, specifically stainless steel
    - coating of the reinforcement, specifically epoxy coated reinforcement or hot dip galvanized reinforcement
    - preventive cathodic protection
    - corrosion inhibitors or pore blockers
    - prestressed reinforcement in protective sheathings with corrosion protective grout
- Provide intensified inspections for early warnings, and introduce regular maintenance routines, supported by using:
    - durability or performance monitoring through sensors
    - selective testing and service life updating

In the EN1990 (2002), classes and examples are given for design working life, which is defined as “the assumed period for which a structure is to be used for its intended purposes with anticipated maintenance but without major repair being necessary.” As such, this is essentially the same as the more commonly used term 'required service life'. These classes and examples are reproduced in the following table.

Table 2.10 - Proposed values for design working lives from ENV 1990.

Design life category	Indicative design life (years)	Examples
1	10	Temporary structures
2	10 to 25	Replaceable structural parts (e.g. gantry girders, bearings)
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures

2.4.2 Performance based design

The concept of design life requires the end of the design life to be defined. It is usually some level of damage caused by the deterioration process, e.g. a level of cracking caused by corrosion. The end of design life will also depend on whether the damage is obvious; less obvious damage would warrant more conservative criteria.

A rational way to determine the service life of a concrete structure is to define a performance for the structure in question. With the performance criteria it should be possible to specify explicitly define desired functions of the structure. The performance can for example be related to load carrying capacity, durability, stiffness, appearance, operability, inspectability and maintainability. The performance criteria may vary between different concrete structures and for different owners and users. For some structures the performance criteria can be related to the ability of the structure to operate without any maintenance, while for other structures the performance criteria is important to have a non-deteriorated appearance at all times.

Therefore, a performance based design for a certain structure is based on performance criteria defined for the considered structure. The performance criteria may be related to for example a required function or the appearance of the structure. Usually the performance criteria are defined as limit states.

Definition of Limit state

In designing for durability, the first step is the definition of the desired/required performance of the structure. The client or the owner of the structure is asked to define the required target service life and the event that identifies the end of service life.

A central part of a performance-based design is the limit state. A limit state can be defined, according to DuraCrete (1999), as “A Limit State is the border that separates desired states from the undesired or adverse states in situations, acceptable to the owner, which a structure may be subjected to during its lifetime.”

The limit state that have been defined in the structural codes are:

- Serviceability limit state (SLS), defined as the limit between the state where the performance of the structure is acceptable and the state where the structure is no longer serviceable. Normally associated with economical consequences. Example of SLS are the onset of corrosion, crack widths, spalling, vibrations, aesthetics, etc.
- Ultimate limit state (ULS), defined as the limit between the state where the structure is no longer serviceable and the state where the structure has collapsed, for example, due to excessive material degradation. Example of ULS are collapse, buckling, and loss of instability of the structure.

The EN 1990 defines a number of limit states as “beyond which the structure no longer fulfils the relevant design criteria”. These limit states define the “failure criteria” dealt with in the design. The “failure criteria” for ULS is linked to structural resistance, while the end of SLS might be characterized by a “Design Service Life” (number of years). The failure criteria for ULS are fairly well defined in the Eurocodes. The failure criteria for the SLS of a concrete structure should be quantified in EN 1992 (basis of design – material specific). Such SLS criteria are however only described in a qualitative way not suited as a direct basis for probabilistic calculations.

The fundamental requirement for a structure is that during the intended service life the probability to exceed specified limit states is sufficiently small (Beeby 1993). Many times ULS and SLS are coupled, e.g. if corrosion of reinforcement is initiated, where first an aesthetic problem occurs, with rust stains on the surface of the structure (SLS), then cracking and spalling due to the formation of corrosion products (SLS) and finally collapse of the structure due to reduction of the cross-section area of the reinforcement (ULS).

Definition of limit states of degradation for corrosion

A number of limit states have been proposed to define critical points in the deterioration of a structure (Cairns 2003). Those proposed include initiation of corrosion, initiation of longitudinal cracking, a limiting longitudinal crack width, loss of steel section to a defined level, and loss of structural integrity.

- *Depassivation of reinforcement* - The service life is limited to the initiation period, which means the time for the aggressive substance to reach the reinforcement and induce depassivation. The initiation phase ends when the chloride concentration at the reinforcement reaches a critical threshold value or when the carbonation front reaches the reinforcement. Depassivation does not necessarily represent an undesirable state. However, this event must have occurred for corrosion to begin.

- *Cracking of concrete cover* - It is evident that initiation of corrosion, although representing a crucial change of condition for durability, is of no immediate consequence for structural capacity, and constitutes a limit state of durability only. The second event is cracking of the concrete cover due to the expansive forces generated by the corrosion products. In this case the service life includes a certain propagation period of corrosion activity during which the cross section area of the reinforcement is progressively reduced. The crack width depends on the amount of corrosion, the cover/diameter ratio, the concrete quality (tensile strength) and the position of the bar. It is typically reported that reductions in reinforcement radius of between 0.01 mm and 0.04mm are required to initiate longitudinal cracking. This represents a reduction of less than 2% of cross sectional area for a small diameter reinforcement, and less for larger reinforcement. Clearly, cracking develops well before loss of bar section becomes structurally significant. Bond strength at initiation of cracking is generally also reported to be similar to or higher than that of the structure as new. Longitudinal cracking will thus precede significant loss of structural capacity, and is also a durability limit state only. According to the DuraCrete project (1998), a crack width of ≈ 0.3 mm is commonly considered to represent a serviceability limit stale for structural design. The limit is selected mainly on aesthetics grounds, as narrower cracks are considered to be not readily noticeable.

- *Spalling of concrete cover* - Corrosion continuing after cracking may lead to spalling of the concrete cover. Depending on the bar spacing, the concrete cover, and the tensile strength of the concrete, spalling may be either in the form of local triangular formed parts of concrete along each corroding bar, or the splitting forces from several corroding bars may interact and spall the cover over larger areas. The loss of reinforcement cross section and to some extent also the loss of concrete section will lead to a reduced load carrying capacity. Spalling of concrete is usually considered an unacceptable condition. However, spalling does not necessarily lead to collapse of the structure and can, therefore, be considered as a serviceability limit state. On the other hand, spalling of concrete on a public structure such as a highway over bridge is clearly unacceptable. In such a case spalling must be considered as an ultimate limit state. Based on available knowledge spalling is supposed to occur when a crack width of approximately 1.0 mm has been reached. The propagation time is assumed to end at this stage. The limit state of spalling, defined according to Rodriguez *et al.* (1996), is the time to growth of longitudinal crack width to 2.0 mm.

- *Collapse* - An ultimate limit state is reached when a structure or a part collapses. Collapse of the concrete structure will occur if the load carrying capacity of the element is reduced sufficiently due to ongoing corrosion, by further cross sectional loss of the concrete and steel, or loss of bond. A reasonable margin of safety must therefore he maintained against an ultimate limit state being reached. In the context of corrosion damaged concrete, an ultimate limit state could be reached at collapse of a structure or object, of a component, or, through spalling of cover concrete of a surface of a component. A section loss (typically of 10% of reinforcement cross section) has on occasion been considered to represent a limiting condition (Cairns 2003).

2.4.3 Calculation methods

The performance function, expressed with a limit state, can be solved either with deterministic or probabilistic methods. With deterministic methods, only one single value is used as input data for each of the parameters in the model, e.g. mean values. With probabilistic methods, mean values, statistical uncertainties and statistical distribution functions, are used as input data for the parameters in the model.

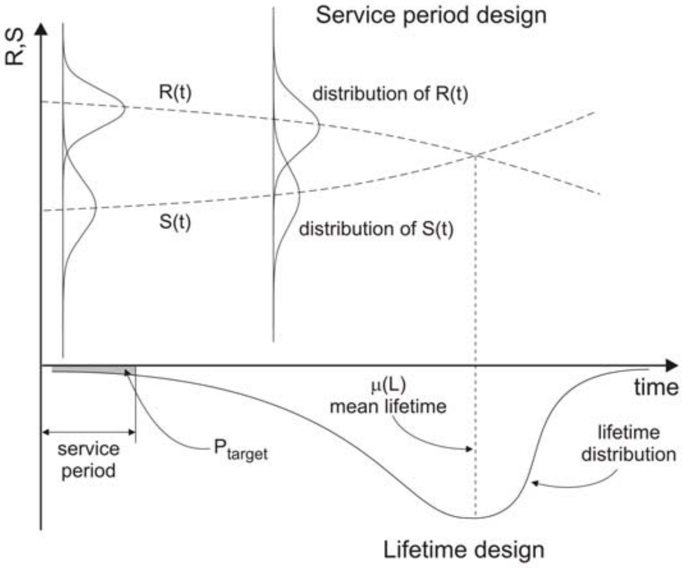


Figure 2.18 - Service life predicted with only mean values and mean values and scatters of the parameters in the prediction models. R(t) and S(t) are expressed with both mean values and scatters. DuraCrete (1999)

In figure 2.18, the difference in predicted service lives, if deterministic methods or probabilistic methods are used, is shown. The bearing capacity or strength, R, and the load, S, vary over time. Both functions are expressed with mean-values and statistical uncertainties expressed as a statistical distribution function. The mean values of R(t) and S(t) are characterized by the dotted lines in figure 2.18. The mean service life is reached when the mean values of $R(t)$ and $S(t)$ cross each other, while the service life determined with probabilistic methods is dependent of the chosen target reliability, $P_{target.}$

The probability of a certain event can be expressed with a probability function. The probability function is defined by events and it can be extended with the use of random variables. The probability functions can be both linear and non-linear. If many independent random variables, with arbitrary distribution functions, are summarized the sum will, according to the "central limit theorem", be normally distributed.

Probabilistic limit state function

The limit states should satisfy the primary demands for the structure, with acceptable means. The primary demands are either set by the owner or by the authorities, e.g. in the building codes. When the primary demands are specified the architect and/or the structural engineer translate them into functional demands.

The performance of a structure can be expressed with a performance function. In a general case the performance is expressed as a number of basic parameters and compared with a defined limit,

$$B\{X_1, X_2, \ldots., X_i, \ldots., X_n\} \geq L \quad or \quad B\{X_1, X_2, \ldots., X_i, \ldots., X_n\} \leq L \tag{2.22}$$

where:

B - the performance function.

X_i - the i-th basic parameter.

n - the total number of parameters.

L - the limit.

Usually in engineering problems it is convenient to divide the performance function (2.22) into two parts representing the strength and the effects of the loading respectively, see (2.23). This function is called a Limit State Function.

$$R(X_1,X_2,\ldots,X_i) - S(X_{i+1},X_{i+2},\ldots,X_n) > 0 \tag{2.23}$$

where:

R - the function (model) that describes the bearing capacity or strength of the structure

S - the function (model) that describes the influence of the load on the structure.

X_i - a basic variable.

In general, the loads *S* that are applied to a structure fluctuate with time and are of uncertain value at any point in time. Similarly, the structural resistance *R* is also a function of time owing to deterioration and other similar actions. Therefore, the probability of failure is also a time dependant quantity.

The measure of risk associated with the specific event of $R(t) < S(t)$ can be expressed as the probability of failure $p_f(t)$ of that event, or generically as:

$$p_f(t) = P\left(R(t) < S(t)\right) \tag{2.24}$$

or

$$p_f(t) = P\left[G(R,S,t) \leq 0\right] \tag{2.25}$$

where *G( )* is defined as the limit state function and the probability of failure is identical with the probability of the limit state violation.

The basis of limit states is that when a structure goes into an adverse state, it does not meet its primary demands (directly measurable) and thus must be repaired to be brought back in the desired state. With this approach all structures can be put into two different classes:

- Safe (S < R): If $R - S > 0$ the structure is considered to be in a safe state and if $R - S = 0$ the structure the structure is considered to be in a state on the border between safe and not safe.
- Not safe (R > S): *If* $R - S < 0$ the structure is considered to be in a not safe state.

The advantages to define limit states are that they provide a good basis for the design process and facilitate the planning for inspection and maintenance, DuraCrete (1999). It is possible to make an economic optimization if for example repair actions can be planned in advance in an effective way. If, for example, failure of a concrete beam is defined as a limit state, economic optimization may lead to the conclusion that it is best to stop carbonation or chloride ingress at the point when the reinforcement starts to corrode.

According to equation 2.24 the probability of failure increases continuously with time as schematically represented in Figure 2.19. The resistance *R*, is represented by a distribution that varies with time. The same applies to the load *S*. With time the distributions approach each other forming an overlapping area of increasing size that can be graphically interpreted as the probability of failure.

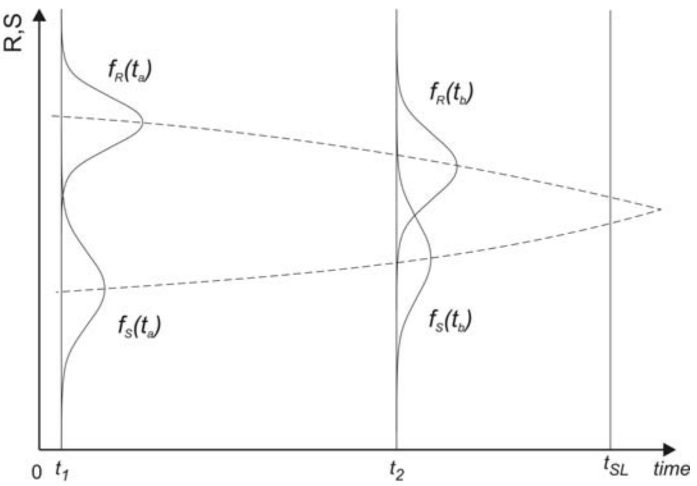


Figure 2.19 - The increase of the probability of failure. Illustrative presentation (Melchers 1999).

When the resistance and load variables are independent and continuous, and each is described by a known probability density function, *f_R (...)* and *f_S (...)*, then the failure probability becomes:

$$p_f = P\left(R - S < 0\right) = \int_{-\infty}^{+\infty} \int_{-\infty}^{s \geq r} f_R\left(r\right) \cdot f_S\left(s\right) dr \cdot ds \tag{2.26}$$

Considering continuous distributions the probability of failure at a certain moment of time can be determined using the convolution integral (Naess 1995). The solution of the convolution integral is only available in a few cases, i.e. when the distributions of *R* and *S* are normal. However, the integral can be solved by approximate numerical methods.

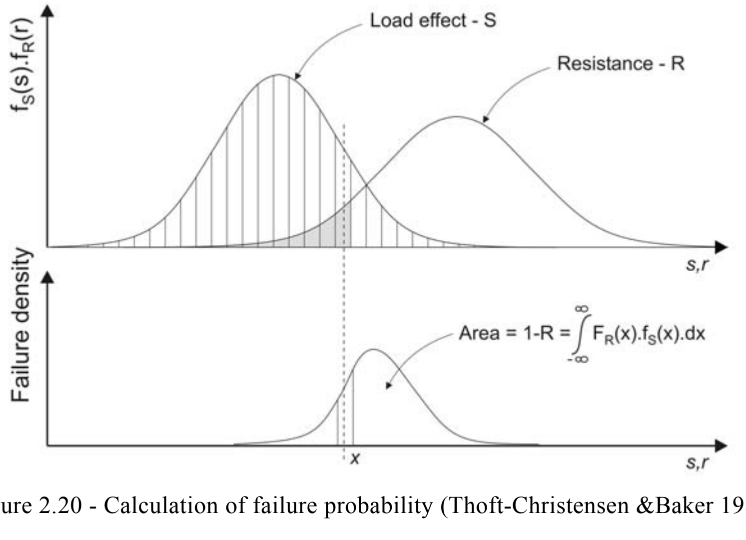


Figure 2.20 - Calculation of failure probability (Thoft-Christensen &Baker 1982).

Equation (2.26) gives the total probability of failure, P_f, as product of the probabilities of two independent events, summed over all possible occurrences. This means that the probability P_1 is that *S* lies in the range *x, x+dx* and the probability P_2 is that *R* is less then or equal to x, see (2.27).

$$\begin{cases} P_1 = f_S(X).dx \\ P_2 = F_R(x) \end{cases} \tag{2.27}$$

With the reliability, for the structure, the probability that the structure will survive when the load is applied is determined with (2.28).

$$Reliability = 1 - p_F = 1 - \int_{-\infty}^{+\infty} F_R(x) \cdot f_S(x)\,dx \tag{2.28}$$

Symmetry gives that (2.28) can be rewritten as (2.29).

$$Reliability = 1 - p_F = 1 - \int_{-\infty}^{+\infty} \left(1 - F_S(x)\right) \cdot f_R(x)\,dx \tag{2.29}$$

In figure 2.20 equations (2.28) and (2.29) are visualized.

There are two approaches to durability problems, that in principal they are equal. (DuraCrete 1999): Intended service life period, and Lifetime design.

The intended service life period imply that the specified limit state should not be reached, with a certain probability, within the intended service life period. The concept of the intended service life period is given by

$$P_{f,T}\,\{R(t)-S(t) < 0\}_T \leq P_{acc} = \Phi(-\beta) \tag{2.30}$$

where:

$P_{f,T}$ - the probability of failure of the structure with the period T.

T - intended service period

P_{acc} - the accepted maximum value of the probability of failure

Φ - the standard normal distribution function

β - the reliability index.

In the lifetime design the reliability of the structure is related to the probability that the design service life will be exceeded. The service life is ended at the moment when the limit state is reached. The concept of lifetime design is shown in (2.31).

$$P_f = P\{L < T\} < P_{acc} = \Phi(-\beta) \tag{2.31}$$

where:

P_f - the probability of failure,

T - the lifetime of the structure.

The lifetime of the structure, L, can be rewriting the limit state function (2.31) and writing it as a function of time,

$$L = t\{ R(t),S(t)\} \qquad (2.32)$$

where:

L - the lifetime of the structure.

$R(t)$ - the strength.

$S(t)$ - the load.

Reliability index β

The probability of failure P_f can be expressed through a performance function G such that a structure is considered to survive if $G > 0$ and to fail if $G < 0$. If R is resistance and E the effect of actions, the performance function is: $G = R – E$. If G is normally distributed, the reliability index β is taken as:

β = (mean value of G)/(its standard distribution)

The relationship between P_f and β for this situation is given in the following table:

Table 2.11 - Relationship between P_f and β.

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}
β	1.28	2.32	3.09	3.72

Target values of reliability index β

From prEN 1990 (2001) and NS 3490 (1999), reliability indexes might be found for typical cases like:

- ULS (residual and office buildings where consequences of failure are medium) - β = 3.8 for 50 years reference period;
- SLS - β is in the range of 1.5 for 50 years reference period.

Tables 2.11 and 2.12 present values for reliability indexes from several sources.

Table 2.11 - Minimum reliability index β_{SLS} during service life.

Proportionality factor	β_{SLS} (Lifecon 2003)	β_{SLS} (LNEC E-464 2004)
Low	2.0	2.0
Normal	1.5	1.5
High	1.0	1.2

Table 2.12 - equired reliability index β_{SLS} for the state of depassivation of reinforcement according to TG 5.6 of fib (2000).

Proportionality factor	Exposure class	β_{SLS}
Low	XC4, XD1, XS1, XS3, XD3	2.0-3.5
Normal	XC2, XC3, XS2, XD2	0.0-3.5
High	XC1	0.0-2.5

The proportionality factor is the quotient between the costs for risk minimization / repair costs caused by limit state failure.

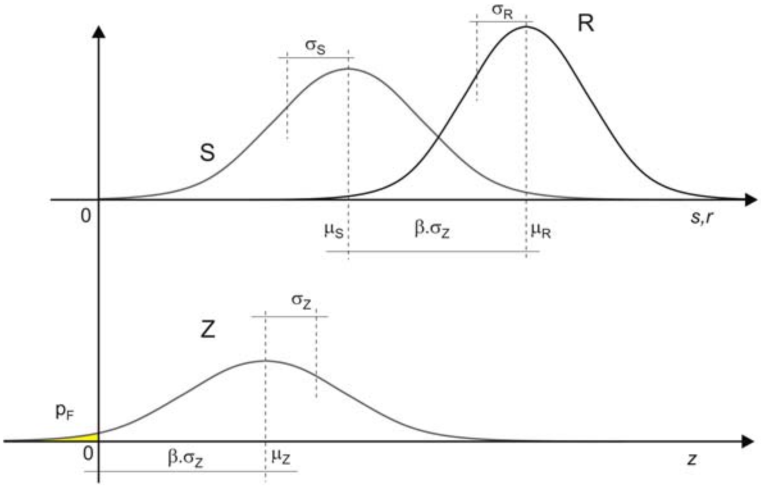


Figure 2.21 - Full-probabilistic performance concept.

Resistance *R* and load *S* are introduced in form of mean and deviation. The variable *Z* represents the reliability ($Z = R - S$) with which the component fulfils the required limit state. If *R* and *S* are both normal distributed, the variable *Z* as the sum of both is a normal distributed variable too.

$$\mu_Z = \mu_R - \mu_S \tag{2.33}$$

$$\sigma_Z = \sqrt{\sigma_R{}^2 + \sigma_S{}^2} \tag{2.34}$$

where:

μ -mean value;

σ - standard deviation.

The mean value and deviation of the reliability *Z* form the reliability index β, which is the factor by which the deviation of *Z* fits into the distance of the mean value and the axis of abscissa.

$$\beta = \frac{\mu_Z}{\sigma_Z} \tag{2.35}$$

If the reliability *Z* is normal distributed, the variable may be transformed into a standard-normal distributed one, for which $\mu = 0$ and $\sigma = 1$. The failure probability is then equal to the probability distribution function of the standard-normal distribution for the value β.

$$p_f = \Phi(-\frac{\mu_Z}{\sigma_Z}) = \Phi(-\beta) = \int_{-\infty}^{\beta} \frac{1}{\sqrt{2\pi}} \cdot e^{-\frac{1}{2}u^2} du \qquad -\infty \leq u \leq \beta \tag{2.36}$$

$\Phi(\,)$: Probability distribution function of standard normal distribution

β: Reliability index;

μ_Z: Mean value of reliability Z;

σ_Z: Deviation of reliability Z.

The cumulative frequency of the standard normal distribution can not be solved analytically, but values are tabled in current statistical literature,

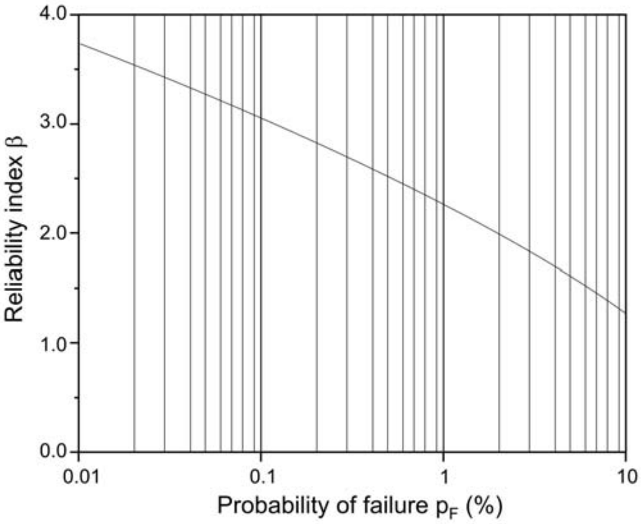


Figure 2.22 - Relationship of failure probability p_f and reliability index β for a normal distributed reliability function.

2.5 Models for Durability Design

Reinforcement corrosion in concrete is the predominant factor in the premature deterioration of reinforced concrete structures, leading to ultimate structural failure (Broomfield 1997, Schiessel 1988). Failure does not necessarily mean structural collapse, but also loss of serviceability characterized by concrete cracking, spalling, and excessive deflection of structural members. There is a clear need, from both the field of research in reinforcement corrosion in concrete structures and industrial demand, that models be developed to determine the effect of the whole corrosion process on structural performance so that the safe service life of corroded reinforced concrete structures can be predicted. Accurate prediction of structural deterioration can assist designers and owners to achieve a cost-effective strategy in the design and management of concrete structures in aggressive environments.

For service life prediction to be possible, the progress of deterioration mechanisms likely to be contributing to loss of serviceability must be modelled. The model is the set of equations and boundary conditions used to represent the mechanism. Because of the dependence of most deterioration mechanisms on transport, most models of deterioration mechanisms are essentially models of transport processes. Service life is generally inversely related to the relevant transport parameter(s) and environmental loadings. Transport parameters are defined by the properties of the concrete (Glanville 1997).

If the model is simple, it may be possible to obtain an analytical solution, involving simple substitution of values into an equation to make a prediction. If an analytical solution is not possible, a numerical solution is required, generally an iterative computer-based approach. The term modelling is used to describe selection of the model, selection and development of a solution and application of the solution to make predictions (Glanville 1997). Generally the model and solution are selected and then the corresponding transport coefficients, the loadings and other parameters required are determined. Ideally solutions should involve only known or measurable variables (Glanville 1997).

Chloride-induced reinforcement corrosion is the most widespread and costly deterioration mechanism affecting concrete structures and it is therefore this mechanism that is seen in detail.

Models vary in levels of complexity from simple analytical models assuming uniaxial diffusion into a homogeneous concrete, to more sophisticated numerical models which take account of depth and time dependent changes in concrete properties, chloride binding, leaching, etc. The approaches adopted by the different models vary considerably and consequently there can be significant variances between the solutions produced by individual models. This has caused some concern among the engineering community.

2.5.1 Deterministic and probabilistic models

Very rarely is their ever a parameter which can be defined accurately by a single value. It is intrinsic to the nature of the variables in durability design their variability, whether they are material, loading or other parameters in nature. The use of deterministic parameters in durability design can result in somewhat erroneous result, therefore with the consequent error in judgement.

Corrosion of reinforcing steel in concrete is a very complex phenomenon that involves many factors, which are not currently well understood. In addition, there are many other factors that affect the estimation of the life expectancy of a structure: type of structure and its design, structural details, materials, construction methods and quality, loads, maintenance practices, inspection routines and environmental factors, among others. All these factors have uncertainties associated with them that arise from the physical randomness of their magnitude and time of occurrence. Other uncertainties associated with the assessment of performance of deteriorating structures are statistical uncertainties, i.e., those related to limited amount of information in estimating the characteristics of a parameter, and model uncertainties, i.e., those related to the assumptions of the adopted prediction models. Because of the stochastic nature of many of the parameters that govern the performance of reinforced concrete structures, a probabilistic approach as opposed to a deterministic one is needed to generate reliable service life predictions. Modelling explicitly the various uncertainties through a probabilistic-based method leads to reliable models for the prediction of service life of reinforced concrete structures.

In order to include the uncertainty of the various parameters in a consistent manner, probabilistic modelling must be utilised. Probabilistic modelling of the deterioration mechanisms has gained strong momentum during the past few years. This opens the possibility of using service life design methods being similar to the methods behind the well established load and resistance factor design method used for structural calculations. In this way durability is treated in a scientifically sound manner and service life design becomes a rational and sound scientifically based engineering discipline.

The approach is now recognized world wide, as seen from the DuraCrete (1999). Also McGee (1999) illustrates the wide and valuable achievements already obtained using probabilistic methods to treat durability of concrete structures.

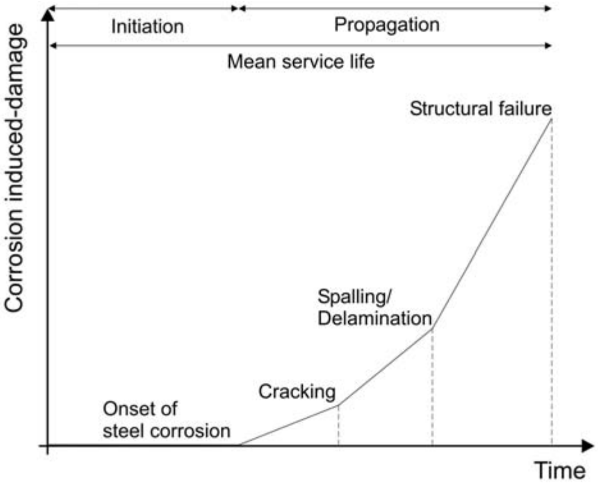


Figure 2.23 – Service life of reinforced concrete structures affected by corrosion.

Major difficulties related to the use of probabilistic modelling of the deterioration mechanisms are connected to the lack of good relevant data. In order to achieve reliable results from the computations the applied uncertainty of the various parameters must be quantified. Generally a huge amount of data is available. The challenge is to sort out the reliable part of the data and to organize these in order to use them for probabilistic calculations. Up till now there have existed very scarce data for this purpose.

2.5.2 Tuutti's degradation process for corrosion

Deterioration of structural concrete as a result of reinforcement corrosion is widely considered to be divided into two phases: initiation and propagation. The model presented by Tuutti (1982) is frequently used for representing the deterioration process, (see Figure 2.23).

The state of corrosion of steel in concrete may be expected to change as a function of time. Figure 2.23 is a modified damage build-up model first introduced by Tuutti (1982). The service life of corrosion-damaged reinforced concrete structures is idealized here as a two-phase process: an initiation stage, in which chlorides penetrate the concrete and reach the reinforcement in sufficient quantities to initiate corrosion, and a propagation stage, in which distinct levels of damage build up are attained (see Figure 2.23). The process of depassivation takes an initiation period, t_P, which is the time from construction to the time of initiation of corrosion (depassivation). The propagation phase starts from the time of depassivation, t_p, to the final state, is reached at a critical time, t_{CR}, at which corrosion would produce spalling of concrete cover or cracking through the whole of concrete cover. During the propagation period, i.e. corrosion period, t_{CORR}, which begins at the moment of depassivation, the reinforcement corrosion is usually assumed to be in a steady state. The critical time as defined above can be expressed as:

$$t_{CR} = t_P + t_{CORR} \tag{2.37}$$

Service life is defined as the time until damage accumulation reaches an unacceptable level or limit state, i.e., it is the time when the failure probability reaches an unacceptable level (which is dependent on the type of structure and failure mode). According to Figure 2.23, limit states considered for structures with reinforcement corrosion include corrosion initiation, concrete cracking, spalling and delamination of the concrete cover, and ultimately structural failure.

Regardless of the considered phase, the development of models for deterioration mechanism comprises the following steps (LIFECON 2003):

- derivation of equations representing the physical and chemical mechanisms and analytical or numerical procedures for application of these;

- sensitivity analysis to identify critical parameters;
- collection of data throughout the condition assessment of existing structures;
- evaluation of the model accuracy by comparison of predictions with the actual behaviour in the field.

Most of the models mentioned following are *diffusion models*, which means that all of them consider diffusion as the main transport mechanism, but they do not necessarily model the actual physical and chemical processes. The aim of the models is rather to describe the result of the chloride transport into the concrete.

2.5.3 Overview of initiation phase models

In the 1970’s Fick’s laws were introduced by Collepardi (1970) for describing the ingress of chlorides into concrete. The diffusion of chloride in concrete can be expressed by means of the ionic flow *J*, which is by definition positive, if the chlorides move in positive x-direction. If the concentration decreases in x-direction the flow is negative

$$J = -D \cdot \frac{\partial c}{\partial x} \qquad (2.38)$$

where,

J - ionic flow, kg/m²s;

D - diffusion coefficient, m²/s ;

c - chloride concentration in solution, kg/m³;

x - distance, m.

Equation 2.38 is known as Fick’s first law of diffusion. However, time dependency was not taken into account. To describe the diffusion process, the particle density must be function of space and time. Starting with Fick’s 1st law and considering the law of mass conservation, Fick’s second law is obtained,

$$\frac{\partial c}{\partial t} = D \cdot \frac{\partial^2 c}{\partial x^2} \qquad (2.39)$$

where t is time in seconds.

The application of Fick’s laws of diffusion to describe the transport processes in concrete is based on the simplified assumption that concrete is homogeneous, isotropic and inert (Tang 1996). The main problem with using Fick´s second law is that the chloride ions interact with the concrete and that diffusion is not the only transport mechanism working. Furthermore the movement of negatively charged ions induces the movement of positively charged ions. It is a well known fact that sodium and chloride move at different velocity (Tang 1996), which is neglected when applying

Fick's law.

Already in the 1980's it was realised that the model gave unrealistic rapid transport of chloride ions, especially when long term predictions are based on results from short term tests (Cairns *et al* 2003). As a result of this, service life predictions were often too short. The reason for this error is the assumed constant diffusion coefficient, which leads to an unrealistic high penetration depth.

Fick's law assumes that the ions move independently of each other (uncharged particles) and that the interaction with the matrix system is very weakly or non existent. However, ions move in strong electrostatic fields created by all the charged particles in the matrix. Many of the ions interact very strongly with the negative charged hydration products. Therefore, the diffusivity of the chloride ions in concrete does not follow Fick's law adequately, particularly in high density concrete with a fine pore structure

Despite this, Fick's second law is today still the theoretical background for many models which define the diffusion based chloride ingress, with adjustments made to compensate for the shortfalls of Fick's laws.

However, in simple cases, for pre-determined border conditions and with a constant diffusion coefficient, the solution for (2.39) is given by (2.40), if the border conditions are specified as $C(x,t) = C_S$ for $x = 0$ and $t > 0$, and also $C(x,t) = C_I$ for $x > 0$ and $t = 0$

$$C(x,t) = C_I + (C_S - C_I)\left[1 - erf\left(\frac{x}{2\sqrt{D \cdot t}}\right)\right] \tag{2.40}$$

where,

$C(x,t)$ - chloride content at depth x and time t, kg/m³;

C_I - initial chloride content, kg/m³;

C_S - surface chloride content, kg/m³;

erf - error function;

x - distance, m;

D - diffusion coefficient, m²/s ;

t - time, s.

In order to determine the time for the chloride content to reach a predefined concentration (usually the critical chloride concentration) at depth x, equation (2.40) can be rewritten as (2.41), considering that $C(x,t) = C_{CR}$-C_I

$$t = \left[\frac{x}{\left(2 \cdot erfc^{-1} \left(\frac{C_{CR} - C_I}{C_S - C_I} \right) \right)} \right]^2 / D \tag{2.41}$$

where,

$C(x,t)$ - chloride content at depth x and time t, kg/m³;

C_I - initial chloride content, kg/m³;

C_S - surface chloride content, kg/m³;

C_{CR} - critical chloride content, kg/m³;

erf - error function;

x - distance, m;

D - diffusion coefficient, m²/s ;

t - time, s.

The chloride diffusion coefficient, D, is an important material parameter that has been considered as a time independent parameter. Takewaka et al. (1988) proposed to describe diffusion coefficient as a power function of time. Laboratory testing (Maage *et al.* 1994; Mangat *et al.* 1994) and results from existing structures (Sandberg 1995) showed that the concrete’s age dependency of the coefficient obeys a straight line in a double logarithmic co-ordination system. This meant that the diffusion coefficient could be written as a power function

$$D(t) = D_R \cdot \left(\frac{t_R}{t} \right)^{\alpha} \tag{2.42}$$

where,

$D(t)$ - time dependant diffusion coefficient, m²/s ;

t_R - time when reference diffusion coefficient D_R was measured, s;

t - time when $D(t)$ is valid, s;

D_R - reference diffusion coefficient at time t_R, m²/s ;

α - parameter to be determined by regression analysis of test results.

In that way, the major drawback of Fick’s second law, the constant diffusion coefficient, is overcome.

In table 2.13 the α parameter values given in the Duracrete Project (1998) are presented.

The exponent α governs the rate of improvement of the diffusion coefficient over time. Research performed during the *EuroLightCon Project* (1998) states that the physical explanation for the

α parameter is two fold. The α exponent reflects the decrease of the achieved diffusion coefficient with age due to the combined effect of hydration and all other mechanisms acting *in situ* as ion exchange with seawater. Thus $\alpha = \beta + \gamma$ where β represents the effect of continued hydration of the cement and γ represents the beneficial effect on the concrete skin by being in contact with the seawater. The values of β and γ are obtained from laboratory experiments or derived from field experiences.

Table 2.13 – α parameter values (Duracrete 1998)

Binder	Environment	Normal Distribution		Beta (a=0; b=1)	
		μ	σ	q	r
OPC	Submerged	0.30	0.05	24.9	58.1
	Tidal+splash	0.37	0.07	17.2	29.3
	Atmospheric	0.65	0.07	29.5	15.9
PFA	Submerged	0.69	0.05	58.3	26.2
	Tidal+splash	0.93	0.07	1234	92.9
	Atmospheric	0.66	0.07	29.6	15.2
GGBS	Submerged	0.71	0.05	57.8	23.6
	Tidal+splash	0.80	0.07	5.8	3.87
	Atmospheric	0.85	0.07	21.3	3.75
SF	Submerged	0.62	0.05	57.8	35.4
	Tidal+splash	0.39	0.07	18.5	29.0
	Atmospheric	0.79	0.07	26	6.9

Care is advised due to the influence light weight aggregates could have on the models performance for normal weight concrete could be an issue. However, average values for α, which appear in the *EuroLightCon* (1998) report are similar to other published values (DuraCrete 1998). However, a reference normal density concrete was used in the project, and the results are consistent with other spublished.

Some researchers observed that the chloride concentration of the exposed concrete surface of a marine structure was time-dependent. On the basis of the observations it was proposed to assume that C_S increases with the square root of time (Uji *et al.* 1990). That was the most convenient choice since Crank (1986) has published a solution of Fick's second law of diffusion under that condition. This solution has been applied by Purvis (1994), Gautefall (1993) and Amey et al. (1996). However, Swamy *et al.* (1995) published a comprehensive amount of observations showing that C_S not always increases with the square root of time. From the observations made by Swamy *et al.* (1995) it is concluded, that a better model of the increase of C_S with time is a power function or the like (see Figure 2.24).

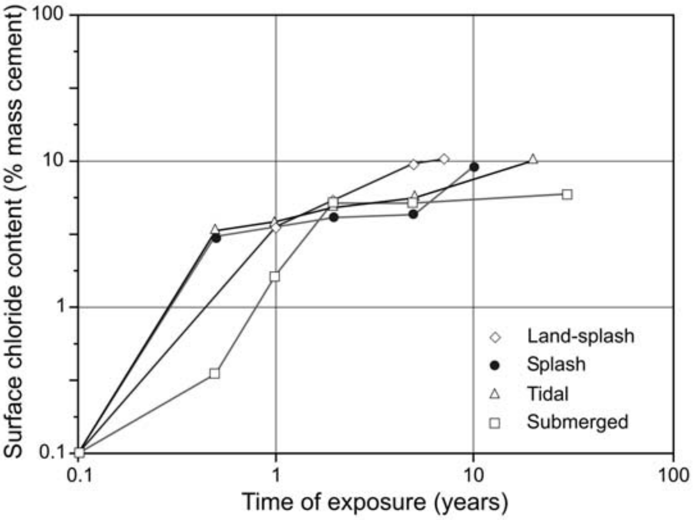


Figure 2.24 - Surface chloride concentration versus the time of exposure to seawater, Swamy et al. (1995).

The observations in figure 2.24 do not form a straight line indicating that a power function is not valid for describing the time-dependence of $C_{S.}$ Observations from many marine concrete structures and concrete specimens exposed at marine exposure stations indicate that C_S also depends on the cement (Uji 1990, Mangat 1995).

Mejlbro (1996) proposes that C_S ought to be described by the following function:

$$C_S = C_0 + S\left[\left(t - t_R\right) D_A \left(\frac{t_R}{t}\right)^{\alpha}\right]^p \tag{2.43}$$

The factor *S* depends of the chloride aggressiveness of the environment and the cement type and the composition of the binder of the concrete. The exponent *p* depends on how fast C_S increases with time, i.e. mainly on the type of binder and the environment. $D_A(t)$ is the diffusion coefficient of the concrete and is time dependant, with the same relationships as in (2.42).

By this formula the following three main conditions are fulfilled:

- C_S becomes a function of time *t* and this function proves to be flexible enough to fit the time dependency found by e.g. Swamy *et al.* (1995) and others.
- C_S becomes dependent of the properties of the concrete through $D_A(t)$, i.e. dependent of mainly the concrete’s w/c ratio and type of binder.
- there exists a simple solution of Fick's second law of diffusion and this solution could be tabulated and is easy to apply, e.g. by simple handling of spread sheet and by programming of

computers.

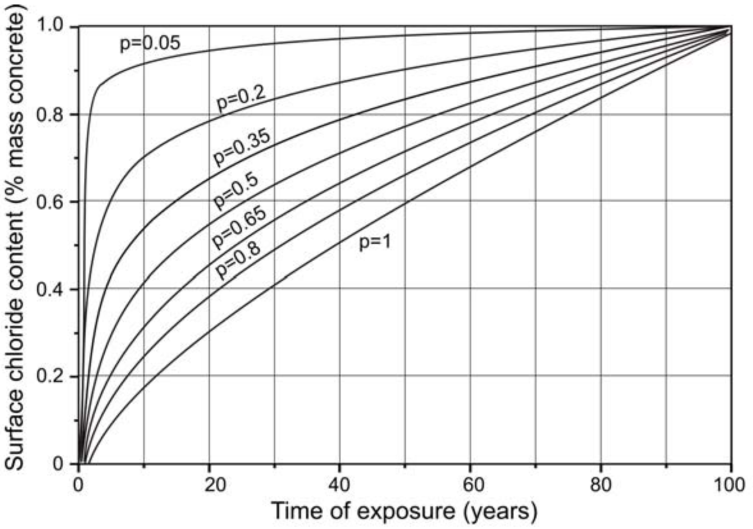


Figure 2.25 - Curves representing the flexibility of (2.43).
The values of the parameters are $\alpha=0.30$; $C_S(t=100)=1.0$; $t=0.1$year.

Figure 2.25 illustrates the flexibility of equation (2.43) for $0 < p < 1$ and C_{SA} fixed to a value of 1.0 % by mass concrete, at $t = 100$ years.

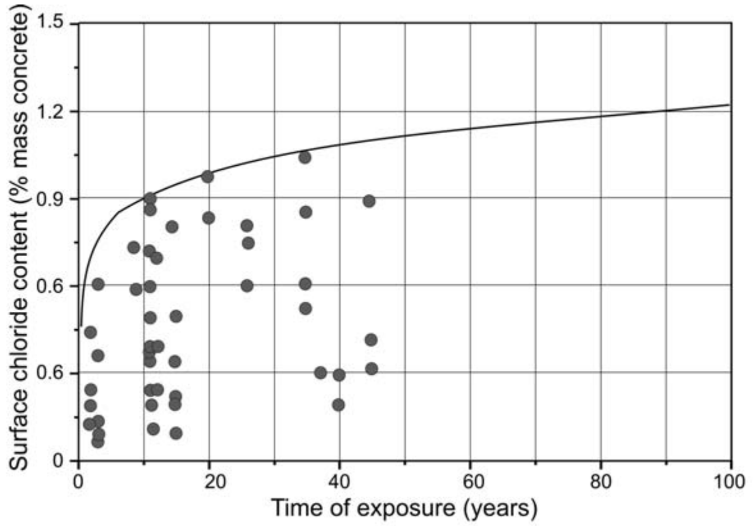


Figure 2.26 - Observed concentration of chloride in concrete surfaces in marine environments, mainly splash zones. The curve illustrates the upper boundary of C_{SA} and is based upon (2.44).

On the basis of observations made by Maage *et al.* (1995), it is concluded that the worst case in practice is covered by the following boundary conditions:

$$C_S = C_0 + 0.5\left(\left(\frac{t}{t_R}\right)^{1-\alpha} - \left(\frac{t_R}{t}\right)^{\alpha}\right)^{0.2} \qquad (2.44)$$

A series of models were developed to determine the time to initiation of corrosion based on the previously described.

The Mejlbro-Poulsen model is based on the equation

$$C(x,t) = C_0 + (C_{SA} - C_0)\,\Psi_P(z) \tag{2.45}$$

where:

$$z = \frac{x}{2\sqrt{\left(t - t_{EX}\right) D_A\left(t\right)}} \tag{2.46}$$

and, cf. Mejlbro (1996):

$$\psi_P\left(z\right) = \sum_{n=0}^{+\infty} \frac{p^{(n)}\left(2z\right)^{2n}}{\left(2n\right)!} - \frac{\Gamma\left(p+1\right)}{\Gamma\left(p+0.5\right)} \sum_{n=0}^{+\infty} \frac{\left(p-0.5\right)^{(n)}\left(2z\right)^{2n+1}}{\left(2n+1\right)!} \tag{2.47}$$

here $\Gamma(y)$ is the Gamma function defined as:

$$\Gamma\left(y\right) = \int_0^{+\infty} u^{y-1} \exp\left(-u\right) du \tag{2.48}$$

For $y \geq 0$. The notation used in (2.47) should be noted:

$$p^{(0)} = 1;\ p^{(1)} = p;\ p^{(2)} = p(p-1);\ \ldots\ p^{(n)} = p(p-1)\ldots(p-n+1) \tag{2.49}$$

where $p^{(n)}$ has $n \geq 1$ factors. Figure 2.27 shows the graphs of Mejlbros's Ψ-functions.

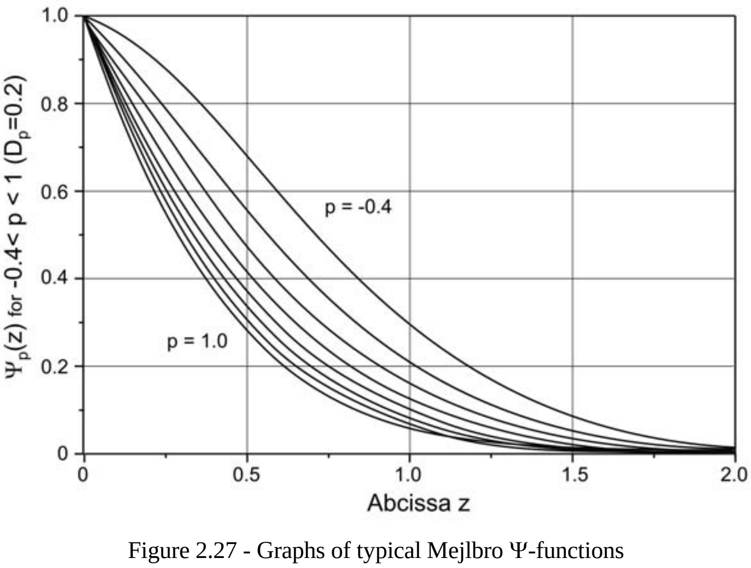


Figure 2.27 - Graphs of typical Mejlbro Ψ-functions

An advantage of the Mejlbro-Poulsen Model is the direct applicability of measured chloride profiles from field exposure. If sampled and measured in a sufficiently systematically and reproducible way, information of the chloride profiles with respect to lifetime predictions can be extracted and used in the model (HETEK 1997). This frees the model from assumptions regarding the concretes response to the environmental load.

However, it does not model the actual physical and chemical processes involved in the chloride ingress into concrete. The aim of the model is to describe the *result* of these physical and chemical processes of this transport, i.e. the chloride profiles (HETEK 1997). The model is dependent on systematic measurements from the environment and the concrete in question. The measurements must represent a fairly long time of exposure to improve predictions. Another drawback is the models sensitivity to the quality of the measurements. Many of the existing measurements must be regarded as unsuitable for prediction purposes with this model (HETEK 1997). These are drawbacks, for durability design using this model, which must be overcome with time when the experiences increases. Thus the experiences for the time being are limiting the application of this model.

The DuraCrete model takes the probabilistic nature of the environmental aggressiveness and the material properties into account by applying partial safety factors for the load and resistance variables. It is an empirical model, which also has Fick's second law of diffusion as a theoretical background. The DuraCrete Model is presented in a report of the DuraCrete Project (1998).

The design equation, *g,* stating that corrosion is initiated when the chloride concentration around the reinforcement exceeds the critical chloride concentration is given by

$$c^d_{cr} - c^d(x,t) = c^d_{cr} - c^d_{s,cl} \cdot \left[1 - erf\left(x^d / \left(2 \cdot \sqrt{\frac{t}{R^d_{cl}(t)}}\right)\right)\right] \tag{2.50}$$

where

$c^d{}_{CR}$ - design value of the critical chloride concentration;

$c^d{}_{S,CL}$ - design value of the surface chloride concentration;

x^d - design value of the cover thickness;

$R^d{}_{CL}$ - design value of the chloride resistance;

t - time.

The design value of the critical chloride concentration can be found as

$$c^d_{CR} = c^c_{CR} \cdot \frac{1}{\gamma_{CR}} \tag{2.51}$$

where γ_{CR} is the partial factor of the critical chloride concentration.

The design value of the surface chloride concentration is determined from the expression

$$c^d_{S.Cl} = Ac_{S.Cl} \cdot \left(w/b\right) \cdot \gamma_{cS.Cl} \tag{2.52}$$

where $A_{Cs,cl}$ is a regression parameter describing the relation between the chloride surface concentration and the w/c ratio, and where $\gamma_{Cs,cl}$ is the partial factor for the surface concentration.

The design value of the cover thickness is found from

$$x^d = x^C - \Delta x \tag{2.53}$$

where Δx is the margin for the cover thickness.

Finally, the design value of the time dependant chloride resistance is derived from

$$R_{Cl}^d(t) = \frac{R_{Cl,0}^c}{k_{e,Cl}^c \cdot k_{c,Cl}^c \cdot \left(\frac{t_0}{t}\right)^{n_{cl}} \cdot \gamma_{RCl}} \tag{2.54}$$

where

$R_{CL,0}$ - resistance with respect to the chloride ingress determined on the basis of compliance tests;

$k_{e,cl}$ - environment factor;

$k_{c,cl}$ - curing factor;

n_{cl} - age factor;

t_0 - age of the concrete, when compliance test is performed;

γ_{Rcl} - partial factor for the resistance with respect to chloride ingress.

By the evaluation of the design value of the chloride resistance also the effect of the temperature should be taken into account.

An apparent drawback of the DuraCrete model is the large need for data to be able to quantify all the variables adequately for a specific kind of environment.

The modelling software Life-365 (2001) uses the following relationship to account for temperature dependent changes in diffusion:

$$D(T) = D_{ref} \cdot \exp\left[\frac{U}{R} \cdot \left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right] \tag{2.55}$$

where:

$D(T)$ - diffusion coefficient at time t and temperature T;

D_{ref} - diffusion coefficient at some reference time t_{ref} and temp. T_{ref};

U - activation energy of the diffusion process (35000 J/mol);

R - gas constant;

T - absolute temperature.

In the model t_{ref} = 28 days and T_{ref} = 293K (20°C). The temperature T of the concrete varies with time according to the geographic location selected by the user. In this way, the effect of the temperature on the diffusion coefficient is taken into account.

2.5.4 Overview of propagation phase models

The propagation time is the time from corrosion initiation until a specified level of corrosion induced damage state is attained. Depending on the type of structure, failure mode and consequence, the time to onset of different damage levels such as longitudinal cracking, spalling/delamination of the concrete cover and a maximum acceptable damage (e.g., flexure or shear failure) can be adopted as propagation times, as illustrated in Figure 2.23.

Only in recent years has research been performed into the modelling of the propagation phase. Due to the novelty of the research area, phenomenon involved are still not understood and therefore no models offer a comprehensive explanation for the degradation process. However, in the following text, some recent models for the different stages of the propagation phase are described.

After initiation of corrosion, the reinforcement corrosion propagates in concrete and produces expansive rusts, which increases the pressure (i.e., stress) at the interface of the reinforcement and concrete. With the propagation of corrosion, its products (mainly ferrous and ferric hydroxides, $Fe(OH)_2$ and $Fe(OH)_3$) occupy much greater volume than the original reinforcement (i.e., iron), thereby generating pressures on the surrounding concrete. The pressure builds up to a level that causes internal concrete cracking at the interface of the reinforcement and concrete. The crack eventually extends through the concrete cover. Eventually the stress due to the expansive causes concrete to crack. Based on this phenomenon, the probability of corrosion induced concrete cracking $p_C(t)$ at time t can be considered as follows:

$$p_C(t) = P[\sigma_C(t) \geq \sigma_T] \qquad (2.56)$$

where $\sigma_C(t)$ is the stress asserted by the expansive corrosion products and σ_T, is the minimum stress required to cause the cracking of concrete cover. At the time that $p_C(t)$ is greater than a minimum acceptable probability, $p_{C,A}$, the time to cracking, T_C , can be determined, i.e.,

$$p_C(T_C) \geq p_{C,A} \qquad (2.57)$$

where $p_{C,A}$ represents the reliability of the prediction. Therefore, equation (2.57) can be used to determine the time to surface cracking of corrosion affected reinforced concrete structures.

Li (2003) suggested a model where the reinforced concrete is modelled as thick-wall cylinder under internal radial pressure. The schematic representation of the process of corrosion induced cracking is shown in Figure 2.28 a), where D is the diameter of reinforcement bar, d_0 is thickness of the annular layer of corrosion products (i.e., a pore band) at the interface of the reinforcement and concrete, *a* and *b* are inner and outer radii of the thick-wall concrete cylinder. *C* is the concrete cover. As corrosion progresses, the rust products will fill the pore band completely and then generate an expansion pressure of the concrete (figure 2.28 b)). Thereafter the inner radius will increase as corrosion grows (i.e., the thickness of rust product) therefore the inner radius can be expressed as $a(t) = [(D + 2d_0)/2] + d_S(t)$, where $d_S(t)$ is the thickness of corrosion products needed to generate tensile stress and it can be expressed as (Li 2003):

$$d_S(t) = \frac{W_{rust}}{\pi(D+2d_0)}\left(\frac{1}{\rho_{rust}} - \frac{\alpha_r}{\rho_{st}}\right) \tag{2.58}$$

where

α_r is a coefficient related to the type of rust products,

ρ_{rust} is the density of corrosion products,

ρ_{st} is the density of the reinforcement,

W_{rust} is the mass of corrosion products (rust) that generate the critical tensile stress.

Generally, not all corrosion products will initiate the critical tensile stress, however, it is assumed in this model that once the rust completely fills the pore band between concrete/steel, the continuing growth of the rust will generate the tensile stress. W_{rust} is related to corrosion rate as measured by corrosion current density I_{CORR} in μA/cm^2 and can be determined by (Liu et al 1998):

$$W_{rust} = \left(s\int_0^t 0.098\frac{1}{\alpha_r}\pi D\cdot I_{CORR}.dt \right)^{1/2} \tag{2.59}$$

From the test results (Li 2003), the corrosion current density, I_{CORR} can be expressed, for reinforced concrete flexural members as:

$$I_{CORR} = 0.3683\, Ln(t) + 1.1305 \tag{2.60}$$

where *t* is the time in years.

The trend of I_{CORR} is in good agreement with other tests (Schiessl 1988, Andrade et al. 1990). In practical application, the values of I_{CORR} should be obtained from the structure of concern. Based on the theory of elasticity (Timoshenko et al 1970, Ugural 1986), the expansive stress can be derived as follows:

$$\sigma_C(t) = P_1(t) = \frac{2E_{ef}d_S(t)}{(D+2d_0)\left(\frac{b^2+a^2(t)}{b^2-a^2(t)} + v_C\right)} \tag{2.61}$$

where E_{ef} is an effective elastic modulus of concrete and v_C is Poisson's ratio for concrete.

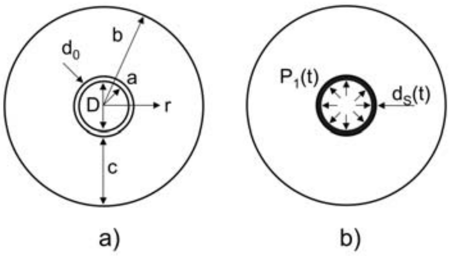


Figure 2.28 – Schematic representation of the parameters used in concrete cracking models (Li 2003).

The minimum stress required to induce cracking of concrete cover is apparently related to the tensile strength of concrete and the thickness of the cover. Based on the mechanism of corrosion induced concrete cracking, the volume of the corrosion products is also related to the thickness of the pore band at the interface of the reinforcement and concrete. According to Bažant (1979) and Liu and Weyers (1998), σ_T can be expressed as:

$$\sigma_T = \frac{2c \cdot f_t}{D + 2d_0} \tag{2.62}$$

where f_t is the tensile strength of the concrete.

It has been generally assumed that one of the factors influencing the severity of corrosion is the crack width. It is therefore a basic design criterion prescribed in all building codes. When the corrosion continues, cracks progressively widen with increasing corrosion and delamination of the concrete cover eventually results. Based on this phenomenon, the probability of corrosion induced concrete crack width $p_{CR}(t)$ at time t can be determined as follows (Li 2003)

$$p_{CR}(t) = P[w(t) \geq w_{CR}] \tag{2.63}$$

where $w(t)$ is the crack width at the concrete surface by the expansive corrosion products and w_{CR} is the limit crack width of concrete cover according to the building code. At the time that $p_{CR}(t)$ is greater than a minimum acceptable probability, $p_{CR,A}$ the time to limit crack width, T_{CR}, can be determined, i.e.,

$$p_{CR}(T_{CR}) \geq p_{CR,A} \tag{2.64}$$

Again $p_{CR,A}$ represents the reliability of the prediction as in the case of time to cracking. Therefore, equation (2.64) can be used to determine the crack limit lifetime of corrosion affected reinforced concrete structures.

With the progress of corrosion the surface crack extends to much wider cracks, which causes concrete spalling and mechanical damage, e.g., deflection and failures. So it is necessary to limit the crack width. The maximum permissible crack width according to the British Standard (BS 8110) is taken to be 0.3 mm.

To assess the mechanical damage resulting from the expansion of the corrosion products around the corroding reinforcing bars, an equivalent uniform internal pressure is applied around the steel/concrete interface, and the resulting state of stress in the surrounding concrete may be evaluated by means of an elastic analysis (Tepfers 1979, Martín-Pérez et al 1999). In this approach, the concrete cover is treated as a thick-wall cylinder subjected to the internal pressure build-up of expansive corrosion products, as shown in Figure 2.29.

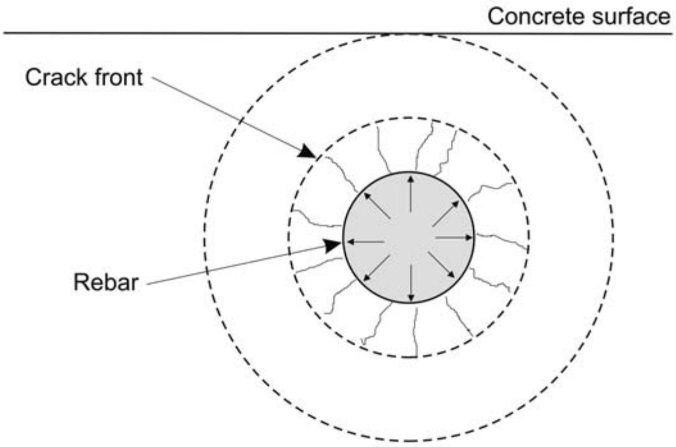
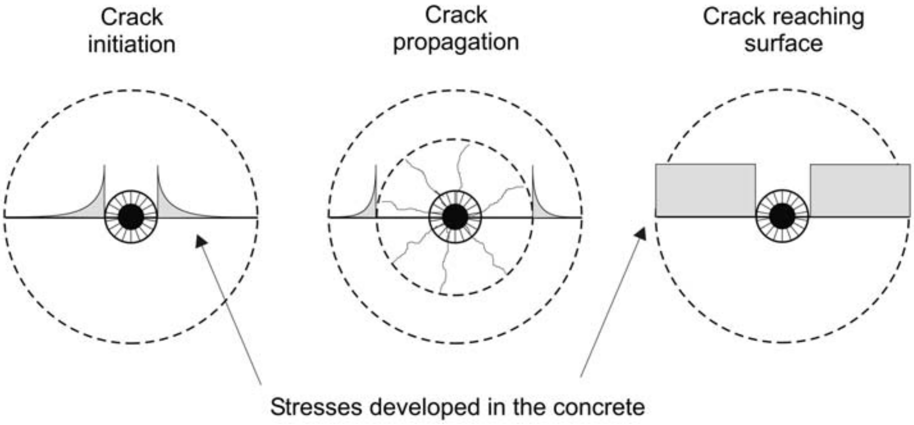


Figure 2.29 – A thick-wall concrete cylinder subject to accumulation of corrosion product.



$$\frac{\sigma_C}{f_t} = \frac{(c/d)^2 + (c/d)}{(c/d)^2 + (c/d) + 0.5} \qquad \frac{\sigma_C}{f_t} = \frac{0.5 + (c/d)}{1.665} \qquad \frac{\sigma_C}{f_t} = 2(c/d + 0.5)$$

Figure 2.30 – Models for crack initiation and propagation through concrete cover.

The times to onset of cracking and spalling are determined from the times at which the developed hoop stresses of the thick-wall cylinder exceed the resistance of the cover to cracking and spalling, respectively, as given by the models in Figure 2.30, where σ_C is the internal pressure induced by the corrosion products, f_t, is the concrete tensile strength, c is the concrete cover, and d is the rebar diameter. The length of these two times depend on the corrosion rate, fracture properties of concrete, cover-to-bar diameter ratio, bar spacing and on the environmental and mechanical load.

There are many mechanisms through which corrosion may impair the serviceability and safety of reinforced concrete structures. The residual capacity of these structures is affected by both the loss of concrete (through cracking and spalling) and reinforcing steel cross-section, and by the loss of bond between the two. To model the ultimate limit state corresponding to structural failure, residual strength functions are formulated as $R(t)=\alpha(t)R_0$, where $\alpha(t)$ is a degradation function independent of the loading history but dependent on the rate of corrosion of the reinforcing steel and R_0 is the 'undamaged' resistance function. A reliability-based assessment of the propagation period is then possible through the modelling of both the load and resistance as stochastic processes (Lounis et al 2001).

In the DuraCrete model (1998) for cracking and spalling, the design rules given ensure that the probability of observing a crack with a critical crack width larger than 1.0 mm is acceptable. Spalling is assumed to occur when the crack width exceeds the critical limit of 1.0 mm. However, it should be noted that this limit does not represent an event where the concrete cover is lost. It represents a situation where the concrete cover no longer can be considered to contribute to the load carrying capacity of the structure.

It is required that the crack width, w, of a given structure must not exceed some critical limit, w_{CR}, i.e. the limit state can be written

$$g(x) = w_{CR} - w^d \tag{2.65}$$

The design value of the actual crack width, w^d, can be estimated on the basis of the following expression determined by regression analysis:

$$w^d = \begin{cases} w_0 & p^d \le p_0^d \\ w_0 + b^d\left(p^d - p_0^d\right) & p^d > p_0^d \end{cases} \tag{2.66}$$

where

w_0 - width of the initial visible crack;

b^d - the design value of a parameter depending on the position of the bar;

p^d - the design value of the corrosion penetration in microns;

p^d_0 - the design value of the corrosion penetration necessary to produce a crack in microns.

The design value of the corrosion penetration, p^d_0, needed for initiation of a crack can be determined on the basis of the following expression:

$$p_0^d = a_1 + a_2\frac{x^d}{d} + a_3 f_{C,SP}^d \tag{2.67}$$

where

a_1, a_2, a_3 - regression parameters;

x^d - design value for cover thickness;

d - diameter of reinforcement bar;

$f^d_{C,SP}$ - design value of tensile splitting strength in MPa.

The actual attack penetration, p^d, can be determined as:

$$p^d = \begin{cases} 0 & t \le t_i^d \\ V^d w_t \left(t - t_i^d\right) & t > t_i^d \end{cases} \qquad (2.68)$$

where

V^d - the design value of the corrosion rate;

w_t - the relative time of wetness;

t^d_i - the design value of the time to initiation of corrosion.

There are numerous parameters related with the concrete and the environment which influence the corrosion rate. The most important factor affecting the corrosion rate of depassivated reinforcement is the electrolytic resistivity of the concrete. This is, in turn, influenced by the mix composition and the moisture content of the concrete. Andrade and Arteaga (1997) have proposed a parametric equation relating corrosion rate and resistivity. The influence of other important factors such as the nature of depassivating species (carbon dioxide, chlorides), macro galvanic effects, formed rust and oxygen availability, are taken into account by introduction of a member of correction factors. In the proposal of Andrade and Arteaga, the moisture content is related to the corrosion rate (or alternatively to the resistivity) by using average values for each of the exposure classes.

A supplementary approach, modelling the material and environmental influence of the development of the electrolytic resistivity has been presented by Nilsson and Gehlen (1998), who propose as well a parametric expression for the resistivity itself. This parametric equation is based in the relative humidity, temperature and chloride content.

The DuraCrete Model adopts the global approach of Andrade and Arteaga, supplemented by the approach of Nilsson and Gehlen (parametric expression for the electrolytic resistivity). Corrosion only occurs if sufficient oxygen and water is present. If these conditions are not fulfilled the rate of corrosion can be assumed to be negligible. Otherwise, the design value of the corrosion rate is given by:

$$V^d = \frac{m_0}{\rho^C} \alpha^C \cdot F^C_{CL} \cdot \gamma_V \qquad (2.69)$$

where

m_0 - constant for corrosion rate versus resistivity;

F^C_{CL} - the characteristic value of the chloride corrosion rate factor;

α^C - the characteristic value of the pitting factor;

ρ^C - the characteristic value of the resistivity;

γ_V - the partial factor for the corrosion rate.

The characteristic value of the resistivity, ρ^C, is given by:

$$\rho^C = \rho_0^C \left(\frac{t_{hydr}}{t_0} \right)^{n_{res}^C} \cdot k_{C,RES}^C \cdot k_{T,RES}^C \cdot k_{RH,RES}^C \cdot k_{Cl,RES}^C \qquad (2.70)$$

where

p^C_0 - the characteristic value of the potential electrolytical resistivity;

t_0 - the age of the concrete at the time when the compliance test is performed;

t_{hydr} - the age of the concrete, maximum value one year;

n^c_{res} - the age factor for the electrolytical resistivity;

$k^c_{c,res}$ - the characteristic value of the curing factor for the resistivity;

$k^c_{T,res}$ - the characteristic value of the temperature factor for the resistivity;

$k^c_{RH,res}$ - the characteristic value of the humidity factor for the resistivity;

$k^c_{cl,res}$ - the characteristic value of the factor accounting for the presence of chloride.

The characteristic value of the temperature factor for the electrolytical resistivity is given by

$$k_{T,RES}^C = \frac{1}{1 + K^C \left(T - 20 \right)} \qquad (2.71)$$

where K^c is the characteristic value of a factor describing the temperature dependency of the conductivity and T is the temperature in °C.

The design value of the cover thickness is given by

$$x^d = x^c - \Delta x \qquad (2.72)$$

where the values of Δx depend on the quality of the workmanship.

The design value of the parameter, *b*, depending on the location of the considered reinforcement bar is determined by

$$b^d = b^c - \gamma_b \qquad (2.73)$$

where b^d and b^c are the design value and the characteristic value of the parameter, respectively, and γ_b is the partial factor for b. Finally, the design value of the splitting tensile strength is defined as the characteristic value.

Bazant's model (1979) for the prediction of corrosion damage considers the volume expansion due to the formation of hydrated red rust, ($Fe(OH)_3$), over the residual reinforcement core. When corrosion is in a steady state with a constant rate, the unacceptable deformation of concrete at cracking can be related to the duration of steady-sate corrosion by

$$t_{CORR} = \rho_{CORR} \frac{D \cdot \Delta D}{pJ_r} \tag{2.74}$$

where

ρ_{CORR} - is combined density factor for steel and rust,

D - is diameter of rebar,

p - is perimeter of rebar,

J_r - is instantaneous corrosion rate,

ΔD - is increase in diameter of reinforcement due to rust formation,

t_{CORR} - is steady-state corrosion period.

Morinaga (1990) suggested that the duration of steady state corrosion, t_{CORR} corresponding to Q_{CR} can be obtained as

$$t_{CORR} = \frac{Q_{CR}}{J_r} \tag{2.75}$$

where

Q_{CR} - is amount of corrosion when concrete cracks (x10^{-4} gm/cm^2).

The expression to estimate Q_{CR} is given by

$$Q_{CR} = 0.602\left(1+\frac{2Cv}{D}\right)^{0.85} \cdot D \tag{2.76}$$

where

Cv - protecting concrete thickness (mm);

D - diameter of reinforcement (mm).

2.6 Current codes of practice

Social and economic developments have caused the construction industry to focus its attention increasingly on the durability of constructions and their components. As a consequence, there are currently several developments aimed at predicting the useful life of construction and their components. These developments are based on research into degradation properties of concrete. The relevance of this research, however, is determined by the relationship of these properties to the requirements imposed on construction by owners or by society. Since this relationship is complex, current durability research often gives it too little consideration. On the other hand, an understanding of this relationship will enable us to add durability as a relevant and controlled quality to the design of a construction and their maintenance.

As a result of the pressure of economic developments, owners are paying more and more attention to the costs of maintenance, in addition to the initial construction costs of a building.

The user requirements for a construction must firstly be formulated as quantifiable and verifiable performance requirements. In this, durability is currently dealt with somehow like a nuisance. In the area of products and materials, project requirements often limit themselves to stating features which cannot be quantified, such as: *resistant to wear*, *corrosion proof*, *durable*, *maintenance friendly*, *easy to clean*, and *easy to replace and repair*. Also reference is frequently made to product standards or directives. In the best case, conditions are also given for the costs of repair, maintenance and cleaning, sometimes given as a percentage of the construction costs. The unqualified requirements provide a designer with little to hold onto in achieving quality (Hoogenboom 1995).

In principle three *design tools* are currently used to integrate durability into a design:

- standards and directives;
- own experience;
- reference data.

Since durability is almost always implicitly integrated into standards, a designer has no proper insight into how effective alternative design measures will be. In general, a well founded opinion on achievable lifetime cannot currently be obtained either from standards or from other publications. If such an opinion on a certain building component is still required, one has to fall back on experience with similar building components. As a result of continuing specialisation in construction - designers are becoming less and less involved in both the completion and maintenance of buildings - a designer normally does not have this experience and has little chance of gaining it. He will therefore have to use reference data. Access to this is predominantly poor and difficult to use for the objective in question. Improvement of the design tools is therefore also a requirement for improving design quality in relation to durability.

2.6.1 Standards for durability design - prescriptive versus performance based standards

Probabilistic service life design is presently on the agenda within the concrete community worldwide. As the technology and philosophy on the subject gradually mature, it is imperative that such an approach can be anchored in the standards and thus being made available to the construction industry.

Specifications, or standards, are the basis to verify quality and maintain product uniformity. Conformance to standards can be determined using either prescriptive or performance specifications. Concrete codes are largely prescriptive in nature. That is, they provide specific details about how to achieve safe design. Prescriptive standards are easy to apply, because they are simplified approaches which generally apply to many, but not all, situations. They also have limitations, including tendency to result in, at best, suboptimal (overly conservative) designs, and in some instances, potentially unsafe designs (Fluge 2001).

The performance standard approach allows the engineer to optimize the design, and potentially achieve a cost benefit, without sacrificing overall safety. With the computing capability available to engineers today, there is little to be gained from short-cut methods and the potential for losses from designing inadequate protection.

Another comparison possible between both types of codes is that the *prescriptive* approach describes an acceptable solution while *a performance* approach describes the required performance of the solution. The performance approach is thinking and working in terms of the *ends* rather than the *means*. Performance-based building standards have the advantage of removing barriers to innovation and permitting cost-optimization. Transitioning from prescriptive to performance-based standards is not always easy; it requires much thought and experience to design test protocols to evaluate the materials under conditions that are applicable to the real world. Also, a clear understanding of the choice of materials is not always easily obtained, which further complicates the performance approach. However, prescriptive standards are also affected by the choice of materials and whether or not the manufacturer and/or builder has a clear understanding of their application. The difference between the two is that performance-based standards call for testing and verification of a specific material or assembly performance, while prescriptive standards assume testing and verification has already occurred, which does not always guarantee proper implementation in the field.

Performance standard considerations should include (Bukowski 1996):

- The required performance, not a method or material to achieve that performance. There can be degrees of prescription in performance. Performance does not have to be absolute. There must be prescriptive solutions if performance cannot be described;
- Performance-based standards must have referenced test methods, acceptable means of calculating performance, or a combination of both;

- Performance requirements must not compromise other attributes of a structure.

As the world transitions to performance codes, standards will still be needed to provide the detail of how to meet the intent of the code. However, since the form of the code has changed, the form of the standard should be re-thought so as to best complement the performance oriented nature of the system. Specifically, what form should a standard take relative to that generally agreed for a performance code?

Since the code is built around explicit objectives, the standard needs to clearly state its intent regarding the purpose of the system or feature covered. Thus, the form of a performance standard is first, to explicitly state its purposes and second, to provide the means to establish quantitatively what constitutes meeting those purposes. In addition, any quantitative measure of the ability of the system to fulfil its purpose needs to include the reliability of systems. This refers to the likelihood that such protective systems will perform as intended when called upon to do so. Like the code, the performance standard will have "deemed to satisfy" provisions which describe arrangements which are known to meet the intent. Current prescriptive standards are likely to comprise early "Approved Documents" for standards as well.

2.6.2 European codes for concrete structures

In 1975 the EU Commission initiated an action plan in the field of construction to eliminate trade barriers and to harmonize technical specifications. In 1989 EU and EFTA decided to transfer this work to CEN (European Committee for Standardization). Through a series of mandates, the Eurocodes then got the status of European Standards (EN). The Eurocodes became then linked to the EU Council Directive 89/106/EEC on construction products and 93/37/EEC, 92/50/EEC, 89/440/EEC on public works and services (Helland 2001).

For concrete related construction, three Eurocodes are essential:

- EN 1990 – Eurocode: Basis of Structural Design
- EN 1991 – Eurocode 1: Actions on Structures
- EN 1992 – Eurocode 2: Design of Concrete Structures

EN 1990 describes the principles and requirements for safety, serviceability and durability of structures. It is based on the limit state concept used in conjunction with a partial factor method. EN 1990 also gives guidelines for the aspects of structural reliability relating to safety, serviceability and durability. EN 1990 implemented in the various European countries will be supplemented with “National Annexes” giving the exact “Nationally Determined Parameters” defining the required level of reliability. These levels of reliability are normally linked to national law.

The hierarchy of documents will then be that (see Figure 2.31):

1. EN 1990 “Basis of structural design” defines the general (not material specific) overall level of reliability to fulfill the requirements of the society;
2. EN 1992 “Design of Concrete Structures” gives the material (concrete) specific “basis of design”. The EN 1992 gives principles and application rules for the verification of the fulfillment of EN 1990 requirements. It states that “*the design procedures are valid only when requirements for execution and workmanship given in ENV 13670 are also complied with”;*
3. ENV 13670 “Execution of Concrete Structures” states that *“concrete shall be specified and produced according to EN 206” – “Reinforcing steel shall conform to EN 10080” – etc;*
4. EN 206 states that *“general suitability is established for cement conforming to prEN 197” – “Air content shall be measured according to prEN 12350” – etc;*

In June 2000 the new European Standard EN 206-1 “Concrete – Part 1: Specification, performance, production and conformity” was launched. To get the document operational in the various European Countries, a “National Annex” for each nation had to be issued.

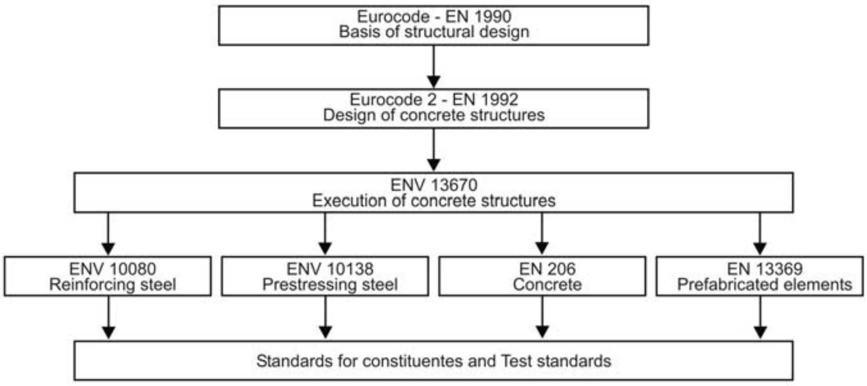


Figure 2.31 Code hierarchy for concrete constructions (Helland 2001).

The EN 206 is made up of two parts – the text of the common European document (EN 206-1:2000) and a national annex. The annex sets out, clause by clause, additional information and guidance for practitioners to complement the text of the common document. It comprises provisions depending on geography and well-established regional traditions and experience, but also where it was not practical to achieve European consensus. Among these provisions are the limiting values for concrete composition and related requirements to the cover to the reinforcement in the design codes to ensure the design working life of the structure.

2.6.3 Requirements for durability design

Various concrete durability requirements present in current standards from an European as well as a national level will be presented. Special attention will be given to Portuguese and Norwegian standards, as the present work has been carried out in these countries.

2.6.3.1 Requirements of European Codes for concrete

The EN 1992 - Eurocode 2 and the EN 206-1 specify the (prescriptive) requirements to obtain concrete durability. A brief presentation of the main requirements is given.

From the Eurocode 2, exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions. The environmental conditions are classified according to Table 2.14, based on EN 206-1. In addition to the conditions in Table 2.14, particular forms of aggressive or indirect action should be considered including:

- chemical attack, arising from e.g.
    - the use of the building or the structure (storage of liquids, etc)
    - solutions of acids or sulphate salts (EN 206-1, ISO 9690)
    - chlorides contained in the concrete (EN 206-1)
    - alkali-aggregate reactions (EN 206-1, National Standards)
- physical attack, arising from e.g.
    - temperature change
    - abrasion
    - water penetration (EN 206-1).

In the EN 206-1, environmental actions are classified as exposure classes in Table 2.14. The various exposure classes are split into six groups covering:

- No risk of corrosion;
- Carbonation-induced corrosion;
- Chloride-induced corrosion resulting primarily from deicing salts;
- Chloride-induced corrosion resulting from seawater exposure;
- Freeze-thaw attack;
- Chemical attack.

The exposure classes to be selected depend on the provisions valid in the place of use of the concrete. This exposure classification does not exclude consideration of special conditions existing in the place of use of the concrete or the application of protective measures such as the use of stainless steel or other corrosion resistant metal and the use of protective coatings for the concrete or the reinforcement. The concrete may be subject to more than one of the actions described in Table 2.14 and the environmental conditions to which it is subjected may thus need to be expressed as a combination of exposure classes.

Table 2.14 - Exposure Classes for concrete structures according to EN 206-1:2000.

Class designation	Description of the environment	Informative examples where exposure classes may occur
1. No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: All exposures except where there is freeze/thaw, abrasion or chemical attack. For concrete with reinforcement or embedded metal: Very dry	Concrete inside buildings with very low air humidity.
2. Corrosion induced by carbonation		
Where concrete containing reinforcement or other embedded metal is exposed to air and moisture, the exposure shall be classified ad follows		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity. Concrete permanently submerged in water.
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact. Many foundations.
XC3	Moderate humidity	Concrete inside building with moderate or high air humidity. External concrete sheltered from rain.
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2.
3. Corrosion induced by chlorides other than from see water		
Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from sea water, the exposure shall be classified as follows:		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides.
XD2	Wet, rarely dry	Swimming pools. Concrete exposed to industrial waters containing chlorides.
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides. Pavements Car park slabs.
4. Corrosion induced by chlorides from sea water		
Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water, the exposure shall be classified as follows:		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast.
XS2	Permanently submerged	Parts of marine structures.
XS3	Tidal, splash and spray zones	Parts of marine structures.
5. Freeze/thaw attack		
Where concrete is exposed to significant attack by freeze/thaw cycles whilst wet, the exposure shall be classified as follows:		

XF1	Moderate water saturation, without deicing agent	Vertical concrete surfaces exposed to rain and freezing.
XF2	Moderate water saturation, with deicing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne deicing agents.
XF3	High water saturation, without deicing agent	Horizontal concrete surfaces exposed to rain and freezing.
XF4	High water saturation, with deicing agent or sea water	Road and bridge decks exposed.
6. Chemical attack		
Where concrete is exposed to chemical attack, which occurs in natural soils and ground water as given in Table 2, the exposure shall be classified as given below. The classification of seawater depends on the geographical location; the classification valid in the place of use of the concrete applies.		
XA1	Slightly aggressive chemical environment according to Table 2	
XA2	Moderately aggressive chemical environment according to Table 2 or exposure to sea water	
XA3	Highly aggressive chemical environment according to Table 2	
Note: The composition of the concrete affects both the protection of the reinforcement and the resistance of the concrete to attack. Annex E gives indicative classes for the particular environmental exposures. This may lead to the choice of higher strength than required for the structural design. In such cases the value of f_{ctm} should be associated with the higher strength in the calculation of minimum reinforcement and crack width control.		

The informative Annex E of the Eurocode 2 (prEN 1992.1-1 2002) indicates strength classes for durability. The choice of adequately durable concrete for corrosion protection of reinforcement and protection of concrete attack, requires consideration of the composition of concrete. This may result in a higher compressive strength of the concrete than is required for structural design. The relationship between concrete strength classes and environmental classes (see Table 2.15) may be described by indicative strength classes. Values of indicative strength classes for use in a country may be found in its National Annex.

Table 2.15 - Indicative strength classes (Exposure Classes according to Table 2.14) (prEN 1992.1-1 2002)

Corrosion										
	Carbonation-induced corrosion				Chloride-induced corrosion			Chloride-induced corrosion from sea water		
	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
ISC*	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	

Damage to Concrete							
	No risk	Freeze/Thaw Attack			Chemical Attack		
	X0	XF1	XF2	XF3	XA1	XA2	XA3
ISC*	C12/15	C30/37	C25/30	C30/37	C30/37		C35/45

** Indicative strength class*

The informative Annex F of the EN 206-1 recommends for limiting values of concrete composition. This annex provides recommendations for the choice of the limiting values of concrete composition and properties in relation to exposure classes according to Table 2.14.

The values in Table 2.16 are based on the assumption of an intended working life of the structure of 50 years. The values in Table 2.16 refer to the use of cement type CEM I conforming to EN 197-1 and aggregate with maximum nominal size in the range of 20 mm to 32 mm. The minimum strength classes were derived from the relationship between w/c ratio and the strength class of concrete made with cement of strength Class 32.5. The limiting values for the maximum w/c ratio and the minimum cement content apply in all cases, whilst the requirements for concrete strength class may be additionally specified.

The Eurocode 2 specifies for concrete cover to reinforcement that it should be established by considering its intended use, service life (see EN 1990), maintenance programme and actions. Corrosion protection of steel reinforcement depends on density, quality and thickness of concrete cover and cracking. The cover density and quality is achieved by controlling the maximum w/c ratio and minimum cement content (see EN 206-1) and may be related to a minimum strength class of concrete. Further information is given in Annex E.

The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface.

The nominal cover shall be specified on the drawings. It is defined as a minimum cover, c_{MIN}, plus an allowance in design for deviation, Δc_{DEV}:

$$c_{NOM} = c_{MIN} + \Delta c_{DEV} \tag{2.77}$$

The minimum concrete cover, c_{MIN}, shall be provided in order to ensure the safe transmission of bond forces, the protection of the steel against corrosion (durability) and an adequate fire resistance.

The minimum cover values for reinforcement according to prEN 10080 (1999) in normal weight concrete take account of the exposure classes and the structural classes is given by $c_{MIN,DUR}$. The structural classification and values of $c_{MIN,DUR}$ for use in a Country may be found in its National Annex. The recommended Structural Class (service life of 50 years) is 4 for the indicative concrete strengths given in Annex E.

Table 2.16 - Recommended limiting values for composition and properties of concrete (EN 206-1 2000).

Exposure classes																	
	No risk of corrosion or attack	Carbonation-induced corrosion				Chloride-induced corrosion						Freeze/thaw attack				Aggressive chemical environments		
						Sea water			Chloride other than from sea water									
	X0	XC 1	XC 2	XC 3	XC 4	XS 1	XS 2	XS 3	XD 1	XD 2	XD 3	XF 1	XF 2	XF 3	XF 4	XA 1	XA 2	XA 3
Maximum w/c	—	0.65	0.60	0.55	0.50	0.50	0.45	0.45	0.55	0.55	0.45	0.55	0.55	0.50	0.45	0.55	0.50	0.45
Minimum strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
Minimum cement content (kg/m3)	—	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360
Minimum air content (%)	—	—	—	—	—	—	—	—	—	—	—	—	4.0a	4.0a	4.0a	—	—	—
Other requirements												Aggregate in accordance with EN 12620:2000 with sufficient freeze/thaw resistance					Sulfate-resisting cement b	

a Where the concrete is not air entrained, the performance of concrete should be tested according to an appropriate test method in comparison with a concrete for which freeze/thaw resistance for the relevant exposure class is proven.

b When SO_4^{2-} leads to exposure Classes XA2 and XA3, it is essential to use sulfate-resisting cement. Where cement is classified with respect to sulfate resistance, moderate or high sulfate-resisting cement should be used in exposure Class XA2 (and in exposure Class XA1 when applicable) and high sulfate-resisting cement should be used in exposure Class XA3.

The recommended values of $c_{MIN,DUR}$ are given in Table 2.17 for reinforcing steel and in Table 2.18 for prestressing steel.

Where stainless steel is used or where other special measures have been taken, the minimum cover may be reduced by $\Delta c_{DUR,ST}$. For such situations the effects on all relevant material properties should be considered, including bond. The value of $\Delta c_{DUR,ST}$ for use in a Country may be found in its National Annex. The recommended value, without further specification, is 0 mm.

Table 2.17 - Values of minimum cover requirements with regard to durability for reinforcement steel. (prEN 1992.1-1 2002)

Environmental Requirement for $c_{MIN,DUR}$ (mm)							
Structural Class	Exposure Class according to Table 2.14						
	X0	XC1	XC2/XC3	XC4	XD1 /XS1	XD2/XS2	XD3/XS3
1	10	10	10	15	20	25	30
2	10	10	15	20	25	30	35
3	10	10	20	25	30	35	40
4	10	15	25	30	35	40	45
5	15	20	30	35	40	45	50
6	20	25	35	40	45	50	55

Table 2.18 - Values of minimum cover requirements with regard to durability for prestressing steel (prEN 1992.1-1 2002)

Environmental Requirement for $c_{MIN,DUR}$ (mm)							
Structural Class	Exposure Class according to Table 2.14						
	X0	XC1	XC2/XC3	XC4	XD1 /XS1	XD2/XS2	XD3/XS3
1	10	15	20	25	30	35	40
2	10	15	25	30	35	40	45
3	10	20	30	35	40	45	50
4	10	25	35	40	45	50	55
5	15	30	40	45	50	55	60
6	20	35	45	50	55	60	65

For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta c_{DUR,ADD}$. The value of $\Delta c_{DUR,ADD}$ for use in a Country may be found in its National Annex. The recommended value, without further specification, is 0 mm.

An addition to the minimum cover shall be made in design to allow for the deviation (Δc_{DEV}). The required minimum cover shall be increased by the accepted negative deviation given in the standard for execution. This may depend on the type of structure. For Buildings, the NP ENV 13670-1(2000) gives the acceptable deviation. This is normally also sufficient for other types of structures. It should be considered when choosing the value of nominal cover for design. The nominal value of cover for design should be used in the calculations and stated on the drawings, unless a value other than the nominal cover is specified (e.g. minimum value).

The value of Δc_{DEV} for use in a Country may be found in its National Annex. The recommended value is 10 mm. In certain situations, the accepted deviation and hence allowance, Δc_{DEV}, may be reduced. The reduction in Δc_{DEV} in such circumstances for use in a Country may be found in its National Annex. The recommended values are:

- where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover, the allowance in design for deviation Δc_{DEV} may be reduced $10 \text{ mm} \geq \Delta c_{DEV} \geq 5 \text{ mm}$;
- where it can be assured that a very accurate measurement device is used for monitoring and non conforming members are rejected (e.g. precast elements), the allowance in design for deviation Δc_{DEV} may be reduced $10 \text{ mm} \geq \Delta c_{DEV} \geq 0 \text{ mm}$.

2.6.3.2 Requirements of Portuguese Codes for concrete

The replacement of the informative limiting requirements for concrete composition indicated in Annex F of the NP EN 206-1 is shown in tables 2.19 and 2.20, when there is a corrosion risk; and in tables 2.21 e 2.22, when there is Freeze/thaw or chemical attack. These indicate maximum water/cement values, minimum cement content and minimum strength classes that concrete must satisfy in Portugal for a 50 years service life and subject to the environmental actions classified in the NP EN 206-1. The X0 durability class is not considered.

Also presented in the tables are the cement types best suited for the environment. This does not imply, however, that following need not be checked:

a) the requirements for concrete placement, compaction, protection and curing are specified in the NP ENV 13670-1 – “Execution of concrete structures - Part 1: Common Rules”;

b) The minimum concrete cover specified in the NP EN 1992-1.1- “Eurocode 2: Design of concrete structures - Part 1: General rules and rules for buildings”.

The cement content mentioned in the tables is for a concrete mix with maximum aggregate size, D_{MAX}, between 20 mm and 32 mm.

For concrete with maximum aggregate sizes smaller than 20 mm, the cement content can be calculated by (NP EN 206-1 2000):

For 20 mm > $D_{MAX} \geq$ 12,5 mm : $C_{20/12,5} = 1{,}10\ C$ (2.78)

for 12,5 mm > D_{MAX} > 4 mm : $C_{12,5/4} = 1{,}23\ C$ (2.79)

Table 2.19 - Limiting values for concrete composition and strength class subject to carbonation (LNEC E 465 2004).

Cement type	CEM I (Referência); CEM II/A (1)				CEM II/B(1); CEM III/A(2); CEM IV(2); CEM V/A(2)			
Exposure class	XC1	XC2	XC1	XC2	XC1	XC2	XC1	XC2
Concrete cover (mm)	20	25	20	25	20	25	20	25
Maximum water/cement ration	0.65	0.65	0.65	0.65	0.65	0.65	0.65	0.65
Minimum cement content, C (kg/m^3)	240	240	240	240	240	240	240	240
Minimum strength class	C25/30 LC25/28	C25/30 LC25/28	C25/30 LC25/28	C25/30 LC25/28	C25/30 LC25/28	C25/30 LC25/28	C25/30 LC25/28	C25/30 LC25/28
(1) Not applicable to the cements II/A-T, II/A-W and to the cements II/B-T and II/B-W, respectively (2) Not applicable to the cements with percentages of portland clinker less than 50%, in mass.								

Table 2.20 - Limiting values for concrete composition and strength class subject to chlorides (LNEC E 465 2004).

Cement type	CEM IV/A (Referência); CEM IV/B; CEM III/A; CEM III/B; CEM V; CEM II/B (1); CEM II/A-D			CEM I; CEM II/A (1)		
Exposure class	XS1/ XD1	XS2/ XD2	XS1/ XD1	XS2/ XD2	XS1/ XD1	XS2/ XD2
Concrete cover (mm)	35	40	35	40	35	40
Maximum water/cement ration	0.55	0.55	0.55	0,.55	0.55	0.55
Minimum cement content, C (kg/m^3)	320	320	320	320	320	320
Minimum strength class	C30/37 LC30/33	C30/37 LC30/33	C30/37 LC30/33	C30/37 LC30/33	C30/37 LC30/33	C30/37 LC30/33

Table 2.21 - Limiting values for concrete mix and strength class subject to chemical attack (NP EN 206-1 2000).

Cement type	CEM I, II, III e IV		
Exposure class	XA1	XA2	XA3
Maximum water/cement ration	0.50	0.45	0.40
Minimum cement content, C (kg/m^3)	340	360	380

Minimum strength class	C30/37	C35/45	C40/50

Table 2.22 - Limiting values for concrete mix and strength class subject to freeze/thaw (NP EN 206-1 2000).

Cement type	CEM I, II, III, IV e V	
Exposure class	XF1	XF2
Maximum water/cement ration	0.55	0.55
Minimum cement content, C (kg/m^3)	300	300
Minimum strength class	C30/37; LC30/33	C30/37; LC30/33
Minumu air content (%)	-------	4,0

There are two national pre-specifications related to the durability design of reinforced concrete structures: LNEC E-464 (2004) and the LNEC E-465 (2004).

The *LNEC E-464 - Concrete. Prescriptive methodology for a design working life of 50 years.*, replaces the specification LNEC E-378 (1996), establishing the suitability of mixtures of cement and additions and the prescriptions for the concrete with a 50 years working life. The suitability of the equivalent performance concept is also established. It suggests alterations for the values presented in Annex E of the Eurocode 2 (2002). The substitution of the limit values from the Annex F of the NP EN 206-1 (1999) is also presented.

Several types of cements can be used. However, the following must still be fulfilled in order to guarantee the durability of the structures:

- the nominal reinforcement cover be the sum of the minimum cover mentioned in the EN 1992-1.1 (2002) for a service life of 50 years, plus an allowed deviation of 10 mm, from the NP ENV 13670-1 (2002). It is the nominal cover that must be guaranteed in design and execution of concrete structures (Table 2.23);
- the requirements for placement, compaction, protection and curing and other requirements established in the NP ENV 13670-1 (2002).

Table 2.23 - Nominal reinforcement concrete cover.

$t_{g\ (anos)}$	XC1	XC2/XC3	XC4	XS1	XS2	XS3
50	25	35	40	45	50	55
100	35	45	50	55	60	65

The *LNEC E-465 - Concrete. Methodology for estimating the performance properties of the* *reinforced or prestressed concrete so as to comply with the design working life, under the environmental exposures XC and XS,* recalls the framework given by the definitions and the principles related to durability stated in the EN 1990 (2001), namely the probabilistic approach, and in the NP EN 206-1 (1999).

Based on the Tuutti model for estimating the working life, this specification details several models for the initiation and propagation periods and quantifies on a probabilistic approach some of the performance properties of those models for each environmental class.

Table 2.24 - Chloride diffusion coefficients for concrete in the environmental exposure class XS1 next to the sea.

D_0 (10^{-12} m^2/s) (XS1, next to the sea) k=2,016		*w/c*	0.40	0.45	0.50	0.55
		C_R*(%)*	0.5	0.4	0.4	0.4
		C_S*(%)*	1.68	1.89	2.10	2.31
t_g = 50 years x_C = 45 mm	RC3	t_{ic}	125			
		D_0	14	10	9.1	8.4
	RC2	t_{ic}	105			
		D_0	15	11	9.7	8.9
	RC1	t_{ic}	90			
		D_0	16	11	10	9.4
t_g = 100 years x_C = 55 mm	RC3	t_{ic}	265			
		D_0	16	11	10	9.7
	RC2	t_{ic}	610			
		D_0	18	12	11	10

The data in table 2.24 can be interpreted in the following manner. For a given service life (t_g), concrete cover (x_C) and reliability class (*RCi*), diffusion coefficients are suggest based on the w/c ratio, C_R and C_S chloride concentration levels that should fulfil the initiation period (t_{ic}) given.

In the Annex A, values for the performance properties of concrete in the various environment exposures are presented. As an example, Table 2.24 presents the chloride diffusion coefficients for concrete in the environmental exposure class XS1 next to the sea, for the service life periods of 50 and 100 years. These are presented in relation to the reliability classes, the w/c ratio and the diffusion coefficients D_0.

This specification established only one limit state, the serviceability limit state of initiation of cracking of the concrete cover due to reinforcement corrosion.

The chloride concentrations adopted to initiated reinforcement corrosion C_{CR} (% weight of cement), is given in table 2.25.

Table 2.25 - Chloride concentration, C_{CR} (% weight of cement)

Water/cement	XS1; XS2	XS3
$w/c \leq 0.30$	0.6	0.5
$0.30 < w/c \leq 0.40$	0.5	0.4
$w/c > 0.40$	0.4	0.3

2.6.3.3 Requirements of Norwegian Codes for concrete

In Norway, it is the national standardisation body, The Norwegian Council for Building Standardisation (NBR) that establishes the code committees to work out the requirements for concrete construction.

Having in mind Norway's long coast and numerous marine structures, the provisions needed to achieve a relevant set of requirements to ensure the expected in-field performance of chloride exposed structures (exposure classes XS), was considered as a key issue.

To derive these provisions, an assessment of the performance of existing structures was performed. These assessments were based on in-field observations processed by means of a mathematical model for ingress of chlorides in concrete. To conclude with the proxy parameters for durability needed for the standard, the probabilistic approach defined in prEN 1990: 2001 “EuroCode: Basis of Structural Design” was applied.

Rules for concrete cover are given in NS 3473 (2002) “Design of concrete structures” and NS 3465 (2003) “Execution of concrete structures. Common Rules”. The NS 3465 will be almost a loyal copy of ENV 13670-1 (2000) “Execution of concrete structures”

In NS 3473 (2003), the minimum required concrete cover x_{min} for the exposures classes is given. Table 2.26 presents the requirements for minimum concrete cover of the new standard, for various exposure classes adjusted by the NS 3473.

Table 2.26 – Minimum concrete cover with respect to corrosion protection (NS 3473 2003).

Exposure classe (XC, XD, XS, XSA)	50 year service life		100 year service life	
	Reinforcement sensitive to corrosion	Reinforcement slightly sensitive to corrosion	Reinforcement sensitive to corrosion	Reinforcement slightly sensitive to corrosion
XC1	25 mm	15 mm	35 mm	25 mm
XC2, XC3, XC4	35 mm	25 mm	45 mm	35 mm
XS1, XS2, XD1, XD2, XD3	50 mm	40 mm	60 mm	50 mm
XS3	60 mm	50 mm	70 mm	60 mm
XSA	To be determined separately			

The nominal cover x_n, defined as the sum of the minimum cover x_{min} and a deviation, is equal to the average target concrete cover. The deviation, $\Delta_{(minus)}$ is, according to ENV 13670-1, equal to 10 mm, which means that the required nominal cover is 60 mm for structures in marine environment. These tolerances are also in agreement with the Norwegian Standards NS 3473 (2003) and the NS 3420 (2002).

The environmental classes SA, MA, NA and LA that appeared in the NS 3420-L (1999) and the NS 3473 will become identical to those in the NS-EN 206-1 and the revised version of the NS 3473 being replaced by

- 18 exposure classes that classify the environmental loading and,
- 6 concrete durability classes that group concrete resistance against degradation.

While the older standards considered simultaneously both the environmental loading and the concrete composition that was be adopted, this is now separated between the environmental loading and the material resistance.

In the new concrete standards six durability classes are defined. These define the concretes durability characteristics and the properties of the concrete necessary to fulfil the requirements in the different exposure classes. Table 2.27 states which durability classes can be used in the various exposure classes required in the NS EN 206-1.

Table 2.27 – Durability classes according to the NS 3473 (2003).

Exposure classes	Durability Classes					
	M-90	M-60	M-45	MF-45	M-40	MF-40
X0	X	X	X	X	X	X
XC1, XC2, XC3, XC4, XF1		X	X	X	X	X
XD1, XS1, XA1, XA2, XA4			X	X	X	X
XF2, XF3, XF4				X		X
XD2, XD3, XS2, XS3, XA3					X	X
XSA	Composition of concrete and protective measure to be defined separately. (verify at least M40)					

For concrete to be produced according to the NS EN 206-1, Table 2.28 presents the durability classes and the requirements needed.

Table 2.28 – Requirements for durability classes (NS EN 206-1).

Requirements	Durability Classes					
	M-90	M-60	M-45	MF-45	M-40	MF-40
w/c ratio	0.90	0.60	0.45	0.45	0.40	0.40
Minimum air content (%)	-	-	-	> 4 %	-	> 4 %
Minimum cement Content (kg/m^3)	225	250’	300	300	330	330

In addition to the limits mentioned in the national annex of the NS EN 206-1, a summary about which cements can be used with the durability classes is given.

CHAPTER 3

PROPOSAL FOR A PROBABILISTIC MODEL FOR DURABILITY ANALYSIS

3.1 General

In designing for durability, the first step is the definition of the desired/required performance of the structure. The deterioration mechanism responsible for the majority of degradated concrete structures in marine environment is the corrosion of reinforcement in the concrete, induced essentially by chloride penetration into the concrete.

During recent years, much research work has been carried out on modelling of chloride penetration and development of new procedures for service life assessment of concrete structures. Since the parameters both for concrete durability and environmental exposure typically show a high scatter and are time dependant, the introduction of a probabilistic-based service life assessment has shown to be valuable (Duracrete 1998, Duracrete 2000, McGee 1999).

However, many of the proposed models in the literature are too detailed and require too much data to be used. Furthermore, there are few engineers with sufficient knowledge of reliability analysis to be able to use a model in a probabilistic manner.

The proposed model is based on the same background as most others (Fick’s 2nd Law), but only essential alterations are made to the model. In this way, the model is kept as simple as possible, while functional.

3.2 Model definition

The widely acceptable model by Tuutti (1982) for this process is presented in figure 2.23. During the initiation phase no weakening of the concrete occurs, but an aggressive environment attacks the inherent protective barrier around the steel reinforcement. The protective layer of the reinforcement deteriorates gradually with time up to a certain point, where the passive layer breaks down allowing the initiation of corrosion. This marks the beginning of the propagation phase. From here on, and in the presence of sufficient oxygen and humidity there is a rapid deterioration of the steel due to corrosion. Gradually, visible defects appear on the surface, cracks followed by spalling and delaminations and eventually the failure of the structure.

The model presented is based on Fick´s 2nd law for diffusion, commonly used to model chloride penetration. Adjustments are made in the diffusion equation to allow for the time dependency of the diffusion coefficient and the effect that temperature has on the parameters. However, it remains still that any model can never fully simulate reality. For this reason, engineers and designers should always understand the model they are working with, its limitations, and know how to interpret the results.

3.2.1 Basic diffusion model

The rate of chloride penetrating into concrete as a function of depth from the surface

$$\frac{dC(x,t)}{dt} = D_C \frac{d^2C(x,t)}{dx^2} \tag{3.1}$$

where *C(x,t)* is the chloride ion concentration, at a distance x from the concrete surface after being exposed for a period of time, *t,* to the source of the chlorides. D_C is the chloride diffusion coefficient. By solving this differential equation for predefined boundary conditions (see chapter 2.5), equation 3.2 is obtained

$$C(x,t) = C_I + (C_S - C_I) \cdot \left[1 - erf\left(\frac{x}{2\sqrt{D_C \cdot t}}\right)\right] \tag{3.2}$$

where C_S is the chloride concentration on the concrete surface, C_I is the initial chloride concentration of the concrete mix, and *erf* is the error function. The limit state equation *G( )* is defined when considering that the corrosion is initiated when the chloride concentration around the reinforcement exceeds the critical chloride content.

Although initiation of corrosion represents a useful defining point for the Service Life design of a new structure, it is evident that many structures continue to function, without repair, once corrosion has become established. Therefore, for the model presented, the target service life or the event that identifies the end of service life is defined by the serviceability limit state represented by the depassivation of the reinforcement in concrete. This limit state was chosen since it represents the end of the depassivation phase. Therefore, it does not coincide with any kind of damage, but is an important turning point of the durability of the reinforced concrete structure. It allows for sufficient time for structural/material assessment and more important, time to evaluate the appropriate intervention.

From a probabilistic point of view, the basic structural reliability problem considers only one load effect *S* resisted by one resistance *R.* Each of *S* and *R* are described by a known probability density function, $f_s(s)$ *and* $f_R(r)$ respectively. If *R* and *S* have known probability density functions (PDF) $f_R(r)$ and $f_S(s)$ respectively and expressed in the same units, then,

$$p_f = P(R - S \le 0) \quad \text{or} \quad P[g(R,S) \le 0] \tag{3.3}$$

where *g( )* is termed the limit state function, and the probability of failure is identical with the probability of limit state violation.

By resolving equation (3.2) in function of the concrete cover x_C, the load *S,* defined as the penetration depth of chlorides in the concrete surface, is given by

$$S(t) = x_C(t) = 2 \cdot erf^{-1}\left(1 - \left(\frac{C_{CR} - C_0}{C_S - C_0}\right)\right) \cdot \sqrt{D_C \cdot t} \tag{3.4}$$

where C_{CR} is the critical chloride threshold level and erf^{-1} is the inverse of the error function.

The resistance *R* is defined by the geometrical characteristics of the stochastic variable reinforcement concrete cover.

3.2.2 Time dependency of the diffusion model

The time dependency of the diffusion coefficient is well known, ever since Takewaka presented an equation to model this behaviour (Takewaka 1988). Much research has been done on this topic (Maage *et al.* 1994; Mangat *et al.* 1994, Duracrete 1998), and, it has shown that the diffusion coefficient variation with time is dependent on various factors, the most important being the w/c ratio of the mix and the cement type and content.

The time dependent variation of the D_C is introduced by the following equation (Takewaka 1988)

$$D_C = D_0 \left(\frac{t_0}{t}\right)^{\alpha} \tag{3.5}$$

where D_0 is the apparent diffusion coefficient at the time t_0 and α is a parameter the defines the evolution of the diffusion coefficient with time.

By substituting equation (3.5) in equation (3.4) we obtain an expression for the prediction of the depth of the critical chloride concentration front based on a time dependent diffusion coefficient, given by

$$S(t) = x_C(t) = 2.erf^{-1}\left(1 - \left(\frac{C_{CR} - C_0}{C_S - C_0}\right)\right) \cdot \sqrt{D_0 \left(\frac{t_0}{t}\right)^{\alpha} \cdot t} \tag{3.6}$$

The variables in equation (3.6) are generally used with the following units:

x_C - concrete cover (m);

D_0 - diffusion coefficient at the time t_0 (m^2/s);

C_{CR} - critical chloride threshold content (wt.% Cl^-/binder);

C_S - surface chloride content in (wt.% Cl^-/binder);

C_0 - initial chloride content in (wt.% Cl^-/binder);

α - factor that takes the influence of age on D_0 into account;

t - exposure period (s);

t_0 - reference period (s);

erf^{-1} - is the inverse of the error function.

Much investigation has been done on the parameter α that affects the diffusion coefficient with time (Duracrete 2000, Bamforth 1999, Gjørv 2002, Gjørv 2002a).

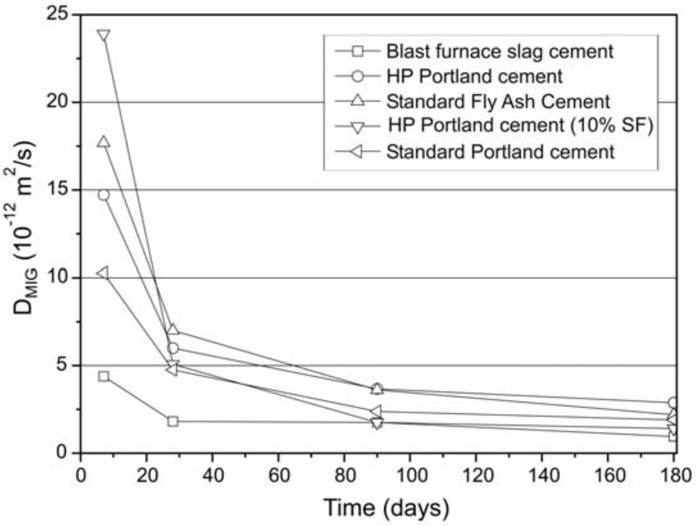


Figure 3.1 – Time dependency of the diffusion coefficient for concrete with different cement types (Gjørv 2002a, Gjørv 2002b).

The variation is defined mainly by the cement type, w/c ratio, and type of exposure. Figures 3.1 and 3.2 show results from Gjørv *et al* (2002, 2002a) and Costa (1997), respectively.

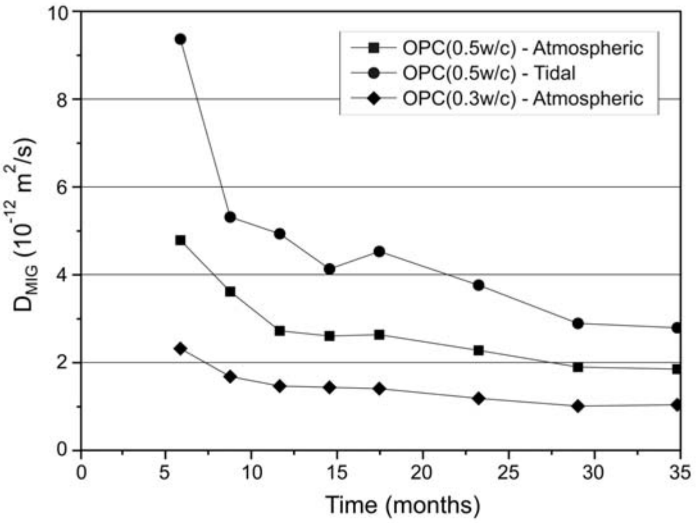


Figure 3.2 –Time dependency of the diffusion coefficient for concrete with different w/c ration and exposure conditions (Costa 1997).

The time dependency of the diffusion coefficient is visible from these figures. The following table (Table 3.1) has been taken from the Duracrete Project (1998) where the α parameter has been characterised according to the cement type and the exposure conditions.

Table 3.1 – α parameter values (Duracrete 1998)

		Normal Distribution		Beta (a=0; b=1)	
Binder	Environment	Mean	Average	q	r
OPC	Submerged	0.30	0.05	24.9	58.1
	Tidal+splash	0.37	0.07	17.2	29.3
	Atmospheric	0.65	0.07	29.5	15.9
PFA	Submerged	0.69	0.05	58.3	26.2
	Tidal+splash	0.93	0.07	1234	92.9
	Atmospheric	0.66	0.07	29.6	15.2
GGBS	Submerged	0.71	0.05	57.8	23.6
	Tidal+splash	0.80	0.07	5.8	3.87
	Atmospheric	0.85	0.07	21.3	3.75
SF	Submerged	0.62	0.05	57.8	35.4
	Tidal+splash	0.39	0.07	18.5	29.0
	Atmospheric	0.79	0.07	26	6.9

However, results from research demonstrate to certain extent the reality that, no general consensus exists with regards to α values. Much research is still needed to produce values that are in general agreement with the engineering community.

3.2.3 Temperature effect on the diffusion coefficient

For temperature levels above freezing, the temperature is a decisive factor regarding the rate of deterioration. Chemical and electro-chemical reactions are accelerated by increases in temperature. A simple rule-of-thumb is that an increase in temperature of 10ºC causes a doubling of the rate of reaction. This factor alone makes hot and tropical environments considerably more aggressive than temperate climates.

The influence of the temperature on the corrosion process of steel in concrete, especially, on the corrosion potential, corrosion rate, concrete resistivity and transport processes in concrete depends on the physicochemical properties of the pore water solution (Pruckner 2001). The most important parameter should be the viscosity of water. Ionic conductivity of an electrolyte is related to ionic mobilities, therefore similar temperature effects as for diffusion processes should be expected. Since the viscosity η is inversely proportional to the mobility of the particles, we should expect that

$$\eta = C \cdot \exp\left(\frac{E_A}{RT}\right) \tag{3.7}$$

where

E_A - activation energy for the viscous flow;

C - constant;

implying that the viscosity should decrease exponentially with increasing temperature. This is found to be the case, at least over reasonably small temperature ranges for the viscosity of water (González 1993).

The temperature dependence of the viscosity of water is mainly important for other effects such as the concrete’s resistivity and diffusion processes. Diffusion processes are strongly dependent on temperature. Using the Nernst-Einstein relation

$$D = \frac{RT}{z^2 F^2} \cdot \lambda \tag{3.8}$$

And

$$\lambda = zcuF \tag{3.9}$$

$$u = \frac{ze_o}{6\pi\eta r} \tag{3.10}$$

with η from eq (3.7), we get the following theoretical expression for the temperature dependence of the diffusion coefficient which is in agreement with the literature (Berke 1994):

$$D = const \cdot T \cdot \exp\left(-\frac{E_A}{RT}\right) \tag{3.11}$$

D - diffusion coefficient;
R - Gas constant;
T - temperature;
z - number of electrons;
F - Faraday constant (96485 C/mol);
c - concentration;
u - mobility;
e_0 - elementary charge;
γ - surface tension pore water;
λ - conductivity.

Values for the diffusion coefficient at ambient temperature (21ºC) are correlated to values at standard temperature according to:

$$D(T) = D_{294K} \cdot \frac{T}{294} \cdot \exp\left[-\frac{E_A}{R}\left(\frac{1}{T} - \frac{1}{294} \right) \right] \tag{3.12}$$

where 21 ºC in Kelvin is approximately 294 K.

In the case of chloride diffusion the situation is somehow more complicated, because in the diffusion process the chemical and physical interaction of chloride with the cement paste has to also be taken into consideration. The value for the activation energy for the diffusion of Cl^- reported by Saetta *et al* (1993) for ordinary Portland cement pastes made with different water cement ratios is shown in Table 3.2. Cement pastes with a more open pore structure (higher w/c ratios) are expected to deliver lower values for E_A.

Table 3.2 – Values of activation energy for chloride diffusion for concretes of different w/c ratios (Saetta 1993)

w/c	E_A (kJ/mol)
0.4	41.8±4.0
0.5	44.6±4.3
0.6	32.0±2.4

By substituting equation (3.12) in equation (3.6) we obtain an expression for the prediction of the chloride concentration based on a time dependent diffusion coefficient with the effect of temperature,

$$S(t) = x_C\left(t\right) =$$

$$= 2.erf^{-1}\left(1-\left(\frac{C_{CR}-C_0}{C_S-C_0}\right)\right)\cdot\sqrt{D_0\cdot\frac{T}{294K}\cdot\exp\left[-\frac{E_A}{R}\left(\frac{1}{T}-\frac{1}{294K}\right)\right]\cdot\left(\frac{t_0}{t}\right)^{\alpha}\cdot t} \tag{3.13}$$

The variables in equation (3.13) are generally used with the following units:

x_C - concrete cover (m);

D_0 - diffusion coefficient at the time t_0 (m^2/s);

C_{CR} - critical chloride threshold content (% chloride/ weight binder);

C_S - surface chloride content in (% chloride/ weight binder);

C_0 - initial chloride content in (% chloride/ weight binder);

α - factor that takes the influence of age on D_0 into account;

E_A - activation energy for chloride diffusion in (J/mol);

T - temperature in (Kelvin);

R - Gas constant;

t - exposure period (s);

t_0 - reference period (s);

erf^{-1} - is the inverse of the error function.

3.2.4 Conversion from Rapid Chloride Migration to Chloride Profile Method

For this model, it is assumed that the diffusion coefficient is determined by the CTH migration method (NT Build 492 1999, LNEC E-465 2004) due to its simplicity and rapidity. However, the need may arise to use other test methods or procedures. Therefore, the relationship between the CTH method and the chloride profile method, based on recent research (Luping 1996, Duracrete 2000a), is considered to be linear. This relationship is based on limited data available. The statistical quantities of the relationship are given in Table 3.3.

Table 3.3 – Factor for conversion of RCM into CPM diffusion coefficient (DuraCrete 1998).

Variable	Distribution type	Mean	Standard Deviation
RCM/CPM	Normal	0.832	0.024

3.2.5 Model uncertainty

A model, by definition can never completely simulate the realistic phenomenon. To the many assumptions made to reduce the complexity of the phenomenon, the models introduce errors in the results. In probabilistic models, this *error* is partially taken into account by increasing the scatter of the results obtained from the model. The parameter for the model uncertainty takes into account the uncertainties that arise due related to the simulation of a real phenomenon process (Leira 2000). The value chosen is presented in table 3.4.

Table 3.4 – Factor for conversion of RCM into CPM diffusion coefficient (Leira 2000)

Distribution type	Mean	Standard Deviation
Normal	1.00	0.01

3.2.6 Evaluating chloride profiles

To evaluating the chloride profiles the second Fick's law is applied. It is assumed that no significant deviation of Fick's second law occurs under constantly immersed conditions. When evaluating the achieved chloride profiles which were found in concretes exposed to real environmental conditions, it is necessary to consider the possibility, that deviations from the ideal shape can appear (due to carbonation, leaching, temperature variations, different chloride loadings, etc.) (Duracrete 2000a).

Different possibilities to determine the parameters D_{CPM} (apparent diffusion coefficient, determined by chloride profiling) and C_{SN} (surface chloride concentration, determined by chloride profiling), of which those are applied in practice are based on a best fit least squares analysis, as illustrated in Figure 3.3.

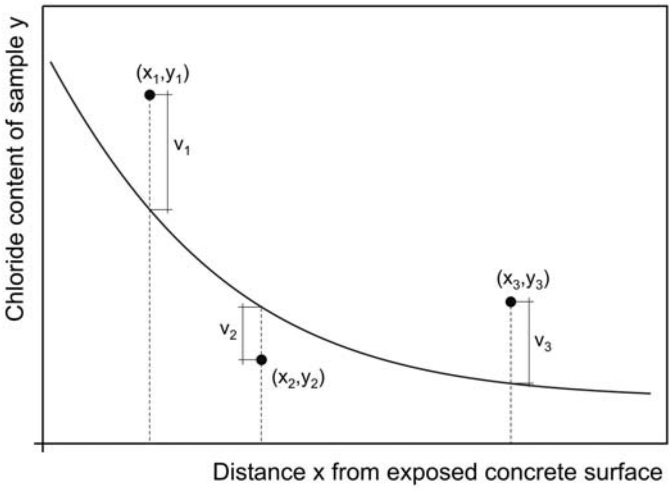


Figure 3.3 - Curve fitting (Fick's 2nd law of diffusion) by the method of least squares.

The determined output derived from the curve fitting carried out will be:

- D_{CPM}, an apparent diffusion coefficient, determined by chloride profiling in [m^2/s] and
- C_{SN}, the notional surface chloride level in % by weight of concrete.

3.2.7 Prediction of diffusion coefficient

To predict the chloride profile in time, a predefined chloride profile must be known including its age. In the proposed model, two chloride profiles are calculated. The first over estimates the concentrations of chloride, while the second underestimates the concentrations of chloride. This is achieved by using different diffusion coefficient to calculate the values in time.

At a given time, the initial chloride profile is measured and curve fitted to determine the *Cs* and the *D*. If the chloride profile is to be predicted for a given time, then using Fick's 2^{nd} law of diffusion the concentrations at different depth can be recalculated (the profile depths) with the *D* and *Cs* obtained from the initial given profile. This procedure will result in an over estimation of chloride profile as we do not take into account the variation with time of the *D*. *Cs* is assumed to be constant (Bamforth 1999).

If however, the value *D* at the prediction time due to the effect of cement hydration and other factors is calculated, and used to determine the new profile, then the profile will be underestimated, because *D* the value used is of improved quality translating a lesser penetration of chlorides.

The procedure adopted to calculate the profiles is just a rough approximation. For this reason, two profiles are calculated, one over estimated and one underestimated so that an idea of the limits the future profile might have are given.

3.3 Mathematical implementation

Due to the stochastic nature of many of the parameters that govern the performance of reinforced concrete structures, a probabilistic approach as opposed to a deterministic one is needed to generate reliable service life predictions. Modelling explicitly the various uncertainties through a probabilistic-based method leads to reliable models for the prediction of service life of reinforced concrete structures.

In order to include the uncertainty of the various parameters in a consistent manner, probabilistic modelling must be utilised. Probabilistic modelling of the deterioration mechanisms has gained strong momentum during the past few years.

3.3.1 Monte Carlo method

Numerical methods that are known as Monte Carlo Methods can be loosely described as statistical simulations methods, where statistical simulation is defined in quite general terms to be any method that utilizes sequences of random numbers to perform the simulation.

In the simplest form of the basic simulation, each random variable in a problem is sampled several times to represent its real distribution according to its probabilistic characteristics. Considering each realization of all the random variables in the problem produces a set of numbers that indicates one realization of the problem itself. Solving the problem deterministically for each realization is known as a simulation cycle, trial, or run. Using many simulation cycles gives the overall probabilistic characteristics of the problem, particularly when the number of cycles N tends to infinity. The simulation technique using a computer is an inexpensive way (compared to laboratory testing) to study the uncertainty in the problem.

The availability of personal computers and software makes the process very simple. In fact, to evaluate the accuracy of these sophisticated techniques or to verify a new technique, simulation is routinely used to independently evaluate the underlying probability of failure (Naess 1995).

This technique has evolved as a very powerful tool for engineers with basic working knowledge of probability and statistics for evaluating the risk or reliability of complicated engineering systems. In many applications of Monte Carlo, the physical process is simulated directly, and there is no need to even write down the differential equations that describe the behaviour of the system. The only requirement is that the physical (or mathematical) system be described by probability density functions (PDF’s). The Monte Carlo simulation technique has seven essential elements (CSEP 1995):

(1) *Problem definition:* defining the problem in terms of all the random variables;
(2) *Probability density functions:* quantifying the probabilistic characteristics of all the random variables in terms of their PDFs and the corresponding parameters;
(3) *Random Number Generators:* generating the values of these random variables;
(4) *Sampling Rules:* evaluating the problem deterministically for each set of realizations of all the random variables, that is, numerical experimentation;
(5) *Scoring:* extracting probabilistic information from N such realizations;
(6) *Error estimation:* determining the accuracy and efficiency of the simulation;
(7) *Variance reduction techniques:* methods for reducing the variance in the estimated solution to reduce the computational time for Monte Carlo Simulation.

Initially, all the random variables are considered to be uncorrelated.

Monte Carlo simulation techniques involve *sampling* at *random* to simulate artificially a large number of experiments and to observe the result. In the case of analysis for structural reliability, this means, in the simplest approach, sampling each random variable X_i, randomly to give a sample value $\hat{x}_i$. The limit state function $G(\hat{x}) = 0$ is then checked. If the limit state is violated (i.e. $G(\hat{x}) < 0$), the structure or structural element has *failed*. The experiment is repeated many times, each time with a randomly chosen vector $\overline{x}$ of $\hat{x}_i$ values. If N trials are conducted, the probability of failure is given approximately by

$$p_f = \frac{n\left(G\left(\hat{x}_i\right) \leq 0\right)}{N} \tag{3.14}$$

where $n(G(\hat{x}) \leq 0)$ denotes the number of trials n for which $(G(\hat{x}) \leq 0)$. Obviously the number N of trials required is related to the desired accuracy for p_f.

In this application of Monte Carlo, the physical process is simulated directly by the use of the model above mentioned to describe the behavior of the phenomena. The only requirement is that the physical/mathematical system be described by probability density functions (PDF), more commonly referred to as distributions. It is clear that, in the Monte Carlo method, a game of chance is constructed from known probabilistic properties in order to solve the problem many times over and from that to deduce the required result, i.e. the failure probability.

3.3.2 Random number generator

A necessary tool in simulation techniques for estimation of the probability of failure is to simulate outcome of stochastic variables with an arbitrary distributions. Provided that the lot size is large and the interval between numbers small, the probability distribution for these numbers would be the uniform or rectangular distribution, given by

$$F_V(v) = \begin{cases} v & if \quad 0 \leq v \leq 1 \\ 0 & if \quad else \end{cases} \tag{3.15}$$

The most common practical approach is to employ a *pseudo random number generator* (PRNG) such as is available on virtually all computer systems. They are termed *pseudo* since they use a formula to generate a sequence of numbers. Although this sequence is reproducible and repeats after (normally) a long cycle interval, for many practical purposes it is indistinguishable from a sequence of strictly true random numbers (Rubinstein 1981). The production of a reproducible sequence has an advantage in certain problems and in research work. If required, reproducibility can be destroyed simply by changing (randomly) the *seed number* required as input for most PRNGs. A simple device is to use the local time as a seed value (Naess 1995).

In this model, the generation of random numbers is performed according to the Minimal Standard generator suggested by Park and Miller (1998). There is good evidence, both theoretical and empirical, that the simple multiplicative congruential algorithm

$$I_{j+1} = aI_j \ (\mathrm{mod}\ \mathrm{m}) \tag{3.16}$$

can be as good as any of the more general linear congruential generators that have $c \neq 0$ if the multiplier a and modulus m are chosen carefully. Park and Miller propose a *Minimal Standard Generator* based on the choices

$$a = 7^5 = 16807 \ \text{ and } \ m = 2^{31} - 1 = 2147483647 \tag{3.17}$$

This generator has in subsequent years passed all new theoretical tests, and (perhaps more importantly) has accumulated a large amount of successful use. Park and Miller do not claim that the generator is "perfect" but only that it is a good minimal standard against which other generators should be judged. It is not possible to implement equations (3.16) and (3.17) directly in a high-level language, since the product of a and $m - 1$ exceeds the maximum value for a 32-bit integer. A trick for multiplying two 32-bit integers module a 32-bit constant, without using any intermediates larger than 32 bits (including a sign bit) is therefore extremely interesting: It allows the Minimal Standard generator to be implemented in essentially any programming language on essentially any machine.

Schrage's algorithm is based on an *approximate factorization* of m (Schrage 1979, Brately 1983).

$$m = aq + r, \quad \text{i.e.,} \quad q = [m/a],\ r = m \ \mathrm{mod}\ \mathrm{a} \tag{3.18}$$

with square brackets denoting integer part. If r is small, specifically $r < q,$ and $0 < z < m\text{-}1$, it can be shown that both $a(z \ mod \ q)$ and $r[z/q]$ lie in the range $0, \ldots, m - 1$, and that

$$az \bmod m = \begin{cases} a\left(z \bmod q\right) - r\left[z/q\right] & if \cdot it \geq 0, \\ a\left(z \bmod q\right) - r\left[z/q\right] + m & otherwise \end{cases} \tag{3.19}$$

The application of Schrage's algorithm to the constants (3.17) uses the values q = 127773 and r = 2836.

Here is an implementation of the Minimal Standard generator in C (Park *et al* 1998):

```
#define IA 16807
#define IM 2147483647
#define AM (1.0/IM)
#define IQ 127773
#define IR 2836
#define MASK 123459876

float ran0(long *idum)
{
    long k;
    long idum;

    idum ^= MASK;
    k=idum/IQ;
```

```
        idum=IA*(idum-k*IQ)-IR*k;
        if (idum < 0) {
            idum=idum+IM;
        } /* if (idum < 0) */
        *ans=AM*(idum);
        idum ^= MASK;
    } /* ran0 */
```

The period of *ran0* is $2^{31} - 2 \approx 2.1 \times 10^9$. A peculiarity of generators of the form (3.17) is that the value 0 must never be allowed as the initial seed (it perpetuates itself) and it never occurs for any nonzero initial seed. Experience has shown that users always manage to call random number generators with the seed *idum = 0*. That is why *ran0* performs its *exclusive-or* with an arbitrary constant both on entry and exit. This *minimal* random number generator of Park and Miller (1998) returns a uniform random deviate between 0.0 and 1.0. Set or reset idum to any integer value (except the unlikely value MASK) to initialize the sequence. The variable *idum* must not be altered between calls for successive deviate in a sequence.

3.3.3 Inverse CDF Method

The next task is to transform the uniform random numbers between 0 and 1, to a random numbers with the appropriate characteristics. This is commonly known as the *Inverse Transformation* method or *Inverse CDF* method.

For a general stochastic variable X the distribution function if $F_X(x)$. In the inverse transformation method two steps are needed to generate an outcome $\hat{x}$ of X:

1) generate an outcome $\hat{v}$ of V (example: using a multiplicative congruence generator)

2) determine the outcome of $\hat{x}$ by;

$$\hat{x} = F_X^{-1}\left(F_V\left(\hat{v}\right)\right) = F_X^{-1}\left(\hat{v}\right) \tag{3.20}$$

The method is illustrated in figure 3.4. It is seen that the distribution function for $\hat{X}$ with outcomes generated by the procedure is

$$F_{\hat{X}}\left(x\right) = P\left(\hat{X} \le x\right) = P\left(F_X^{-1}\left(V\right) \le x\right) = P\left(V \le F_X\left(x\right)\right) = F_X\left(x\right) \tag{3.21}$$

Specialized techniques for generating random variates from specific distributions often are computationally more efficient than the inverse transform method. Most computer systems have standard subroutines available.

The *Inverse Transform* method is the simplest Monte Carlo approach for reliability problems. The basis for its application is as follows. The probability of limit state violation may be expressed as (Naess 1995):

$$p_f = J = \int \ldots \int I\left[G\left(x\right) \le 0\right] f_X .dx \tag{3.22}$$

where the indicator function I [G(x)] is defined by

$$I\left[G\left(x_j\right)\right] = \begin{cases} 0 \quad if \quad g\left(x_j\right) > 0 & \left(safe\right) \\ 1 \quad if \quad g\left(x_j\right) > 0 & \left(failure\right) \end{cases} \tag{3.23}$$

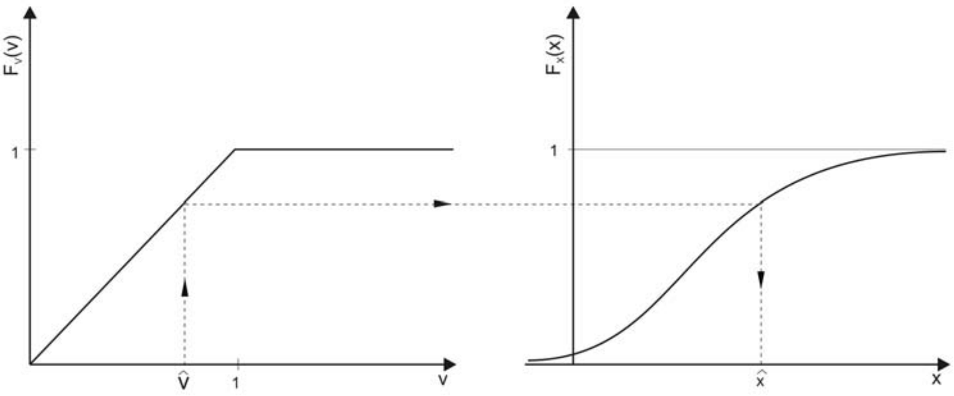


Figure 3.4 - Illustration of inverse method.

Thus the indicator function identifies the integration domain. If $\hat{x}_j$ represents the *j-th* vector of random observations from $f_x\,(\,)$, it follows directly from sample statistics that:

$$p_f = J_1 = \frac{1}{N}\sum_{j=1}^{N} I\left[G\left(\hat{x}_j\right) \le 0\right] \tag{3.24}$$

is an unbiased estimator of *the* p_f. Thus expression (3.24) provides a direct estimate of p_f. It is known that if the value of *g( )* is less than zero, it indicates failure. Let N_f be the number of simulation cycles when *g( )* is less than zero and let *N* be the total number of simulation cycles. Therefore, an estimate of the probability of failure can be expressed as

$$p_f = \frac{N_f}{N} \tag{3.25}$$

The standard error of the probability of failure is estimated by

$$s = \sqrt{\frac{P_f\left(1 - P_f\right)}{N}} \tag{3.26}$$

Confidence intervals for the estimate of the probability of failure can be determined using that, the probability of failure becomes normal distributed when N → ∞.

3.4 Summary

Many of the proposed models in the literature are too detailed and require too much data to be used. Furthermore, there are few engineers with sufficient knowledge of reliability analysis to be able to use a model in a probabilistic manner.

The proposed model is based on the same background as most others (Fick's 2nd Law), but only essential alterations are made to the model. In this way, the model is kept as simple as possible, while functional.

Improvements to the basic second Law of Fick include the time dependency of the model, and the temperature effect on the diffusion coefficient. Some of the models advantages are the ability to use different diffusion coefficients obtained either from the Rapid Chloride Migration and the Chloride Profile Method, the evaluation of the chloride profiles and the determination of the surface concentration of chlorides as well as the diffusion coefficient based on the best fit curve of the profile by Fick's second law and the prediction of future chloride profiles based on present profiles.

From a probabilistic point of view, the use of several distribution types and the incorporation of a variable to account for model variability are also advantages.

CHAPTER 4
DEVELOPMENT OF SOFTWARE BASED ON THE PROPOSED MODEL

4.1 General

In this chapter a brief explanation of the developed software is given. The software is named DURACON after *DURAbility of CONcrete.* It is written in C++ and has a visual interface written in Visual Basic. The software represents the implementation of the model proposed in the previous chapter. The software is only presented visually here, but more detailed user instructions can be found in the manual (Appendix 2).

The study of the sensitivity of the main parameters that belong to the model is of more interest to the designer. This is of importance as it gives the designer a notion of which parameters need to be quantified with greater precision. The extent of influence the parameters have in the outcome of the analysis is important.

The software developed manages to aids the designer to identify the most important parameters and illustrate the sensitivity of concrete durability towards each one.

4.2 The DURACON software

The previous chapter dealt with the details of the model used to describe the degradation process as well as the mathematical solution adopted for the Monte Carlo simulation.

The software, to be used by the engineer at both the design and construction phase, requires few but critical inputs that can be easily obtained, by commercially available equipment and well know test procedures, or, based on previous knowledge and experience. The manual for this software, with step by step information for its application, is given in the Appendix 2.

Some of the main characteristics of the software include:

- characterizations of model parameters with the use of three distributions functions (normal, lognormal and beta) as well as deterministic variables;
- adjustment of variables for temperature effect;
- adjustment of variables for age effect on diffusion coefficients;
- curve fitting of profiles for calculation of D_C and C_S;

- prediction of chloride profiles for future ages.

Version 1.0 (Beta Version) - Freeware Mode (#0) - Serial # 1107.464.301.0181
Registered to Rui Miguel Ferreira (University of Minho)

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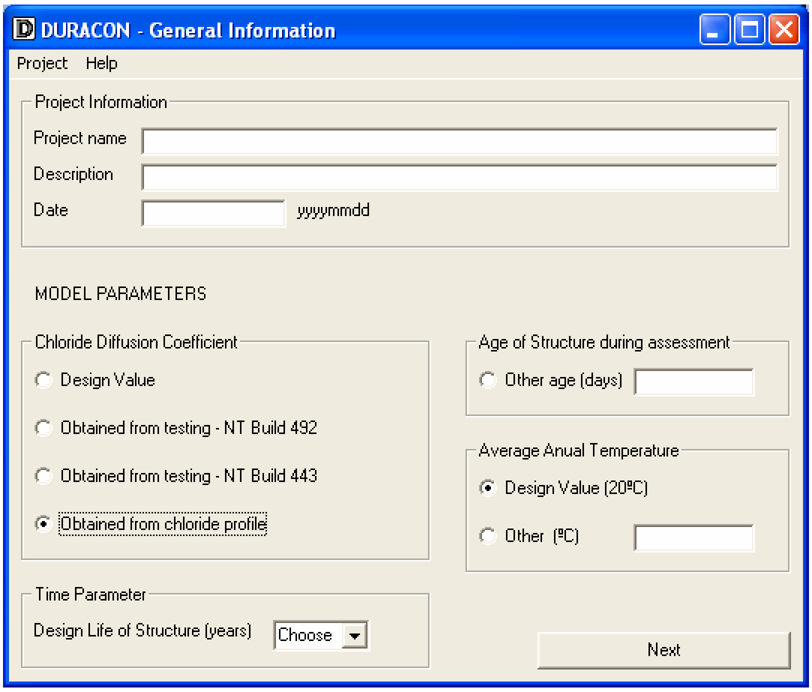
Figure 4.1 – Introduction window for DURACON.

The software is composed of three different data windows: general information, distribution data and profile parameters (see Figures 4.2 - 4.4).

In the General Information window, the project information is introduced as well as the model parameters. The project information is composed of the project name, a brief description and the date of the project. The model parameters are divided into four sections: definition of diffusion coefficients, definition of the time parameter, definition of the age parameter and the definition of the average annual temperature.

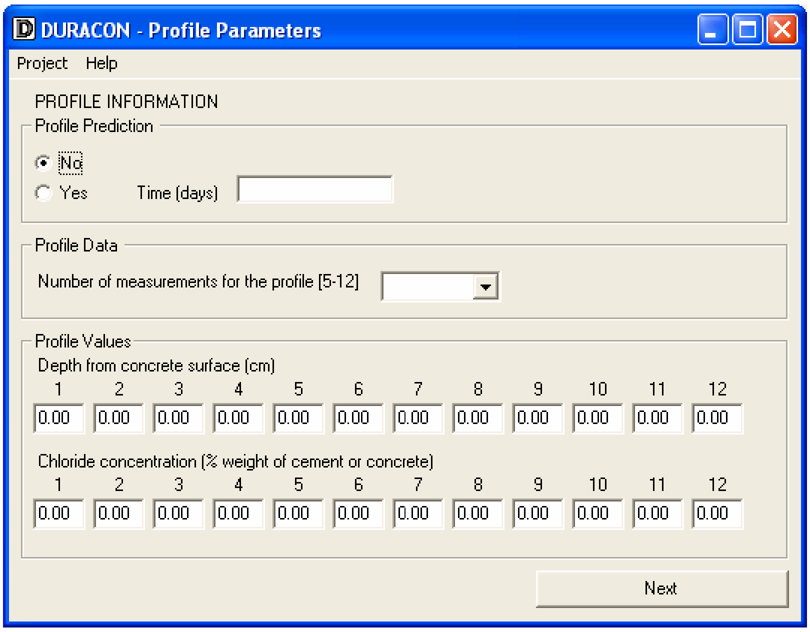


Chloride Diffusion Coefficient:
○ Design Value
○ Obtained from testing - NT Build 492
○ Obtained from testing - NT Build 443
● Obtained from chloride profile

Average Anual Temperature:
● Design Value (20ºC)
○ Other (ºC)

Figure 4.2 – General Information window.

The profile parameters window is where the user defines the information related to the profiles collected, and the profiles to be predicted.



● No
○ Yes Time (days)

Number of measurements for the profile [5-12]

Depth from concrete surface (cm)

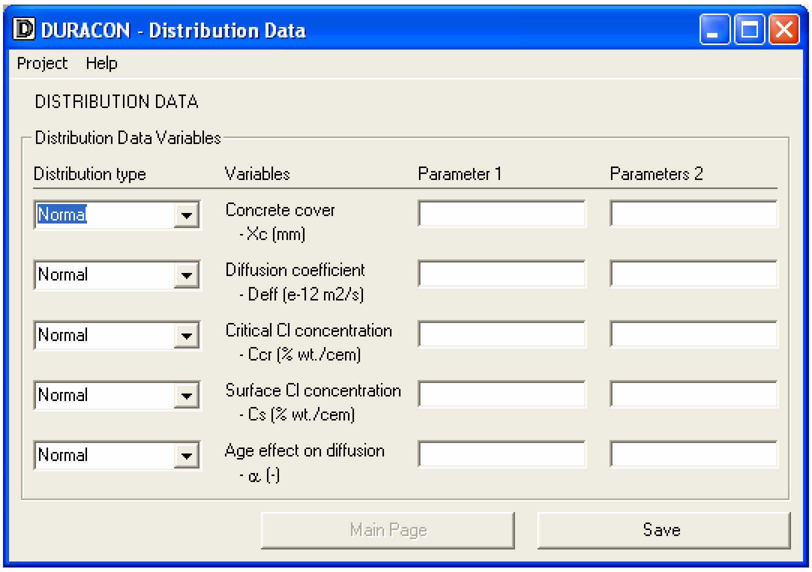
1	2	3	4	5	6	7	8	9	10	11	12
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Chloride concentration (% weight of cement or concrete)

1	2	3	4	5	6	7	8	9	10	11	12
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Figure 4.3 – Profile parameters window.

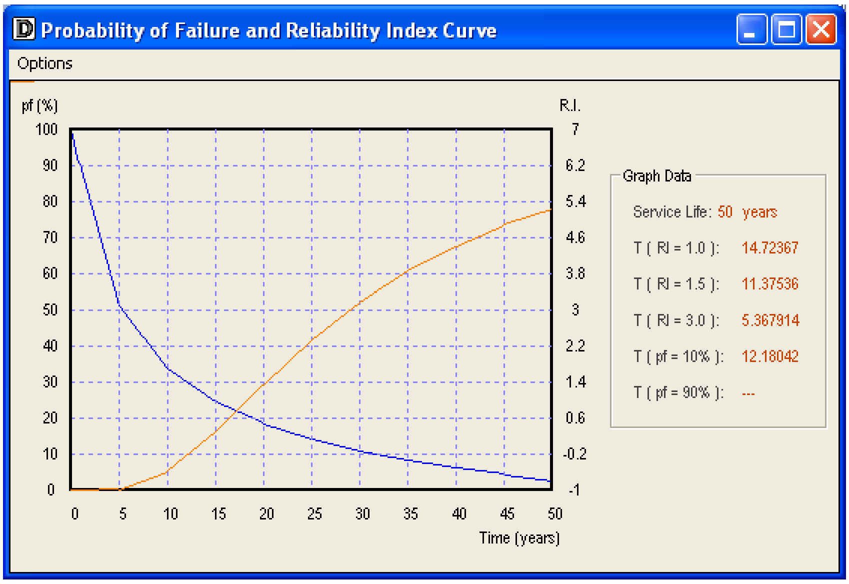
In the data distribution window, all the durability parameter data is introduced. Different distribution types can be chosen for each parameter.



Distribution type	Variables	Parameter 1	Parameters 2
Normal	Concrete cover - Xc (mm)		
Normal	Diffusion coefficient - Deff (e-12 m2/s)		
Normal	Critical Cl concentration - Ccr (% wt./cem)		
Normal	Surface Cl concentration - Cs (% wt./cem)		
Normal	Age effect on diffusion - α (-)		

Figure 4.4 – Distribution Data window.

Once all the necessary information has been introduced, the software executes the model simulation. The software presents the results in graphs in addition to the result files. Figure 4.5 is an example of the graphs presented.



Graph Data

Service Life: 50 years

T (RI = 1.0): 14.72367

T (RI = 1.5): 11.37536

T (RI = 3.0): 5.367914

T (pf = 10%): 12.18042

T (pf = 90%): ---

Figure 4.5 – Probability of Failure and Reliability Index vs. time.

4.3 Evaluation of the proposed model's parameters

The sensitivity analysis of the proposed model's parameters is performed by systematic analysis of the effects of changes in a variable on our outcome of the simulation. The parameters are varied over a likely range of values. The equation used to model probabilistically the durability and simulate the chloride concentration based on a time dependent diffusion coefficient with the effect of temperature is described in Chapter 3. The analysis is applied to the following model parameters:

- x_C - concrete cover (m);
- D_0 - diffusion coefficient at the time t_0 (m^2/s);
- C_{CR} - critical chloride threshold content (wt.% Cl^-/binder);
- C_S - surface chloride content in (wt.% Cl^-/binder);
- α - age factor that takes the influence of age of D_0 into account;
- T - temperature in (°C);

4.3.1 Reference case definition

Three distinct reference case situations are defined. The analysis of the sensitivity of the model parameters in these reference cases allows for conclusions to be drawn regarding the effect of the parameter variation on the outcome of the simulation, hence, the structures performance.

In the first reference case, the average values of the model parameters are varied over a valid range, while the standard deviations are constant. Table 4.1 shows the parameter information needed for the simulation. In bold are the initial values of the parameters for each simulation.

Table 4.1 – Value of variables for simulation for reference case 1.

Variable	Distribution Type	Parameter 1 - Average	Parameter 2 - Standard deviation
x_C (mm)	N (normal)	35, **45**, 55, 65, 75, 85	10
D_{COEF} (e-12.m^2/s)	N (normal)	1, 3, 6, 9, **12**, 15	2.5
C_S (% /weight concrete)	N (normal)	0.4, 0.6, 0.8, **1.0**, 1.2	0.3
C_{CR} (%/weight concrete)	N (normal)	0.06, **0.08**, 0.10, 0.15, 0.2	0.02
α	N (normal)	0.3, **0.4**, 0.5, 0.6, 0.7, 0.8	0.06
t_0 (days)	D (deterministic)	28	---
T (ºC)	N (normal)	1, 7, 14, **21**, 28	2

In the second reference case, an average value for the model parameters is chosen from the range presented in Table 4.1, and the standard deviation is varied. This is obtained by varying the coefficient of variation (CoV) over a valid range, from 5 % to 30 %. Table 4.2 shows the parameter information needed for the simulation.

Table 4.2 – Value of variables for simulation for reference case 2.

Variable	Distribution Type	Parameter 1 - Average	Parameter 2 - Standard deviation (CoV - %)
x_C (mm)	N (normal)	55	55 (5,10,20,30)
D_C (e-12.m^2/s)	N (normal)	9	9 (5,10,20,30)
C_S (% /weight concrete)	N (normal)	0.8	0.8 (5,10,20,30)
C_{CR} (% /weight concrete)	N (normal)	0.1	0.1 (5,10,20,30)
α (-)	N (normal)	0.5	0.5 (5,10,20,30)
t_0 (days)	D (deterministic)	28	---
T (ºC)	N (normal)	15	15 (5,10,20,30)

In the third reference case, the effect of low quality (LQ) and high quality (HQ) of concrete is analysed at an age of 50 years, varying the concrete reinforcement cover from 5 to 125 mm. Table 4.3 shows the values used in the simulation. A CoV of 10 and 30% was used for each of the concrete qualities. In this way, the effect of different quality concretes on the performance is analysed, each with a large and small scatter of the parameter in question.

Table 4.3 - Value of variables for simulation for reference case 3.

Variable	Dist. Type	High Quality		Low Quality	
		Parameter 1	Parameter 2	Parameter 1	Parameter 2
D_{COEF} ($1e^{-12}.m^2/s$)	N (normal)	3.5	0.7	10.0	2.5
C_S (% /wt.concrete)	N (normal)	1.0	0.3	1.0	0.3
C_{CR} (% /wt.concrete)	N (normal)	0.08	0.008	0.07	0.007
α (-)	N (normal)	0.4	0.04	0.4	0.04
t_0 (days)	D (determ.)	28	---	28	---
T (ºC)	N (normal)	21	2	21	2
x_C (mm)	variable from 5 - 125 mm depth				
t (years)	50 years				

4.3.2 Concrete cover

The average concrete cover parameter was varied from 35 mm to 85 mm with a 10 mm standard deviation. The minimum value chosen is smaller than the minimum requirement for concrete cover in standards for marine environments. The 10 mm standard deviation is based on the value given for the deviation of x_C in ENV 13670-1, for Portugal. From figure 4.6, the effect of concrete cover on service life is clearly noticeable. Small increases in concrete cover vary significantly the performance of the concrete through out the service life. This effect is even greater at earlier ages, for example 10 years, where the difference in probability of failure between a concrete cover of 35 mm and 55 mm is almost 50 %.

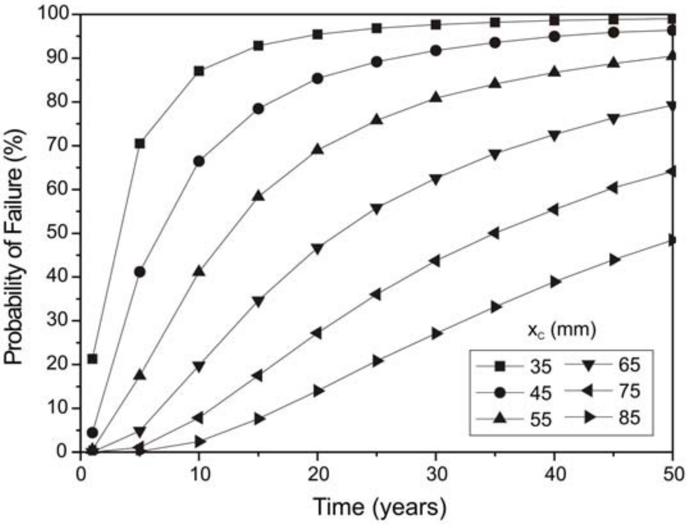


Figure 4.6 – Effect of various concrete covers (Xc) with time.

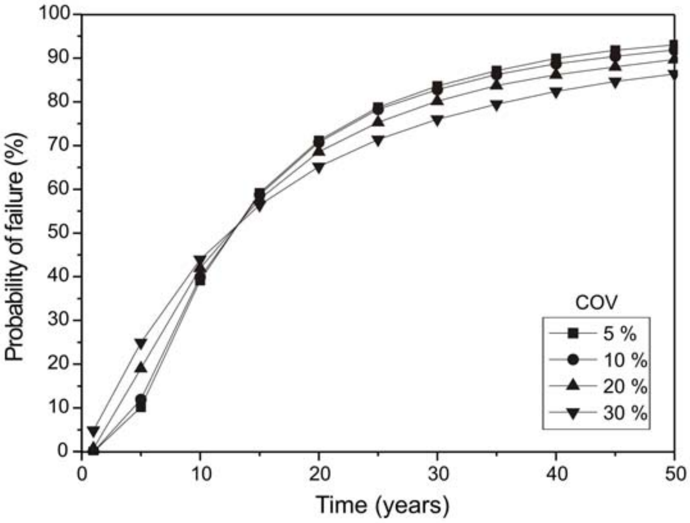


Figure 4.7 – Influence of concrete cover scatter on the probability of failure.

The influence of the scatter on the concrete cover measurements on the probability of failure can be observed in figure 4.7. With a CoV of up to 30 %, the final result, i.e the probability of failure is affected by approximately 10 %. The greater the CoV, the lower the probability of failure is, for ages greater than 12 years. Before this age, the trend is the opposite.

4.3.3 Diffusion coefficient

The average diffusion coefficient parameter was varied from 1.0 x 10^{-12} m^2/s to 15.0 x 10^{-12} m^2/s with a 10 % CoV. These two extreme values represent extremely high and low concrete qualities according to Nilsson (1998). Figure 4.8 shows the importance of the diffusion coefficient as a parameter for service life design. The differences between 1.0e^{-12} m^2/s and 15.0e^{-12} m^2/s are significant. An increase of one order of magnitude in the diffusion coefficient results in an improvement of the performance by at least 90 % at 50 years. This effect is still large at earlier ages, for example 10 years, where the difference in probability of failure is almost 55 %.

From figure 4.9, the influence of the scatter of the diffusion coefficient on the performance can be observed. A CoV up to 30% influences the final result, i.e the probability of failure, by approximately 5 %. A pattern seems to appear, the higher the scatter the lower the probability of failure.

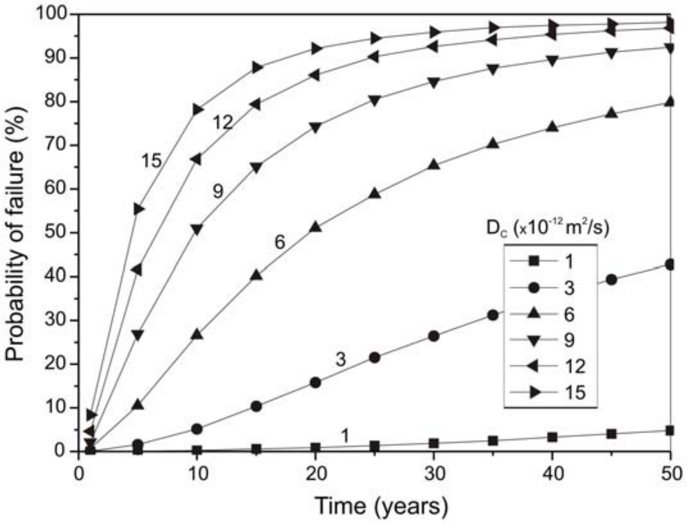


Figure 4.8 – Effect of various diffusion coefficient with time.

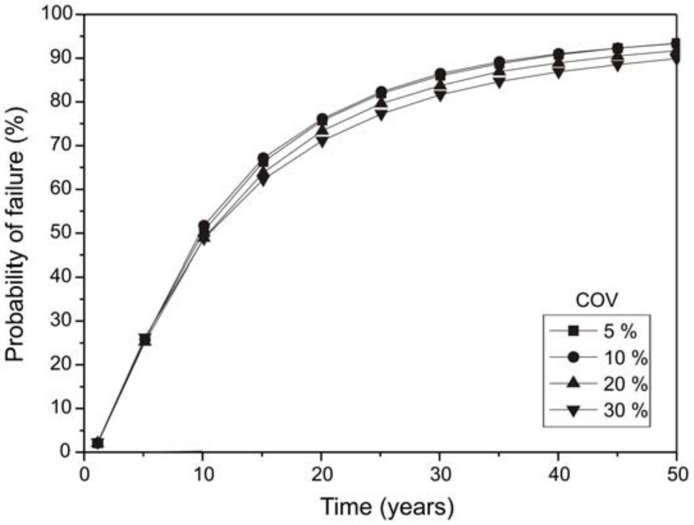


Figure 4.9 – Influence of diffusion coefficient scatter on the probability of failure.

In figure 4.10, a comparison is made between the performance of a high and low quality concrete at the age of 50 years. The diffusion coefficients used were 3.5e^{-12} m^2/s and 10.0e^{-12} m^2/s, respectively. If the structure is assumed to have a 50 mm concrete cover, the low quality concrete cover would have a probability of failure of 100 % meaning that corrosion has started. The high quality concrete cover would have a probability of failure of approximately 35 %. According to the Norwegian Standard for requirements to reliability in design of structures (NS 3490 1999), the probability of failure for a serviceability limit state should not exceed 10 %, and according to Eurocode 1 (2002) the probability of failure for a serviceability limit state should not be more than approximately 7 %. If the 10 % limit is adopted, the low quality concrete would require a concrete cover of approximately 105 mm to reach this limit after 50 years. The high quality concrete would only require approximately 60 mm. This is a huge difference that influences the design of a structure drastically.

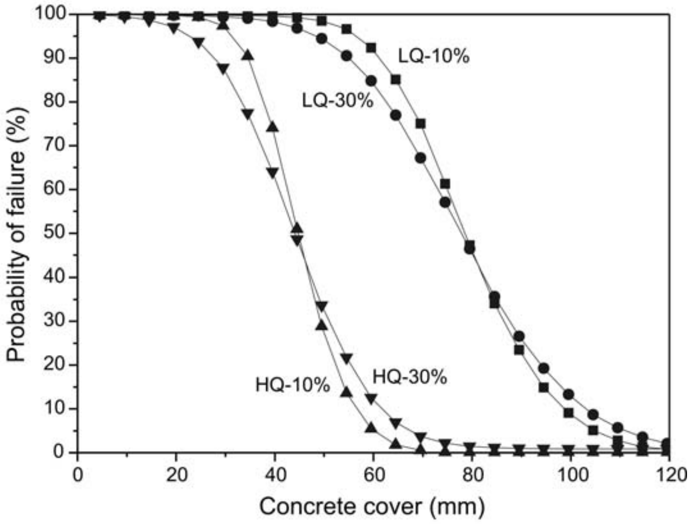


Figure 4.10 – Comparison of the performance of high quality and low quality concrete (influence of the diffusion coefficients) at various concrete cover depths after 50 years.

The influences of the diffusion coefficient scatter (CoV of 10 % and 30 %) on the performance of the concrete result in the need to increase the concrete cover by approximately 5 mm in concrete cover, if the scatter is increased.

4.3.4 Critical chloride content

The average critical chloride thresholds were varied from 0.06 % to 0.2 % by weight of concrete with a 10 % CoV. These values correspond to the limits of values presented in literature, irrespective of the concrete type. It is clear that this parameter, although still important, does not influence the performance as much as the concrete cover or the diffusion coefficient (Figures 4.6 and 4.8 respectively). The probabilities of failure curves are further apart in Figures 4.6 and 4.8 than in Figure 4.11, indicating greater influence on the performance. This separation is maintained until 50 years. After 50 years, the influence in the probability of failure between the maximum value and the minimum value is 20%. However, at 10 years, the difference is large being approximately 45 %. This suggests that with time, the effect of the different critical chloride thresholds become less significant. This is due to the fact that with time, ever more parts of the structure begin to deteriorate and corrode.

The influence of the scatter of the critical chloride thresholds on the performance is observed to be negligible (see Figure 4.12). This suggests that this parameter can be considered to be deterministic. Its quantification is still vital, while the scatter seems to have no influence.

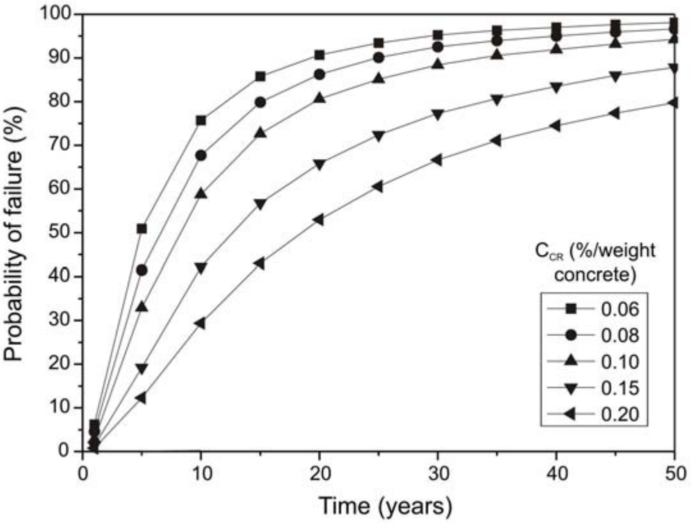


Figure 4.11 – The effect of different critical chloride thresholds with time.

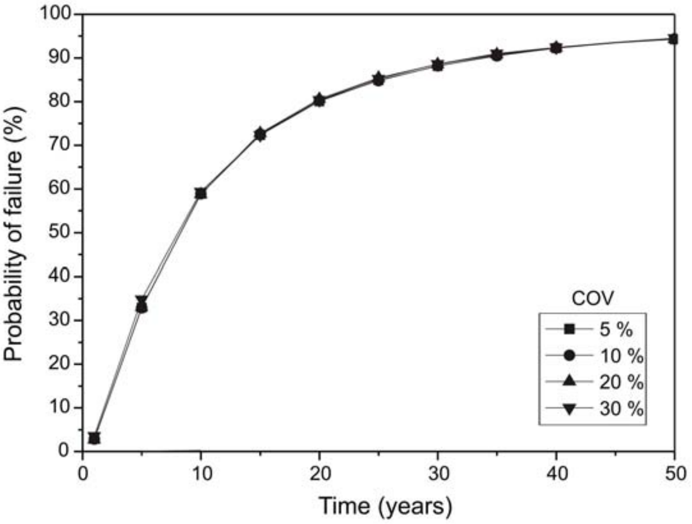


Figure 4.12 – Influence of the critical chloride thresholds scatter on the probability of failure

In figure 4.13, a comparison is made between the performance of a high and low quality concrete at the age of 50 years. The critical chloride thresholds used were 0.08 % and 0.07 % respectively, by weight of concrete, with CoV of 10 %. If the structure is assumed to have a 50 mm concrete cover, the low quality concrete cover would have a probability of failure of over 95 %, where as, the high quality concrete cover would be approximately 30 %. According to the Norwegian Standard for requirements to reliability in design of structures (NS 3490, 1999), the probability of failure for a serviceability limit state should not exceed 10 %, and according to Eurocode 1 (2002) the probability of failure for a serviceability limit state should not be more than approximately 7 %. If the 10 % limit is adopted, the low quality concrete would require a concrete cover of approximately 100 mm to reach this limit after 50 years. The high quality concrete would only require approximately 60 mm. This is a significant difference that can influence the design of a structure markedly.

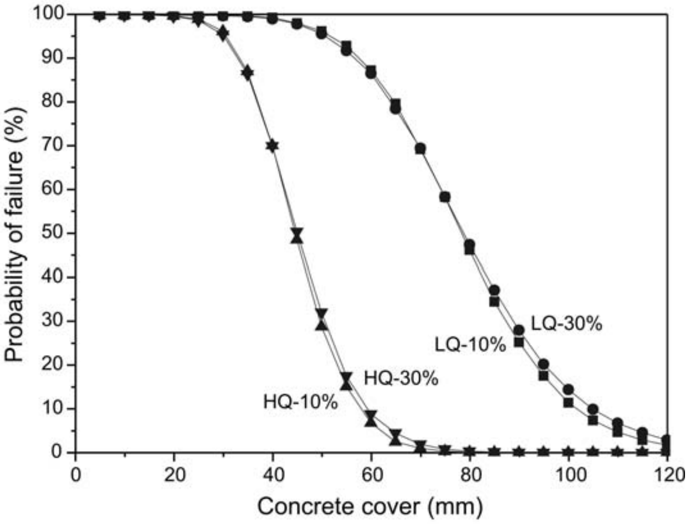


Figure 4.13 - Comparison of the performance of high quality and low quality concrete (influence of the critical chloride thresholds) at various concrete cover depths after 50 years.

The influence of the critical chloride content scatter (CoV of 10 % and 30 %) in the performance of concrete results in the need to increase the concrete cover by approximately 2-3 mm. This indicates that the scatter of the critical chloride content has little bearing on the concrete cover in the proposed model (see Figure 4.13).

4.3.5 Surface chloride content

The average surface chloride content is varied between 0.4 % and 1.2 % by weight of concrete, with a 10 % CoV. These values correspond to a moderately low concentration of chloride ions, which can be associated with brackish waters, and a moderately high concentration of chloride ions.

It is clear from Figure 4.14 that this parameter, although still important, does not influence the performance as much as the concrete cover (figure 4.6) or the diffusion coefficient (figure 4.8). After 50 years, the influence in the probability of failure between the maximum value and the minimum value is 18%, however, at 10 years, the difference is large, approximately 42 %. This is probable obvious because the lower the surface chloride content, longer time is needed for the chloride to penetrate the concrete cover in sufficient quantities to initiate corrosion. Therefore, at 50 years, the values are closer to one another than at 10 years.

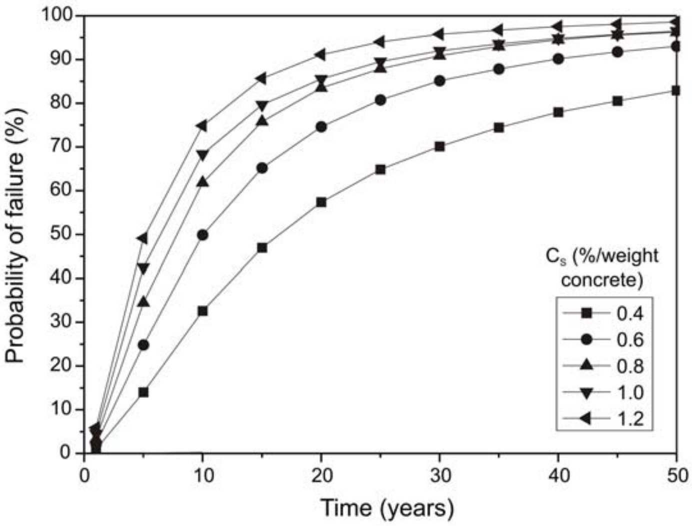


Figure 4.14 - The effect of different surface chloride content on the probability of failure with time.

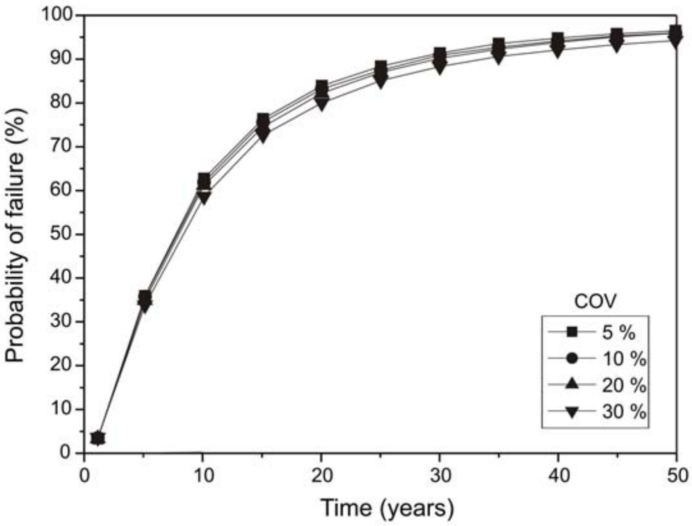


Figure 4.15 – Influence of the scatter of surface chloride content on the probability of failure

From figure 4.15, the influence of the scatter of the surface chloride content on the performance is relatively small, approximately 5 %.

In figure 4.16, a comparison is made between the performance of a high quality and a low quality concrete, at the age of 50 years. The surface chloride content used was 1.0 % by weight of concrete. If the structure is assumed to have a 50 mm concrete cover, the low quality concrete cover would have a probability of failure of 100 %, indicating that corrosion has started, whereas the high quality concrete cover would be less than 30 %.

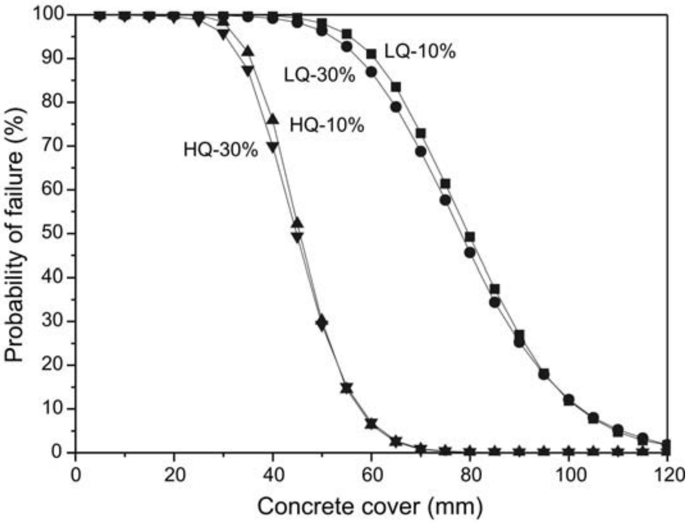


Figure 4.16 - Comparison of the performance of high quality and low quality concrete (influence of the surface chloride content) at various concrete cover depths after 50 years.

If a 10 % probability of failure for a serviceability limit is adopted, the low quality concrete would require a concrete cover of approximately 102 mm to reach this limit after 50 years. The high quality concrete would only require approximately 57 mm. This is a significant difference that can influence the design of structures markedly.

The influence of the scatter of surface chloride content (CoV of 10 % and 30 %) on the performance of the concrete results in the need to increase the concrete cover by approximately 3-5 mm (Figure 4.16). Although small, this value is large enough to influence the design.

4.3.6 Age factor

The age factor α of the diffusion coefficient was varied between 0.3 and 0.8, with a 10 % CoV. This corresponds to an intermediate range of values since the limiting values for the age factor are 0.0 and 1.0. Figure 4.17 indicates that the age factor influences the performance significantly even more than the concrete cover (figure 4.6) or the diffusion coefficient (figure 4.8). After 50 years, the influence in the probability of failure between the maximum value and the minimum value is 95%. At 10 years, the difference is still very large, approximately 90 %.

For this reason much care must be taken in choosing this parameter. From figure 4.18, the influence of the scatter of the age factors of diffusion coefficient on the performance is the largest observed, approximately 15 %. If determining the probability of failure for 50 years, the greater the scatter of the age factor measurements the lower the probability of failure will be.

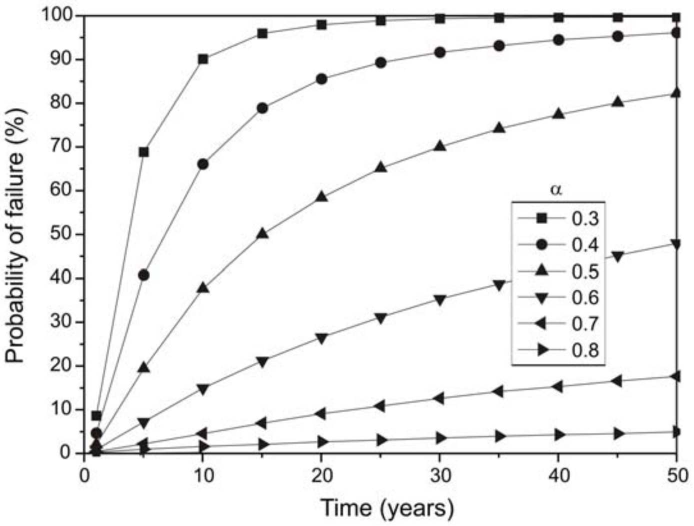


Figure 4.17 – The effect of age factors of diffusion coefficient on the probability of failure with time.

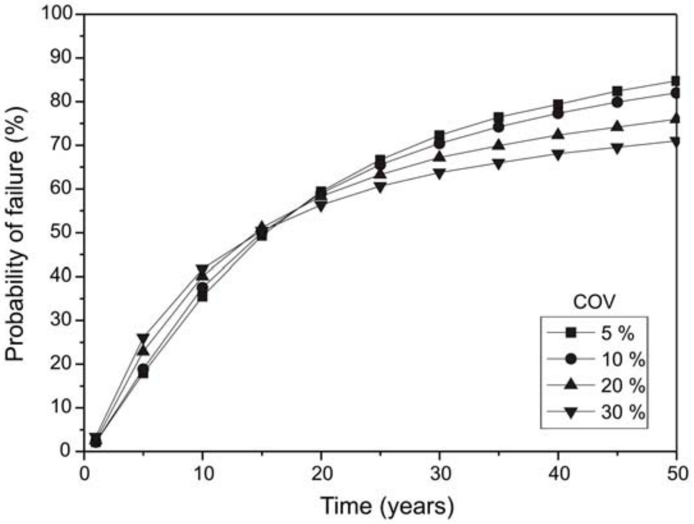


Figure 4.18 - Influence of the scatter of the age factors of diffusion coefficient on the probability of failure.

In figure 4.19, a comparison is made between the performance of a high and low quality concrete at the age of 50 years. The age factor used was 0.4. If the structure is assumed to have a 50 mm concrete cover, the low quality concrete cover would have a probability of failure of 85-95 % where as the high quality concrete cover would be approximately 30-40 %. If the 10 % probability of failure for a serviceability limit is adopted, the low quality concrete would require a concrete cover of approximately 100 mm and over 140 mm for the 10% CoV and the 30% CoV, respectively, to reach this limit after 50 years. The high quality concrete would only require approximately 55 mm or a 75 mm of concrete cover, for the 10% CoV and the 30% CoV, respectively. Not only the difference is significant between the concrete qualities, but also the effect of the scatter of the age factor is high for the same concrete quality.

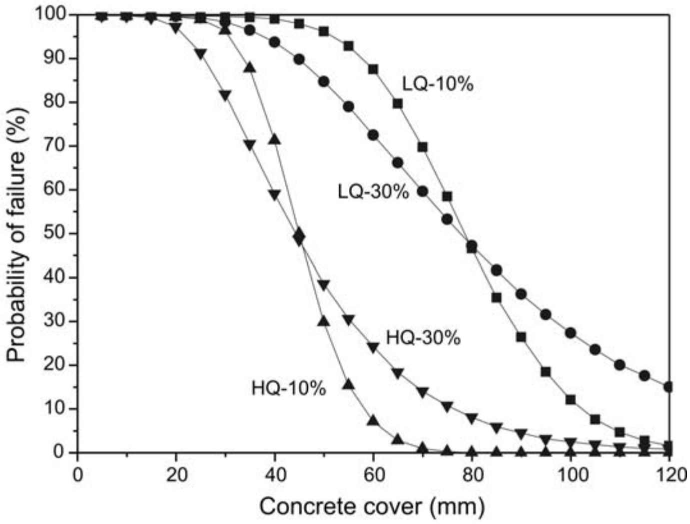


Figure 4.19 - Comparison of the performance of high quality and low quality concrete (influence of the diffusion coefficient age factors) at various concrete cover depths after 50 years.

The influences of the scatter of the age factor of diffusion coefficient (CoV of 10 % and 30 %) on the performance of concrete result in the need to increase the concrete cover by approximately 20-25 mm. Hence, the need to quantify adequately the age factor is crucial.

4.3.7 Temperature

The average temperature parameter was varied from 0 ºC to 28 ºC with a 10 % CoV. This is the normal range of average temperature for cold and hot climates. Figure 4.20 indicates that temperature, although still important, does not influence the performance as much as the concrete cover (figure 4.6) or the diffusion coefficient (figure 4.8). After 50 years, the influence in the probability of failure between the maximum value and the minimum value is 40 %. At 10 years, the difference is still higher, approximately 70 %. With time, the effect of temperature on the rate of the probability of failure decreases.

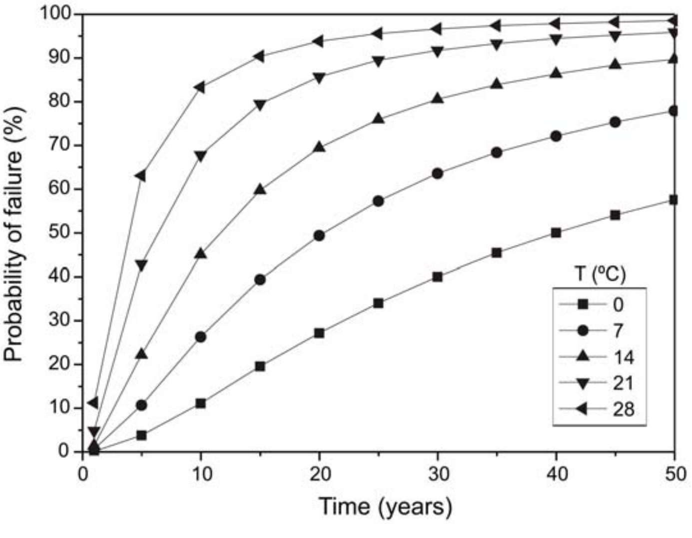


Figure 4.20 - The effect of different temperature with time.

Figure 4.21 shows that the influence of the scatter of the temperature on the performance is considerable, approximately 10 %. It further indicates that the higher the scatter of the parameter, the lower the probability of failure.

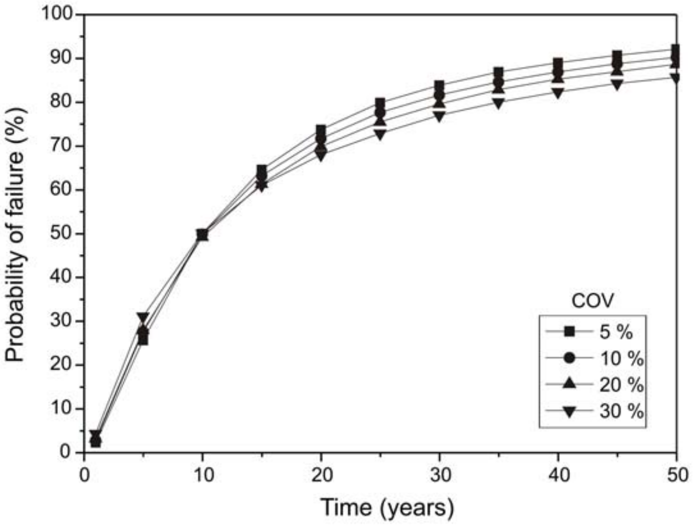


Figure 4.21 - Influence of the temperature scatter on the probability of failure

In figure 4.22, a comparison is made between the performance of a high and low quality concrete at the age of 50 years. The temperature used was 21 ºC. If the structure is assumed to have a 50 mm concrete cover, the low quality concrete cover would have a probability of failure of 95 % where as the high quality concrete cover would be approximately 35 %.

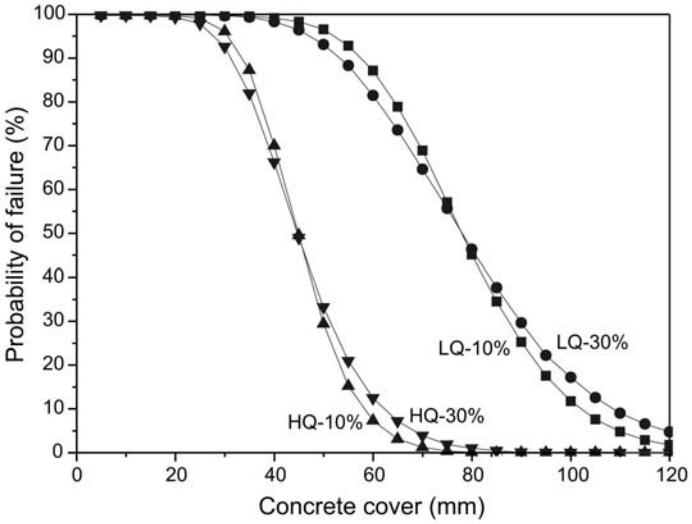


Figure 4.22 - Comparison of the performance of high quality and low quality concrete (influence of the temperature) at various concrete cover depths after 50 years.

If the 10 % probability of failure for a serviceability limit is adopted, the low quality concrete would require a concrete cover of approximately 105 mm to reach this limit after 50 years. The high quality concrete would only require approximately 65 mm. This is a considerable difference that can influence the design of a structure markedly.

The influence of the temperature scatter (CoV of 10 % and 30 %) on the performance of the concrete results in the need to increase the concrete cover by approximately 5 mm.

4.4 Summary

The software developed manages to aids the designer to identify the most important parameters and illustrate the sensitivity of concrete durability towards each one.

It is the actual use of the model in the software that gives the critical advantage in the design/assessment procedure of a concrete structure. The output received from the software informs the user what influence a change in one or more parameters will have on the performance of the structure with time. Normally, a designer would have to rely on the information available in standards and, more importantly, on the experience gained with past structures. Even an experienced designer would have difficulty in evaluating the influence of many changes in the performance, not to mention the use of new materials.

The sensitivity analysis helps the user of the software to understand how each variable influences the performance of the concrete structure, and how susceptible to change each variable is. As a resume of the sensitivity analysis presented in this chapter, Table 4.4 shows how the stochastic parameters influence the performance of the concrete structure. The effect of an increase of a parameter (either average or standard deviation) on concrete performance is presented in the following table.

Table 4.4 - Effect of an increase of a parameter on concrete performance.

Parameter	Average value	Standard Deviation
Concrete cover (m)	Improves greatly	Slight positive effect
Diffusion coefficient (m^2/s)	Improves greatly	Slight positive effect
Critical chloride threshold content (wt.% Cl^-/binder)	Improves	No effect
Surface chloride content in (wt.% Cl^-/binder)	Improves	Slight negative effect
α - age factor of D_0	Improves greatly	Large negative effect
Temperature in (ºC)	Improves	Negative effect

CHAPTER 5
APPLICABILITY OF PROPOSED MODEL FOR DURABILITY ANALYSIS

5.1 General

The proposed model has primarily been developed in order to provide a general basis for durability analysis of concrete structures in marine environment. Such a durability analysis can partly be applied for obtaining a more controlled durability and long-term performance of new concrete structures, and partly be used as an improved basis for condition assessment of existing concrete structures in marine environment. In the following, a brief outline of both of these applications will be given.

5.2 Durability analysis for design of new concrete structures

Often, when designing, the designer must make several assumptions concerning the information used in the simulation. Values for diffusion coefficients, parameters that quantify the environment, etc. must be defined. More often than not, relevant information does not exist. Therefore, the necessary guidelines are described in the following, to assist the designer in the design process. For more detailed information, see Appendix 3 the *Recommendations for probability-based durability design of concrete structures*. This document has been prepared to be used in conjunction with the developed software.

Defining the service life of the structures

According to the EN 1990 – Basis for structural design (2002), the service life of the structure can be defined according to the type of structure.

Table 5.1 – Indicative design working life

Design life category	Indicative design life (years)	Examples
1	10	Temporary structures
2	10 to 25	Replaceable structural parts (e.g. gantry girders, bearings)
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures

It should be noted that the structure should be designed in such a way that deterioration over its design working life does not impair the performance of the structure below that intended, regarding the environment and the anticipated level of maintenance. Therefore, if limit states other than the ultimate are defined, the service life must be defined according to these limit states.

Defining the diffusion coefficient

For initial estimation of the diffusion coefficient based on the w/c ratio, equation (5.1) can be used (HETEK 53, 1996),

$$D_{RCM} = k_1 \cdot \exp\left(-\sqrt{\frac{k_2}{m}}\right) \tag{5.1}$$

$$m = \frac{w}{c + k_3 \cdot s} \tag{5.2}$$

where

D_{RCM} – is the coefficient for the Rapid Chloride Migration method;

k_1 – constant (570000 mm^2/year or 1.80746e-08 m^2/s);

k_2 – constant (23);

m – water /binder ratio calculated according to EN206-1 (2002) to account for the effect of additions.

w – water content of the concrete mix;

c – cement content of the concrete mix;

s – addition content of the concrete mix;

k_3 – k-value parameter to account for type II additions (EN206-1 2002).

Table 5.2 gives an idea of the values obtained for various combinations, assuming a CEM I 42.5 is used.

The diffusion coefficient needs not necessarily be determined by the above described procedure. It can be based on experience or obtained from available sources.

Defining the critical chloride concentration

A realistic range for initiation conditions of pitting corrosion, based on tests by (Breit 1997, Hansson 1990), was determined between total chloride contents of 0.20 and 0.80 % chloride by weight of binder. The value suggested in literature (Duracrete 2000) for probabilistic evaluation is N(0.48, 0.15) % chloride by weight of binder, or N(0.07, 0.02) % chloride by weight of concrete for an OPC concrete with about 350 kg/m^3. *N* indicates the type of distribution, in this case, normal, and the values in between brackets are the average and the standard deviation.

Table 5.2 – Initial estimation of the diffusion coefficient.

c kg/m^3	w l/m^3	Additions kg/m^3	s kg/m^3	k_3	m	D 10^{-12} m^2/s
370	166.5	---	0	0	0.45	14.2
370	148	---	0	0	0.40	9.2
370	129.5	---	0	0	0.35	5.5
370	203.5	15%PFA	55.5	0.4	0.52	23.2
370	185	15%PFA	55.5	0.4	0.47	16.8
370	148	15%PFA	55.5	0.4	0.38	7.4
370	185	7%SF	18.5	1	0.48	17.3
370	166.5	7%SF	18.5	2	0.41	10.0
370	148	7%SF	18.5	2	0.36	6.4

Defining the age factor for the diffusion coefficient

The age factor can be chosen by consulting Table 3.1. The main factors that affect the age factor are the same as those that affect the diffusion coefficient, namely the cement type and the environmental exposure conditions. Care should be taken in distinguishing which elements of the structure will be the most severely exposed. The position of the structure in relation to the surrounding environment, the water run off, the protection from rain and other similar considerations should be taken into account.

Estimating the environmental loading

The concentration of chloride in the surrounding environment is highly dependant on many factors, such as local climate and location of the construction. For this reason, each case must be treated individually and as much local information as possible should be collected.

The penetration of chlorides into the concrete depends on the surface concentration of chlorides. If possible, this information should be collected from the surrounding structures so that a realistic notion of the values is obtained. Once again, experience and available data is important in choosing adequate values.

A basis for the estimation of C_S can be found in (HETEK 83 1997, Breit 1997, Fluge 2000). The surface concentration is described as a function of w/b and an error term (Duracrete 2000).

$$C_{SN} = A\left(w/b\right) + \varepsilon \tag{5.3}$$

The quantification is made by a Bayesian regression analysis to find a relationship between w/b and C_{SN}. The parameter A is a constant that results from the analysis and ε is the error-term. Values for A and ε be can be found in Duracrete (2000).

For conversion of values from *Cl%* by weight of concrete into *Cl%* by weight of binder and vice versa, figure 5.1 can be used for an OPC concrete with density of approximately 2400kg/m^3.

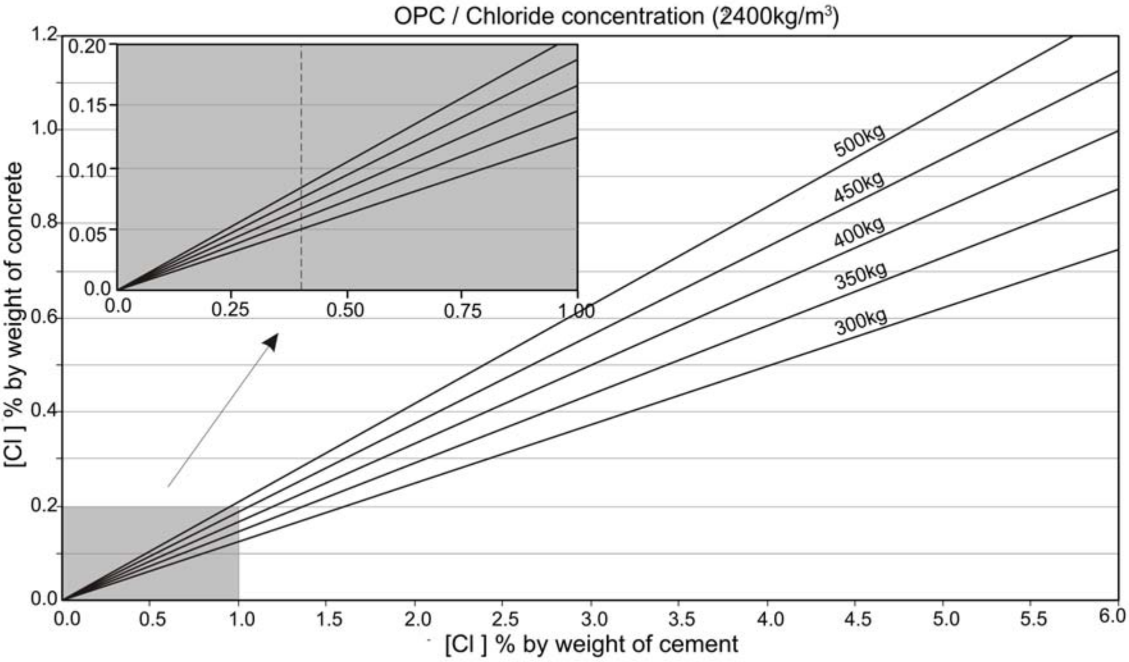


Figure 5.1 – Relationship between Cl% by weight of concrete and Cl% by weight of binder (Ferreira 2002).

Defining the concrete cover

The concrete cover x_C, is usually based on the values estimated in the design phase. Consideration should be given, during the design phase, to the values presented in the Eurocode2 (2002) and the NS 3473 (2003) for minimum concrete cover according to the environmental exposure class in question (See 2.6).

The concrete cover is the distance between the surface of the concrete and the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant).

The nominal cover should be specified in the design project. It is defined as a minimum cover, c_{MIN}, plus an allowance in design for deviation, Δc_{DEV} (Eurocode 2 - 2002):

$$c_{NOM} = c_{MIN} + \Delta c_{DEV} \tag{5.4}$$

Due to the quality of the workmanship, the placement of the concrete and other factors, a standard deviation of between 5 and 15 mm can be adopted. The value of x_C is assumed to be normally distributed.

From a statistical point of view, the minimum cover has to be associated with a probability of failure, meaning that a certain percentage of the reinforcement has a smaller cover than the required minimum. A reasonable assumption may be that 5 % of the reinforcement has a lower cover than the required minimum. Assuming that the concrete cover is normally distributed, the distance from the mean value to the 5 % percentile is 1.645 times the standard deviation, giving the standard deviation, σ_R:

$$\sigma_R = \Delta c_{DEV} / 1.645 \qquad (5.5)$$

In the calculations, the average concrete cover, μ_R, will therefore be set by equation 5.4, and the standard deviation, σ_R, as the design assumption for new structures (NS 3473 2003, NS 3465 2003).

Table 5.3 - Values of minimum cover, $c_{MIN,DUR}$, requirements with regard to durability for reinforcement steel (prEN 1992.1-1 2002).

Structural Class	Exposure Class						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
1	10	10	10	15	20	25	30
2	10	10	15	20	25	30	35
3	10	10	20	25	30	35	40
4	10	15	25	30	35	40	45
5	15	20	30	35	40	45	50
6	20	25	35	40	45	50	55

5.2.1 Effect of design options on durability performance

In order to demonstrate how the software can be applied, some typical results of probability based durability analyses are given in the following. For a new concrete harbour structure, the overall requirement for safety and durability was based on a maximum risk level of 10 % for steel corrosion to occur during a service period of at least 100 years (Årskog 2004). At the same time, the minimum durability requirements according to the current concrete code with respect to w/c ratio, cement content and concrete cover also had to be fulfilled (Gjørv 2002).

Two different cases of durability analysis are given:

- In the first case, the objective was to select a suitable type of cement or binder system in order to produce a concrete with adequate resistance against chloride penetration (chloride diffusivity).

- In the second case, the objective was to find out how an increased concrete cover in the beams of the concrete deck would improve the risk level for corrosion to occur.

5.2.2 Effect of cement type on design

Based on three possible types of cement with and without addition of silica fume, some concrete test mixtures were produced in order to test the chloride diffusivity (NT Build 492 1999). The three types of cement included a high-performance Portland cement (CEM I 52.5 LA), a blended fly-ash cement (CEM IV/A) and a blast-furnace slag cement with approximately 70 % slag (CEM III/B 42.5 LH HS). The test mixtures without silica fume had a cement content of 420 kg/m^3 and a w/c ratio of 0.45. The test mixtures with silica fume had a cement content and silica fume content of 390 kg/m^3 and 39 kg/m^3, respectively, while the w/c ratio was 0.38 and 0.35 for k-factors of 1 and 2, respectively.

For all the concrete mixtures, the durability analysis was based on the same concrete cover and surface chloride concentration (exposure conditions). The average concrete cover was chosen in accordance with current standards (NS 3473 2003, Eurocode 2002), with an average value of 50 mm assuming a normal distribution and a standard deviation of 5 mm. The average value of surface chloride concentration was 5.4 % by weight of cement, also assuming a normal distribution with a standard deviation of 0.54 %.

Table 5.4 - Input parameters for analyzing the effect of cement type (Årskog 2004).

Concrete Quality	Durability Parameters		
	D_0 (e-12.m^2/s)	α	C_{CR} (% wt. cement)
Type 1 (CEM I 52.5 LA)	N(10.5;0.66) 1)	N(0.37;0.07)	N(0.48;0.15)
Type 2 (CEM I 52.5 LA + 10% CSF)	N(4.74;0.51)	N(0.39;0.07)	
Type 3 (CEM III/B 42.5 LH HS)	N(5.3;0.59)	N(0.60;0.15)	
Type 4 (CEM III/B 42.5 LH HS + 10% CSF)	N(1.42;0.09)		
Type 5 (CEM IV/A)	N(10.1;0.81)	N(0.51;0.07)	
Type 6 (CEM IV/A + 10% CSF)	N(5.5;0.86)		

1) Normal distribution

Based on the obtained chloride diffusivities and input parameters as shown in Table 5.4, the durability analysis was carried out with the results shown in Figure 5.2. For the pure Portland cement (Type 1), a service period of less than 5 years would be obtained, while a combination with 10 % silica fume (Type 2) would give an increased service period of just under 10 years before the risk level of 10 % for corrosion to occur would be reached. Based on the slag cement without (Type 3) and with silica fume (Type 4), service periods of approximately 32 years and more than 100 years, respectively, would be reached. For the fly-ash cement (Type 5), a service period similar to that of Type 2 would be reached, while a combination with 10 % silica fume would give a service period of somewhat more than 25 years. These results are roughly in accordance with practical experience (Gjørv 2002).

The results as obtained from the software for the six concrete mixtures are displayed in Figure 5.3.

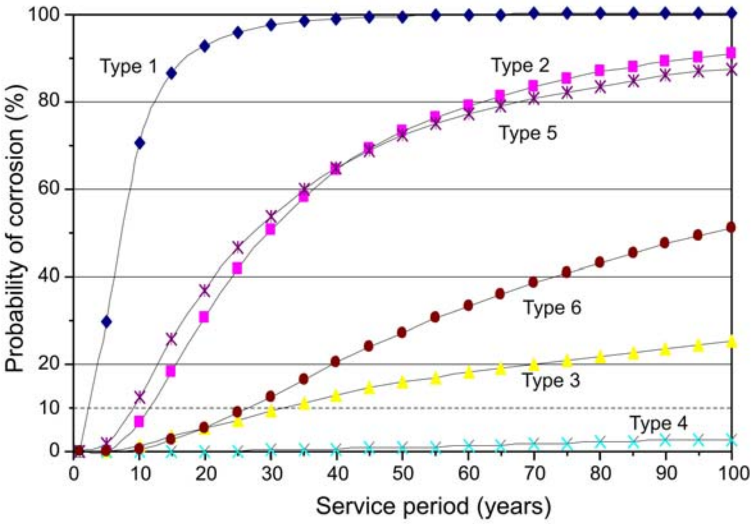
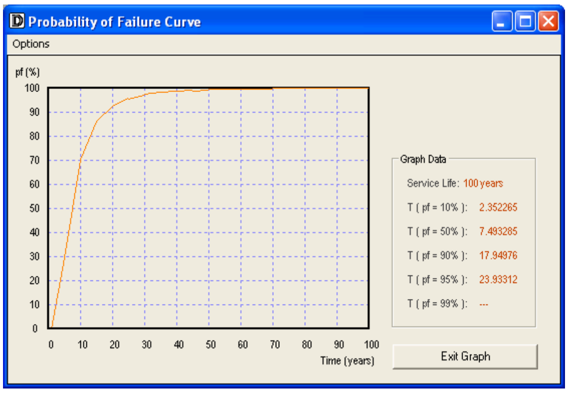


Figure 5.2 - Effect of cement type on probability of corrosion in a concrete harbour structure (Årskog 2004).



Graph Data

Service Life: 100 years

T (pf = 10%): 2.352265

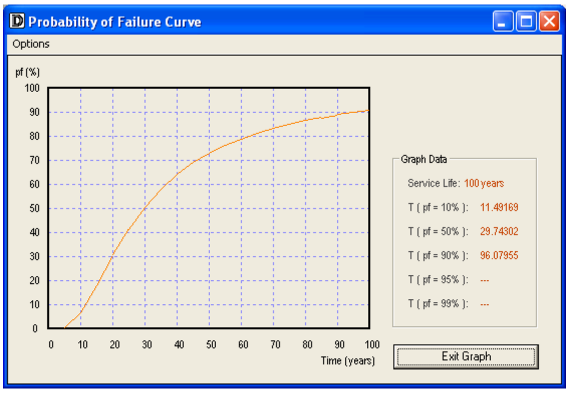
T (pf = 50%): 7.493285

T (pf = 90%): 17.94976

T (pf = 95%): 23.93312

T (pf = 99%): ---

a) Type 1



Graph Data

Service Life: 100 years

T (pf = 10%): 11.49169

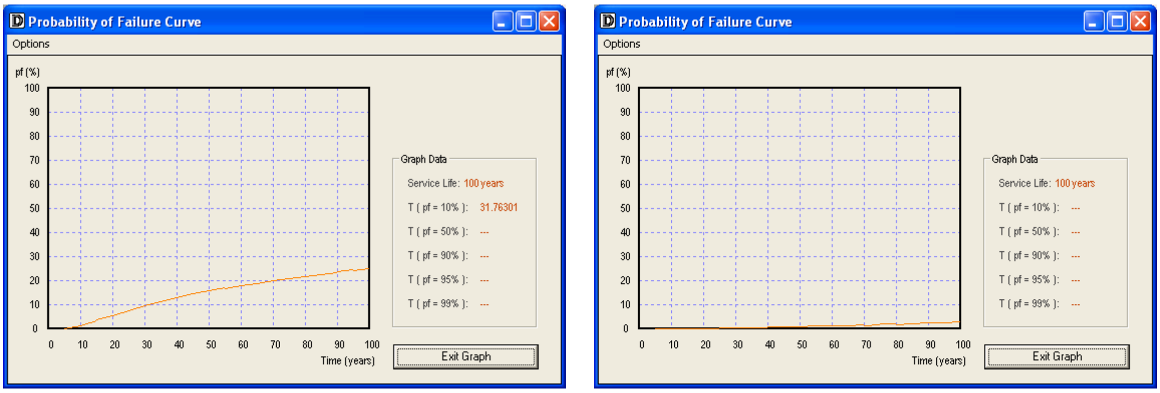
T (pf = 50%): 29.74302

T (pf = 90%): 96.07955

T (pf = 95%): ---

T (pf = 99%): ---

b) Type 2



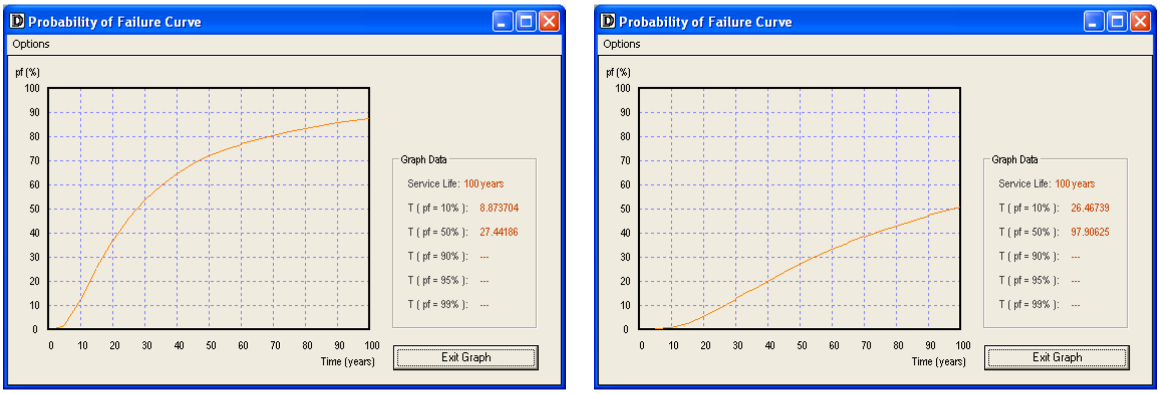
Graph Data

Service Life: 100 years
T (pf = 10%): 31.76301
T (pf = 50%): ---
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

Graph Data

Service Life: 100 years
T (pf = 10%): ---
T (pf = 50%): ---
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

c) Type 3 d) Type 4



Graph Data

Service Life: 100 years
T (pf = 10%): 8.873704
T (pf = 50%): 27.44186
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

Graph Data

Service Life: 100 years
T (pf = 10%): 26.46739
T (pf = 50%): 97.90625
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

e) Type 5 f) Type 6

Figure 5.3 - Graph results from the software for the six concrete mixtures.

The above example demonstrates how important a proper selection of cement type is in order to obtain a concrete with adequate resistance against chloride penetration (chloride diffusivity) and thus adequate safety against steel corrosion.

For the above example, the slag cement in combination with silica fume would fulfill the overall requirement to safety and durability without any additional protective measures. If it would not be possible to produce such a concrete quality, however, other strategies and protective measures for ensuring proper durability should be considered.

These results demonstrate the ability of blast-furnace slag cements to resist chloride penetration which is in general agreement with current experience (Bijen 1998) and provides a good basis for further decision making.

It is, therefore, interesting to evaluate the effect of different binders on the durability performance of the concrete. Despite the fact that the Type 1 concrete satisfies all European standards for concrete mix design and durability requirements, its performance is still the poorest of those simulated, not guaranteeing the necessary service life of the structure. Such analysis that can have a significant impact on the decision making process during the design phase of a concrete structure.

5.2.3 Effect of concrete cover on design

Based on the same data as for Concrete Mixture Type 2 in the previous example (Portland cement with 10 % silica fume) (Table 5.5), a new durability analysis in order to find out the effect of increased concrete cover above the minimum requirement of 50 mm was carried out (Figure 5.4).

Table 5.5 - Input parameters for analyzing the effect of concrete cover (Årskog 2004).

Parameter	Average	Standard deviation	Comments
D_0	4.74	0.51	Chloride diffusivity (x 10^{-12} m^2/sec)
α	0.39	0.07	Age factor of diffusion coefficient (-)
C_{CR}	0.48	0.15	Critical chloride content (% by weight of cement)
C_S	5.4	0.54	Surface chloride content (% by weight of cement)
x_C	50	5	Concrete cover (mm)
	60	5	
	70	5	
	90	5	
	120	5	

As can be seen from Figure 5.4, the concrete cover is also of great importance for the probability for corrosion. While a nominal concrete cover of 50 mm only would give a service period of approximately 10 years, an effective, nominal concrete cover of 120 mm would give a service period of more than 100 years.

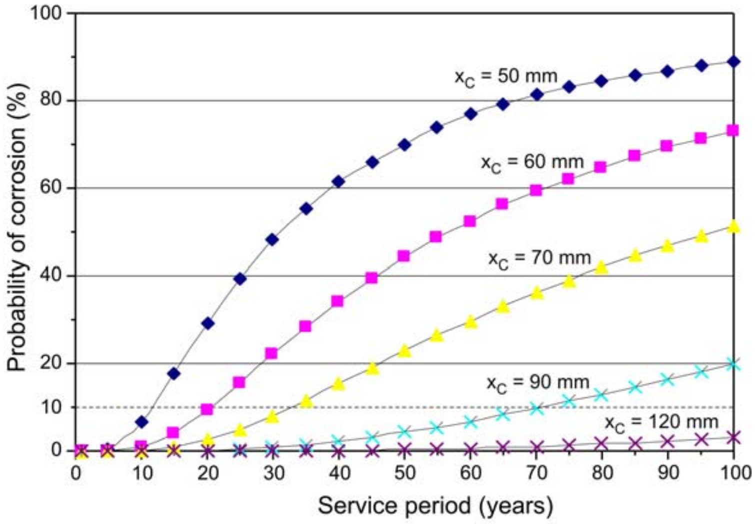
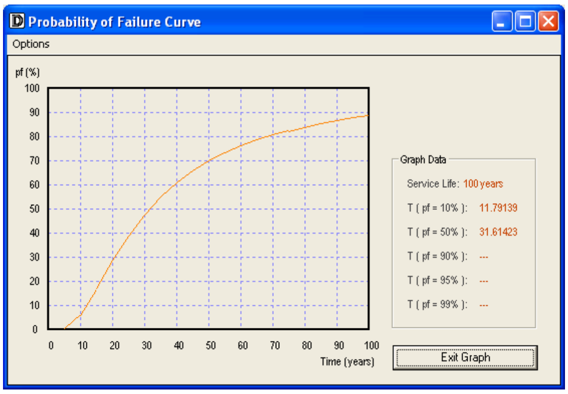


Figure 5.4 - Effect of concrete cover on the probability of corrosion in a concrete harbour structure (Årskog 2004).

Since it may be difficult to significantly increase the concrete cover in the lower part of the deck beams beyond 70 mm, only a partial replacement of the conventional reinforcement with stainless steel reinforcement could be used in order to obtain a substantial increase of the effective concrete cover beyond 70 mm.

The results as obtained from the software for the five depths of concrete cover are displayed in Figure 5.5.



Graph Data

Service Life: 100 years

T (pf = 10%): 11.79139

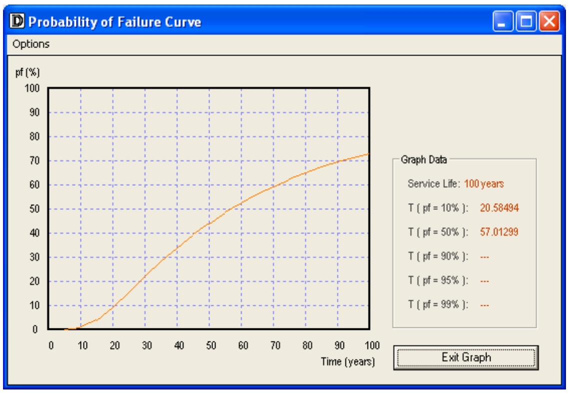
T (pf = 50%): 31.61423

T (pf = 90%): ---

T (pf = 95%): ---

T (pf = 99%): ---

a) 50 mm



Graph Data

Service Life: 100 years

T (pf = 10%): 20.58494

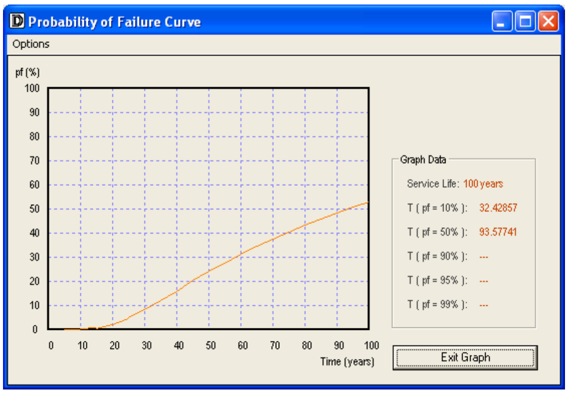
T (pf = 50%): 57.01299

T (pf = 90%): ---

T (pf = 95%): ---

T (pf = 99%): ---

b) 60 mm



Graph Data

Service Life: 100 years

T (pf = 10%): 32.42857

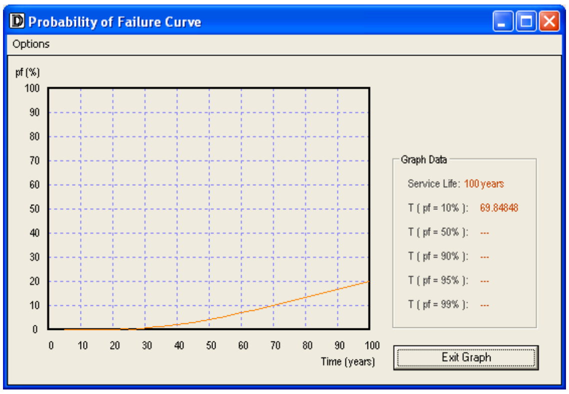
T (pf = 50%): 93.57741

T (pf = 90%): ---

T (pf = 95%): ---

T (pf = 99%): ---

c) 70 mm



Graph Data

Service Life: 100 years

T (pf = 10%): 69.84848

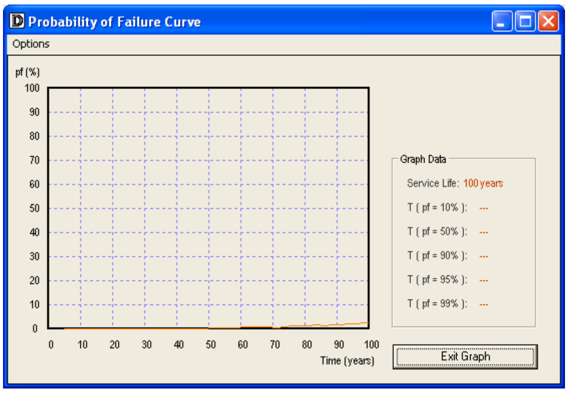
T (pf = 50%): ---

T (pf = 90%): ---

T (pf = 95%): ---

T (pf = 99%): ---

d) 90 mm



Graph Data

Service Life: 100 years

T (pf = 10%): ---

T (pf = 50%): ---

T (pf = 90%): ---

T (pf = 95%): ---

T (pf = 99%): ---

e) 120 mm

Figure 5.5 - Graph results from DURACON for the five concrete cover depths.

5.3 Durability analysis for condition assessment of existing concrete structures

Besides the durability design of new structures, the proposed model can also be used as an improved basis for condition assessment of existing concrete structures in marine environment. For condition assessment, the way in which the model parameters are obtained differs in relation to the durability design of new structures.

As the structure already exists, both the concrete quality and the environmental loading can be measured directly on the structure with appropriate testing procedures. The specified concrete cover can be measured using a covermeter. Both the diffusion coefficient and the surface chloride concentration can be determined by extracting cores from the structure and testing them in the laboratory. Another possibility is to collect concrete dust samples from the structure and determines the chloride profiles. Based on this information, the surface chloride concentration can be determined as well as the diffusion coefficient.

For the age dependency of the diffusion coefficient parameter and the critical chloride concentration, information is needed on the concrete mix design, namely the cement type and cement content. As discussed previously (Chapter 2.3), the age dependency of the diffusion coefficient parameter is dependent on a number of factors such as the water/cement ratio, the cement type, the pH of the pore solution, the capacity of the cement paste to bind the chloride, etc. To quantify this parameter, various suggested literature is available (Browne 1982, Breit 1997, Hansson 1990, Frederiksen 2002, Duracrete 1998).

The only remaining parameter is the temperature which is quantified in the same manner as for durability design.

5.4 Summary

In this chapter, a brief outline of both the durability design of new concrete structures as well as the condition assessment of existing concrete structures in marine environment was presented. In the following part of the thesis, the applicability of the proposed model for an improved condition assessment of existing concrete structures in marine environment are demonstrated and discussed.

CHAPTER 6
DURABILITY INVESTIGATION OF EXISTING CONCRETE STRUCTURES

6.1 General

To evaluate the proposed model as a basis for an improved condition assessment of existing concrete structures, realistic data was needed. For this reason, four reinforced concrete wharf structures were assessed to collect the necessary data. In this chapter, the durability investigations of the structures are presented, with the description of the test procedures adopted, and the results obtained.

6.2 Test methods and procedures followed

Several structures situated in a marine environment were investigated with the objective to collect data for an assessment of their durability performance and an evaluation of their service life, using the proposed model. In order to obtain the information necessary to assess the structures performance, several test were performed on the structures to gather sufficient relevant information.

Inspection procedures involved quality assessment of the built reinforced concrete structures as well as the measurement of the environmental response. The inspection of the built quality of the structures was performed through the evaluation of the concrete cover depth, the chloride diffusivity and by electrical potential mapping of the concrete cover. The environmental response was evaluated by measuring the chloride penetration (profiles), the carbonation depth, and the corrosion activity through visual damage assessment.

The four structures examined in this investigation had ages ranging from 7 years up to 14 years. An effort was made to collect as much information as possible, from the most significant elements of these structures. For reasons concerning the confidentiality of the owners of the structures, these shall only be referred to as *Structure 1* through to *Structure 4.*

6.2.1 Laboratory and *in situ* test methods

Non-destructive and semi-destructive techniques were used for the assessment of the various structures and their concrete properties, either *in situ* by direct or indirect measurement, or in the laboratory.

Several test methods were used. *In situ* tests consisted of concrete cover measurements, half-cell potential measurements, core extraction for carbonation depths and diffusion coefficients, and collection of samples for chloride profiling. Laboratory test methods consisted of chloride profile determination (chloride contents calculation and diffusion coefficient determination through curve fitting of profiles) and diffusion coefficient from migration testing.

6.2.1.1 Concrete cover measurements

The depth of reinforcement below the surface of concrete was measured using a covermeter. Covermeters are electro-magnetic devices consisting of a search head and a control box. This equipment has digital readout and an audible output which increases in loudness or pitch as the search head approaches the position of a reinforcing bar.

This method for locating reinforcing steel within a concrete member is truly non-destructive. Under reasonable conditions, a site accuracy of estimated cover of ± 5 mm within the working range of the instrument may be expected (BRE Report 216 1994).

The effect of bar size is important if this is less than 10 mm or greater than 32 mm. Calibrations are sensitive to steel type, bar diameter and deformity, aggregate and cement type, and these factors have to be taken into account. Most commercially available equipments are calibrated for medium-sized mild steel round bars in ordinary Portland cement concrete. The range of the equipment is limited according to type, but care and experience in interpretation of data are required in the following cases:

a) multiple bars, e.g. laps, transverse steel or closely spaced parallel bars;
b) light wire mesh, buried nails or other metals between the reinforcing bars and the surface;
c) metal tie wires;
d) aggregates with magnetic properties;
e) stability of calibration, which may be particularly important in relation to temperature changes within the magnetic field;
f) stray magnetic fields.

The principal applications are the location of reinforcement and estimation of cover, orientation and, in some cases, diameter of reinforcing bars. In this investigation, the technique was used to ensure correct location and cover of reinforcing bars of concrete members, and for determining the location of reinforcement as a preliminary work for other tests in which reinforcement should be avoided, e.g. extracting cores.

6.2.1.2 Chloride profiles

Dust sample collection

The chloride profiles were measured either on cores immediately after extraction by grinding off material in layers parallel to the exposed surface, or directly with dust samples collected from the structures.

Generally dust-samples were taken in intervals of 5-10 mm, up to a depth of 40-70 mm into the structure. The dust samples are analysed and the total chloride content in each sample is determined. The chloride analyses are made in the laboratory.

The equipment used consists of a hammer drill with depth-measurement gauge, equipment to measure the thickness of the concrete cover and localise the reinforcement-bars, and equipment to collect the dust-samples (a short brush or a glazing knife, a plastic bag and cup to collect the dust, a tire inflator, to clean the holes from dust and scalable plastic bags or cups to store the dust.) The diameter of the drill was 12 mm. Finally, repair material to repair the holes is also needed. All the equipment used for drilling, collecting and storing of dust-samples must be properly cleaned and free from contamination, especially thawing-salt and salt from sea water.

The drilling-places should be selected in such a way, that the reinforcement-bars are avoided and the dust-samples be the most representative for each depth. A cover-thickness-measuring gauge can be used to localise the reinforcement-bars. The total depth of the boreholes should be adjusted to the specific construction (concrete cover thickness, penetration depth and the required accuracy of measurements), the division in different depth-levels and the purpose of the investigation.

The outer layer of each structure, approximately 5 mm, shall be treated as the initial layer. The dust is dependent of the equipment used for the drilling, collected in filter bags, special dust collectors or cups made for collecting dust. The samples should be stored in scalable plastic bags or cups to prevent the samples from being contaminated. When samples are taken from different depth-layers the hole shall be cleaned between each sampling.

The number of holes should be adjusted to depth-layer interval, the amount of dust and the method for collecting the dust (See Figure 6.1). As a basic rule each dust sample should contain dust from at least two holes. The amount of concrete dust from each hole is between 25-40 g (with a thickness of each layer equal to 10 mm). Dusts from at least eight depths were collected.

Profile grinding was used when cores were extracted from the structure. It is a simple method where concrete dust is cut from different layers of a concrete sample. The samples used in the method are cores with a diameter of 100 mm. The concrete dust is used to determine the chloride content in different layers of the concrete structure.

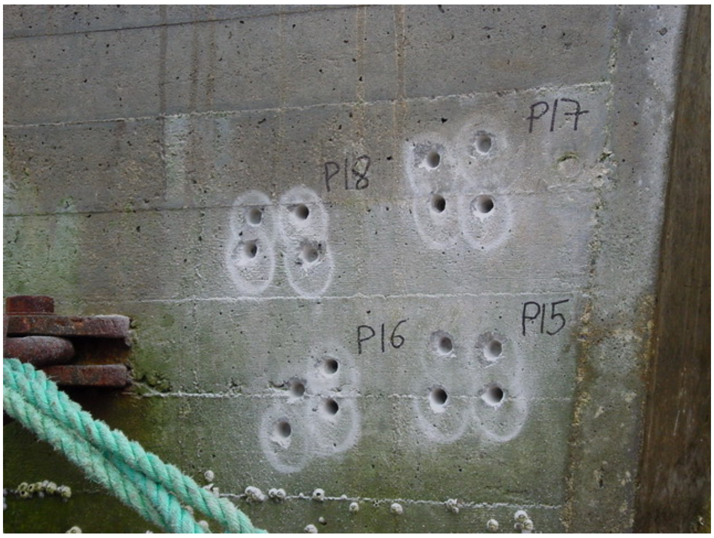


Figure 6.1 – Example of location of dust samples extraction from a concrete structure.

The equipment used consists mainly of a turning lathe, to which the sample is attached, and a cutter. The area of the cutter should be smaller than the area that is to be cut. A container, to collect the dust samples, is placed under the cutter. The dust from each layer is collected separately and put into a plastic bag.

Chloride content determination

In a spectrophotometric analysis, a sample solution in a glass or quartz "cell" is inserted into a spectrophotometer. Radiant energy (ultraviolet, visible, or infrared) of a very narrow (monochromatic) wavelength range is selected from a source by means of a diffraction grating, and the resulting beam is directed at the cell containing the sample. Some of the radiant power incident on the cell is absorbed by the chemical substance(s) in the sample, and the remainder is transmitted. The amount of energy absorbed is proportional to the concentration of the chemical substance(s) absorbing it. This proportionality is the quantitative basis of spectrophotometry, or spectrophotometric methods.

The spectrophotometric analysis of chloride was performed by use of the mercury (II) thiocyanate method (Sorensen 1994).

Spectrophotometry is applied to dust samples prepared either as dust directly taken from the structure or with profile grinding on cores. The concrete dust is dissolved in hot nitrous acid (80

°C) and then filtered. A certain amount of the filtered material is mixed in a chemical-solution which together with chlorides becomes coloured brown. Depending on the chloride content in the solution the intensity of the orange colour changes, as shown in Figure 6.2. The chloride content is determined with a spectrophotometer and compared with analysis made with a solution with known chloride contents. The acid-soluble chloride content of the samples is determined to three decimals using a spectrophotometric method.

Figure 6.2 – Colour changes according to chloride concentration.

The dust samples used in the spectrophotometry become representative for the structure when they average the dust from various extractions. For all the structures evaluated, the dust was collected from five different locations. This is to avoid hitting large aggregates, when dust samples are taken directly from the structure.

The test results are influenced by the temperature and the accuracy of the drilling or profile grinding.

6.2.1.3 Diffusion coefficients

Migration testing

The CTH method, as described in NT Build 492 1999 which is equivalent to the recent LNEC E-465 2004, was applied to specimens drilled from field structures. The chloride migration coefficient determined by the method is considered to be a measure of the resistance of the tested material to chloride penetration. This non-steady-state migration coefficient cannot be directly compared with chloride diffusion coefficients obtained from the other test methods, such as the non-steady-state immersion test or the steady-state migration test. The method requires cylindrical specimens with a diameter of 100 mm and a thickness of 50 mm obtained from drilled cores with a minimum length of 100 mm.

An external electrical potential is applied axially across the specimen and forces the chloride ions outside to migrate into the specimen. Figure 6.3 shows the set up of the test procedure used in the laboratory. After a certain test duration, the specimens are axially split and a silver nitrate solution is sprayed on to one of the freshly split sections. The chloride penetration depth can then be measured from the visible white silver chloride precipitation, after which the chloride migration coefficient can be calculated from this penetration depth.

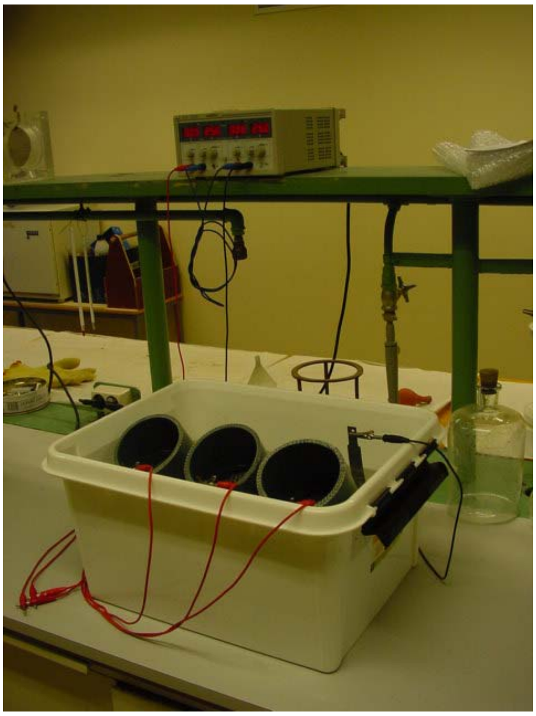


Figure 6.3 – Concrete specimens being tested in the laboratory according to the RCM test procedure.

Chloride profile fitting

To evaluate the chloride profiles the second Fick's law is applied. It is assumed that no significant deviation of Fick's second law occurs under constantly immersed conditions. When evaluating the achieved chloride profiles which were found in concretes exposed to real environmental conditions, it is necessary to consider the possibility, that deviations from the ideal shape can occur (due to carbonation, leaching, temperature variations, different chloride loadings, etc.) (Duracrete 2000a).

Different possibilities to determine the parameters D_{CPM} (apparent diffusion coefficient, determined by chloride profiling) and C_{SN} (surface chloride concentration, determined by chloride profiling) and which the values obtained are applied in practice are based on a best fit least squares analysis.

The determined output derived from the curve fitting carried out will be:

- D_{CPM}, an apparent diffusion coefficient, determined by chloride profiling in m/s^2 and
- C_{SN}, the notional surface chloride level in % by weight of concrete.

6.2.1.4 Half-cell potential measurements

Half-cell potential measurements are often carried out when reinforcement corrosion is suspected or evident. It is a measure of the electrical potential on the surface of the reinforcement and can be interpreted in terms of the likelihood of corrosion activity.

The equipment consists of a half-cell and a high-impedance voltmeter. The half-cell is a tube with a porous end, which contains a metal rod in a saturated solution of its own salt e.g. copper in copper sulfate or silver in silver chloride. Silver in silver chloride half-cells, while more expensive, provide a more stable system, although the copper in copper sulfate system is widely used.

The equipment is simple and enables a non-destructive survey to produce isopotential contour maps of the surface of a concrete member. Zones of varying degrees of corrosion risk may be identified from these maps. This method cannot indicate the actual corrosion rate.

Figure 6.4 - Half-cell potential measurements being made on a wharf structure.

It is important to recognize that the use and interpretation of the results obtained from the test require an experienced operator who is aware of its limitations such as the effect of protective or decorative coatings applied to the concrete (Concrete Society TR.54 2000).

This technique is most likely to be used for assessment or monitoring of the durability of reinforced concrete members where reinforcement corrosion is suspected. Reported uses (Concrete Society TR.54 2000) include the location of areas of high reinforcement corrosion risk in marine structures, bridge decks and abutments. When used in conjunction with other tests, it has been found helpful for investigating concrete contaminated by chlorides.

The procedure usually requires a small hole to be drilled to enable electrical contact to be made with the reinforcement in the member under examination and no surface preparation is required. One terminal of the voltmeter is connected to the reinforcement and the other terminal is connected to the half-cell as shown in Figure 6.4. It is usual to take readings on a grid pattern on the concrete surface. A closer grid may be used in areas of special interest. It may be necessary to spray water on the surface locally to obtain good electrolytic contact.

Electrochemical potential mapping.

The electrical potential relative to a half-cell indicates the risk of reinforcement corrosion. It is only a qualitative measurement and may be affected by several factors, such as reinforcement connectivity, concrete cover depth, chloride presence, relative humidity and temperature. Readings are normally taken over the concrete surface, initially on a large grid. The greatest corrosion risk is usually associated with areas in which the potential gradient is steep; where these occur the grid should be reduced. If there is uncertainty, further testing (e.g. drillings or cores to inspect the state of the steel) is required to indicate the corrosion risk. For ease of interpretation, results are usually plotted as contours of equal potential.

6.3 Assessment of concrete structures

6.3.1 Structure 1

Structure 1 is an eight year old Norwegian concrete harbour structure. The structure has a water front of 94 m, and consists of an open concrete deck on top of steel tubes filled with concrete. The durability requirements had been specified according to the Norwegian Concrete Code NS 3420 (2003) with a maximum w/c ratio of 0.45 and a minimum content of Portland cement (PC) of 300 kg/m^3 and minimum concrete cover of 50 mm. However, more cement had been used as shown in Table 6.1. A 5 % addition of condensed silica fume (CSF) by weight of cement had also been used. Hence, the durability requirements according to the new European Concrete Code EN 206-1 (2000) were also fulfilled.

Table 6.1 – Information on concrete mix design and project requirements

Material/Requirement	**Value**
Cement (PC) (kg/m^3)	380
Silica fume (kg/m^3)	19.2
w/c	0.45
Concrete cover (mm)	50

Condition assessment was primarily based on a large number of chloride penetration measurements and concrete cover measurements. A few Ø100 mm concrete cores were also drilled out for testing of chloride diffusivity and carbonation depths. Since the deck beams represented the most exposed and vulnerable parts of the structures, all measurements were concentrated on three representative elements of the structure, two separate beams and the concrete access bridge, as shown in Figure 6.5.

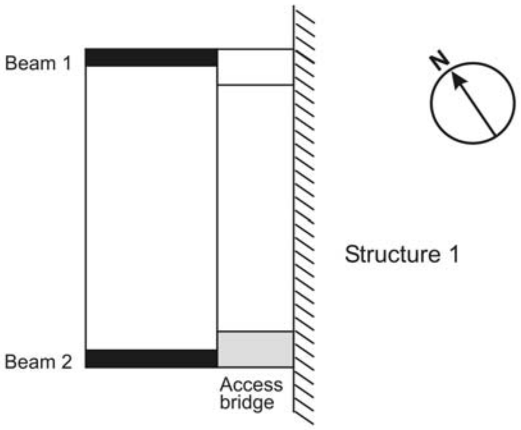


Figure 6.5 – Schematic representation of the wharf structure 1 and the elements assessed.

Figure 6.6 – A photograph of the concrete harbour structures investigated, suffering particularly severe weather.

Both the underside and the lateral side of the beams were evaluated. These were located successively further away from the most exposed side of the wharf. The location of these can be seen in figure 6.5.

The dominant wind directions were NW and SW. The top level of the deck is positioned + 4.00 m above the mean sea level. Figure 6.6 shows that the structure is subject to severe weather.

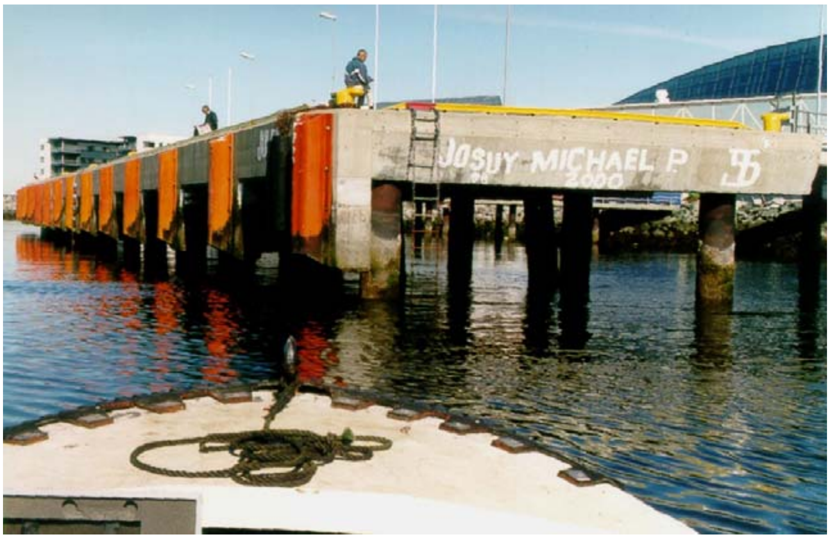


Figure 6.7 – Approaching Structure 1 by boat.

6.3.2 Condition assessment of Structure 1

6.3.2.1 Concrete cover measurements

An extensive measurement routine of the concrete cover was performed on both beams. The results are summarized in Table 6.2. No distinction was made between the concrete cover measurements of the two sides of the beam.

Table 6.2 – Concrete cover data for Structure 1.

Elements of S1	**Concrete Cover (mm)**		
	Average	Standard Deviation	Nº measurements
Beam 1 (side)	48.3	5.9	96
Beam 2 (side)	48.7	5.3	80

Figure 6.8 shows the probability density curve (PDF) of the concrete cover measurements, in comparison with a normative requirement of 45 mm and an assumed standard deviation of 5 mm (EN 206-1, Eurocode 2), for the environmental exposure class XS3. It can be seen from the figure that the majority of the measurements performed indicate a concrete cover greater than the minimum values.

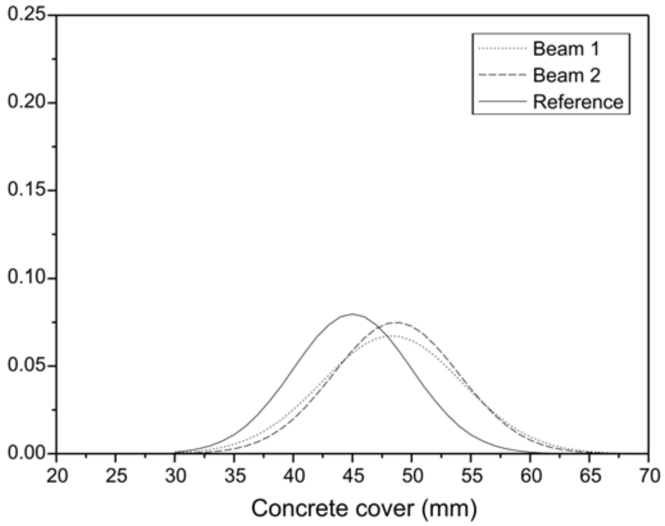


Figure 6.8 – PDF curve of concrete cover readings from both beams in comparison to the normative requirement.

6.3.2.2 Chloride profiles

Chloride profiles were measured on two distinct surfaces of each beam: the underside and the lateral side. On beam 1, the lateral side is facing northeast (exposed to the atmospheric agents) where as beam 2 is facing southwest (protected from the atmospheric agents). Due to the difficulty in access to the beams, a limited number of samples were collected. Figures 6.9 to 6.13 illustrate the measurements made and the average chloride profile curve obtained.

The profiles in figure 6.9 were obtained from concrete cores extracted from the top side of the access bridge. The high concentration illustrates the lack of convenient runoff and therefore accumulation of chlorides. Assuming the reinforcement is located at an average depth of 50 mm (no concrete cover measurements were made on the access bridge), by analyzing the profiles it can be seen that the chloride content is less than 0.1 % by weight of concrete. This is considered to be insufficient to initiated depassivation and therefore the upper reinforcement of the bridge elements might not have initiated corrosion yet.

When comparing the chloride profiles from the different surface of beam 1, it is apparent that the lateral side of the beam is directly exposed to the atmospheric elements; i.e. the wind and rain, and therefore subject to the wash off of the surface. In comparison, the underneath surface is not subject to the rain and therefore higher chloride concentrations is present.

For beam 2, the most protected beam of the wharf structure, the surface concentrations of both surfaces are identical (see location on figure 6.5) due to the protective nature of its location.

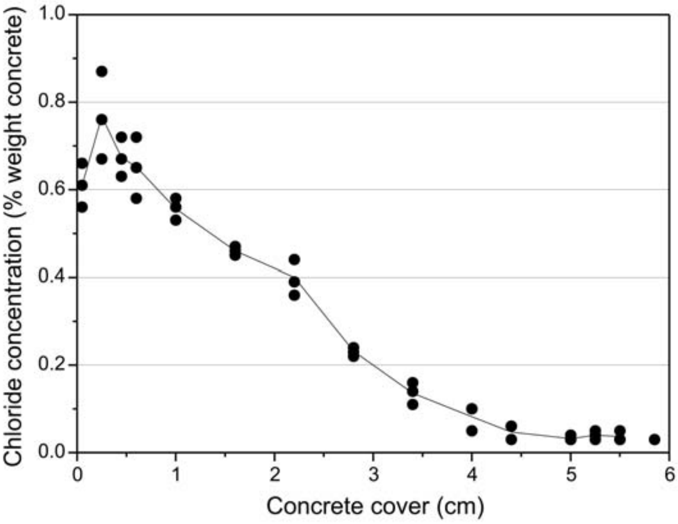


Figure 6.9 – Average chloride profile for the access concrete bridge.

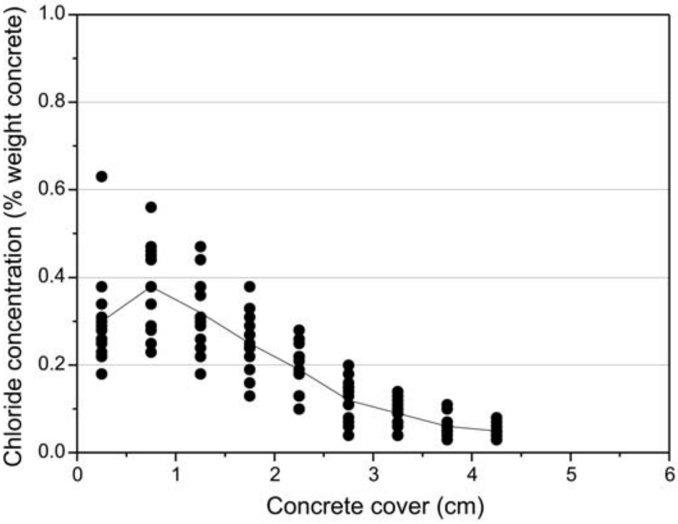


Figure 6.10 – Average chloride profile for beam 1 (side).

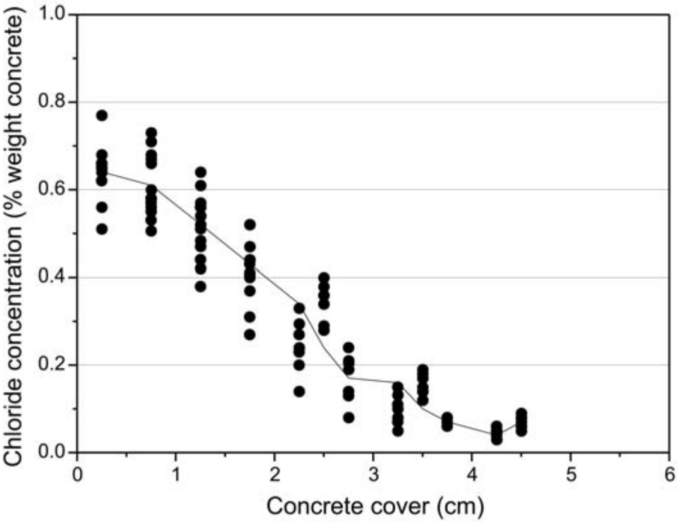


Figure 6.11 – Average chloride profile for beam 1 (under).

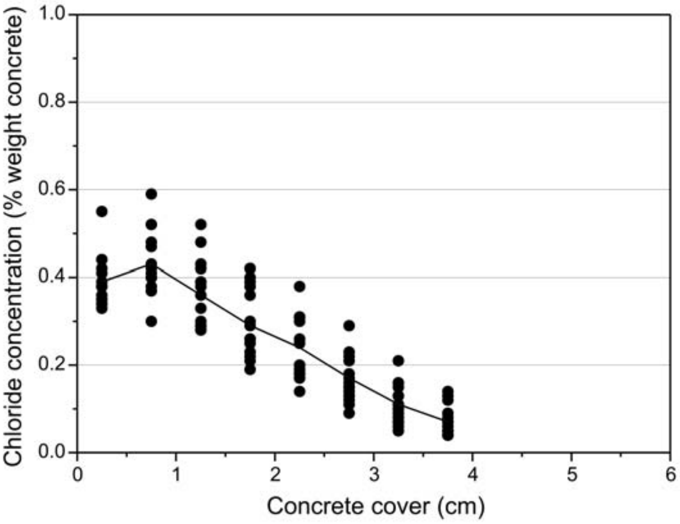


Figure 6.12 – Average chloride profile for beam 2 (side).

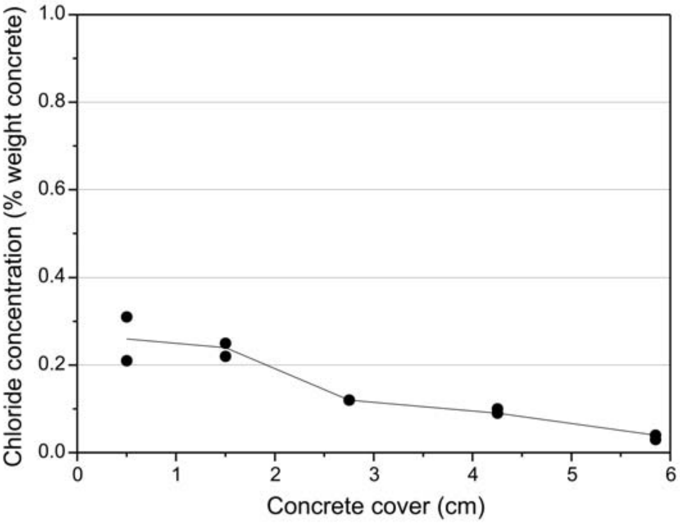


Figure 6.13 – Average chloride profile for beam 2 (under).

Assuming the reinforcement is located at an average depth of 50 mm, by analysing the profiles for either surfaces of both beams it can be seen that the chloride content is less than 0.1 % by weight of concrete. This is considered to be insufficient to initiated depassivation and therefore suggests that the elements might not experience yet corrosion on the reinforcement.

A summary of the results from the chloride profile analysis is shown in table 6.3. The statistical quantification of the data is presented for the elements analyzed. To obtain these results, the effect of the washout of chlorides from the surface was taken into consideration. This is done by replacing the surface measurement of chloride concentration measured with the value calculated from the curve fitting of the chloride profile.

Table 6.3 – Data calculated from profile curve fitting.

Elements	Chloride Diffusion (10^{-12} m^2/s)		Chloride Concentration (% weight of concrete)	
	Average	Standard Dev.	Average	Standard Dev.
Beam 1 (under)	1.140	0.162	0.828	0.079
Beam 1 (side)	0.948	0.166	0.555	0.158
Beam 2 (under)	1.028	0.170	0.645	0.161
Beam 2 (side	1.338	0.266	0.462	0.084
Access bridge	1.088	0.196	0.802	0.058

6.3.2.3 Concrete cores

In order to assist in the evaluation of the concrete quality, several concrete cores were extracted from the surface of the concrete access bridge. These cores were grinded for profile determination, for calculation of the diffusion coefficient and the surface concentration of chlorides.

Table 6.4 – Results of tests performed on cores.

Reference	D (10^{-12} m^2/s)	Cs (%weight concrete)
C1	1.301	0.741
C2	0.916	0.810
C3	1.045	0.855

From the values of the diffusion coefficients of the concrete access bridge, it appears that the same concrete has been used on the deck as well as on the beams. The chloride concentrations seem rather high compared to the values shown in Table 6.2, but due to the lack of adequate run-off for the water the chloride would normally accumulates on the surface.

6.3.3 Structure 2

Structure 2 is a fourteen year old Norwegian concrete harbour structure. The structure has a water front of 100 m, and consists of an open concrete deck on top of steel tubes filled with concrete. Unfortunately, very little information was available on the concrete composition of the wharf structure. The information obtained is presented in Table 6.5.

Condition assessment was primarily based on a large number of chloride penetration measurements; surface potential mapping and concrete cover measurements. A few Ø100 mm concrete cores were also drilled out for testing of chloride diffusivity and carbonation depths. Since the deck beams represented the most exposed and vulnerable parts of the structure, all measurements were concentrated on these three representative elements of the structure.

Table 6.5 - The data available on the concrete structure 2.

Cement (PC) (kg/m^3)	Grade C35 → C < 350
w/c	≈ 0.60
Concrete cover (mm)	unknown
Age (years)	14

The beams investigated were located successively further away from the most exposed side (north) of the wharf. The location of these can be seen in figure 6.14. Only the northeast lateral sides of the beams were assessed.

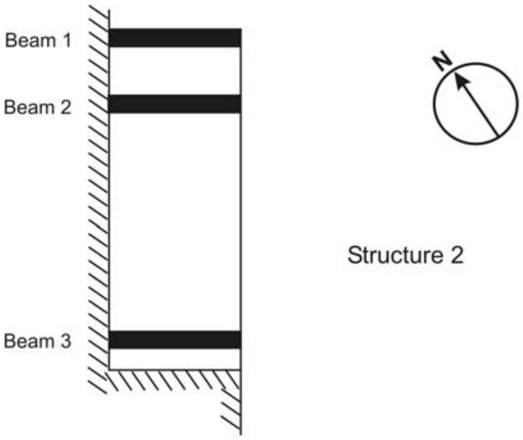


Figure 6.14 - Schematic representation of the wharf structure 2 and the elements assessed.

Since this harbour structure is an internal port structure, it is also subject to brackish water. The dominant wind direction is mainly from the north. The top level of the deck is approximately +4.00 m above the mean sea level. Figures 6.15 and 6.16 show different views of structure.

Figure 6.15 – View of S2 (Beam 1 is visible) while a large ship is moored.

Figure 6.16 – View under the harbour structure.

6.3.4 Condition assessment of Structure 2

6.3.4.1 Concrete cover measurements

An extensive measurement routine of the concrete cover was performed on all three beams. The results are summarized in the following table. The reinforcement was exposed locally at a few points, and measurements made showed that the main reinforcement was located at depths of approximately 40 mm.

Table 6.6 – Concrete cover data for Structure 2.

Elements of S1	**Concrete Cover (mm)**		
	Average	Standard Deviation	Nº measurements
Beam 1	43.4	1.7	22
Beam 2	42.0	2.8	27
Beam 3	38.7	4.0	26

Figure 6.17 shows the probability density curve (PDF) of the concrete cover measurements, in comparison with a normative requirement of 45 mm and an assumed standard deviation of 5 mm (EN 206-1, Eurocode 2), for the environmental exposure class XS3.

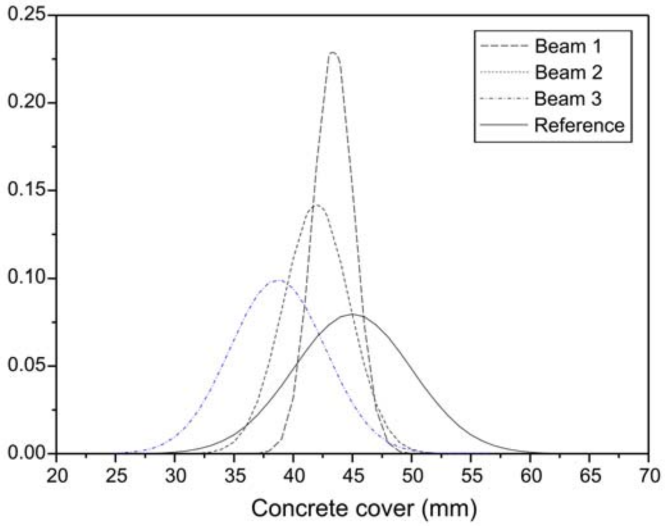


Figure 6.17 – PDF curve of concrete cover readings from the three beams in comparison to the normative requirement.

It can be seen from the figure that the majority of the measurements performed indicate a concrete cover lower than the minimum value required.

6.3.4.2 Chloride profiles

The chloride profile were measured on the north eastern lateral surfaces of the beams. Due to the difficulty in access to this area, a limited number of profiles were collected. Figures 6.18 to 6.20 illustrate the measurements made and the average chloride profile curves.

This structure is not subject to severe aggressive action from the sea, due to the protective nature of its location. For this reason, there is no significant variation between the chloride profiles of Beam 1, Beam 2 and Beam 3.

The chloride concentration at the depth of the reinforcement is approximately 0.2 % by weight of concrete. This value is double that generally considered necessary to initiate depassivation of the steel reinforcement, therefore it is highly possible that corrosion is taking place. This was confirmed by the extensive rust staining visible (figures 6.21 and 6.22) and when the reinforcement was exposed (Figure 6.23).

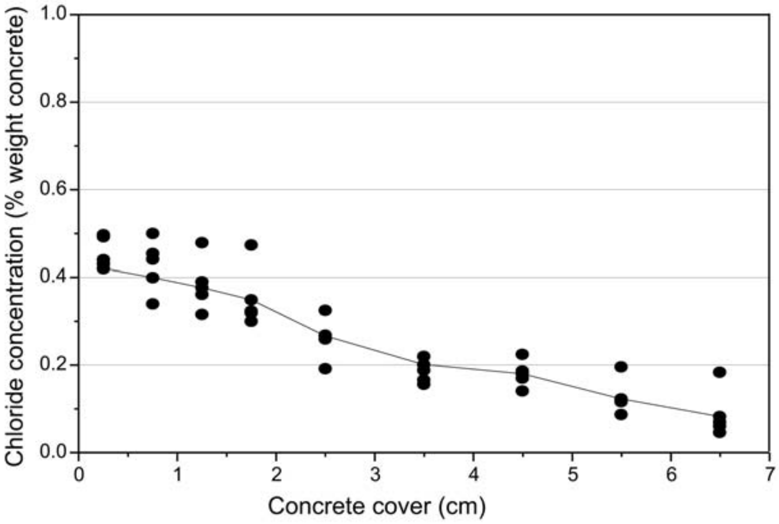


Figure 6.18 –Average chloride profile for the elements of the beam 1.

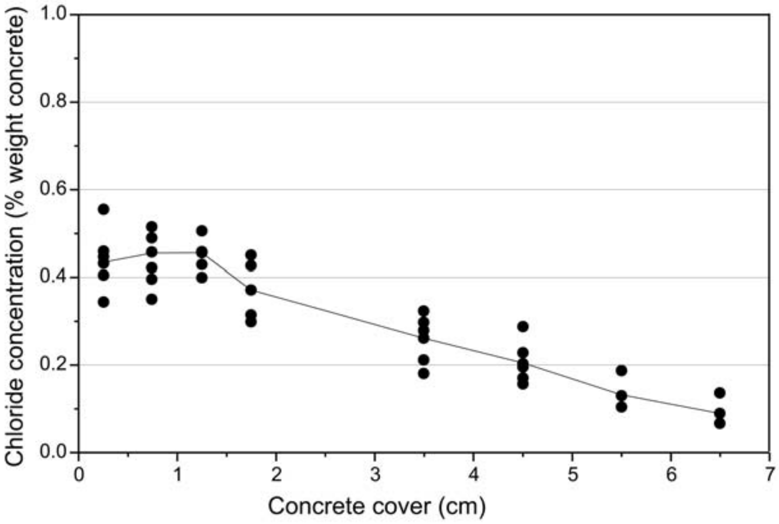


Figure 6.19 – Average chloride profile for the elements of the beam 2.

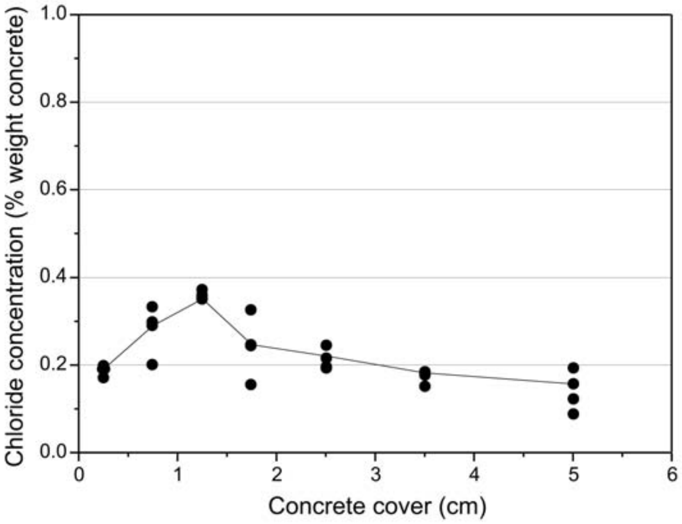


Figure 6.20 – Average chloride profile for the elements of the beam 3.

Figure 6.21 – Rust stains and corrosion cracking on beam 3.

Figure 6.22 – Rust stains on beam 2.

Figure 6.23 – Exposed reinforcement on beam 3.

A summary of the results from the chloride profile analysis is shown in table 6.7. The statistical quantification of the data is presented for the elements analysed. To obtain these results, the effect of the wash out of chlorides from the surface was taken into consideration. This is done by replacing the surface measurement of chloride concentration measured with the value calculated from the curve fitting of the chloride profile.

Table 6.7 – Data calculated from profile curve fitting

Elements	Chloride Diffusion (10^{-12} m^2/s)		Chloride Concentration (% /weight of concrete)	
	Average	St.Deviation	Average	St.Deviation
Beam 1	2.534	1.044	0.492	0.036
Beam 5	2.542	0.938	0.575	0.06
Beam 12	3.311	1.373	0.377	0.024

6.3.4.3 Concrete cores

In order to assist in the evaluation of the concrete quality, several concrete cores where extracted from the surface concrete deck. These cores were tested according to the NT Build 492 (1999), the results of which are presented in Table 6.8.

Table 6.8 – Results of tests performed on cores.

Reference	D (10^{-12} m^2/s)	Cs (% /weight of concrete)
C1	3.899	0.273
C2	4.442	0.247
C3	5.632	0.285
C4	6.948	0.267
C5	4.838	0.318

The results from the concrete cores seem to indicate that the deck concrete is not the same, due to the higher diffusion coefficients.

6.3.4.4 Electro-chemical potential measurements

Figures 6.24 to 6.26 illustrate the results of the electrical-potential measurements made on the beams. These were performed with the use of the BLOODHOUND® device. The length of the electrical potential map varies from 3.50 m to 4.50 m. The variation is a result of the difficulty in the access to the beams.

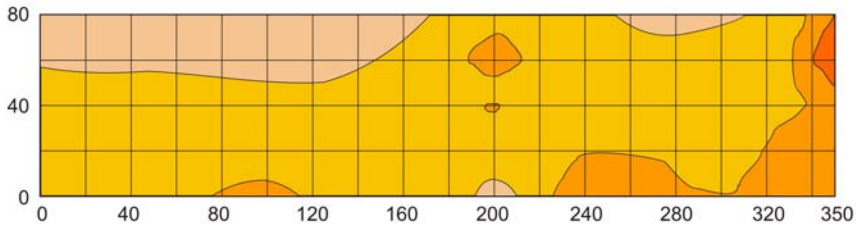


Figure 6.24 - Potential mapping of the lateral surface of beam 1.

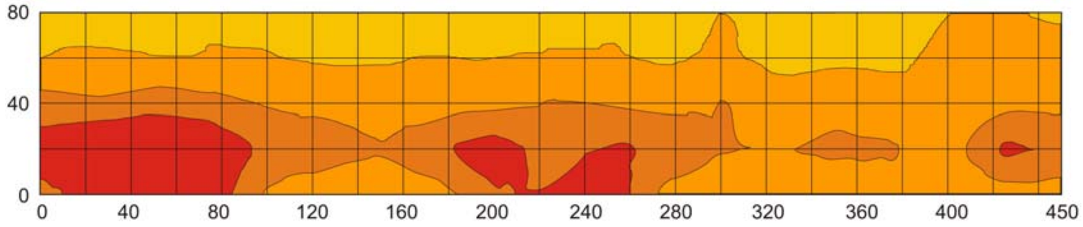


Figure 6.25 - Potential mapping of the lateral surface of beam 2.

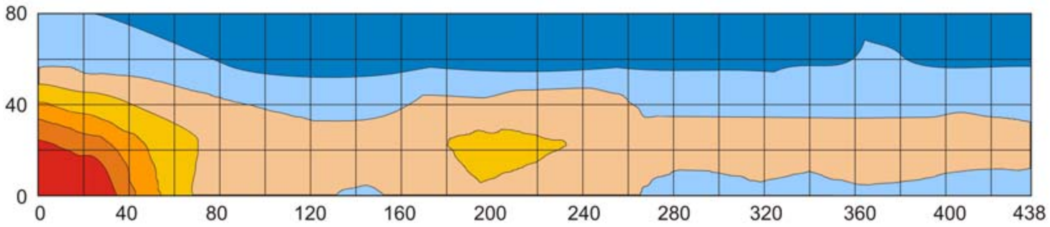


Figure 6.26 - Potential mapping of the lateral surface of beam 3

Figure 6.27 – Potential gradient values (mV).

All the three beams evaluated had extensive rust staining and longitudinal cracking of the beams. Beam 1 has a generalised low potential with the possibility of localised corrosion on the extremity.

Figure 6.25 shows beam 2 and the intense dark red area indicating that along the level of the main reinforcement very low potentials exist, therefore, most probably corrosion has started. These low potential readings coincide with corrosion rust stains on the surface of the beam.

Beam 3 appears to possess the least negative potential with only a high gradient in the lower left corner of the beam, possibly indicating initiation of pitting corrosion. These tendencies were confirmed by visual investigation of the reinforcement, as shown in figure 6.23.

6.3.5 Structure 3

Structure 3 is a seven year old Norwegian concrete harbour structure. The structure has a water front of 80 m and consists of an open concrete deck on top of steel tubes filled with concrete. The durability had been specified according to the Norwegian Concrete Code NS 3420 (2003) with a maximum w/c ratio of 0.45, a minimum content of Portland cement (PC) of 300 kg/m^3 and a minimum concrete cover of 50 mm. However, more cement had been used as shown in Table 6.9. Hence, the durability requirements according to the new European Concrete Code EN 206-1 (2000) were also fulfilled.

Table 6.9 - Some data on the concrete structures investigated.

Cement (PC) (kg/m^3)	400
w/c	0.45
Concrete cover (mm)	50
Age (years)	7

Condition assessment was primarily based on a large number of chloride penetration measurements, surface potential mapping and concrete cover measurements. A few 100 mm diameter concrete cores were also drilled out for testing of chloride diffusivity and carbonation depths. Since the deck beams represented the most exposed and vulnerable parts of the structures, all measurements were concentrated on a few representative elements of the structure.

On Structure 3, four separate elements were investigated. These were comprised of a central beam, two anchor slabs and a front skirt. The location of these can be seen in figure 6.28.

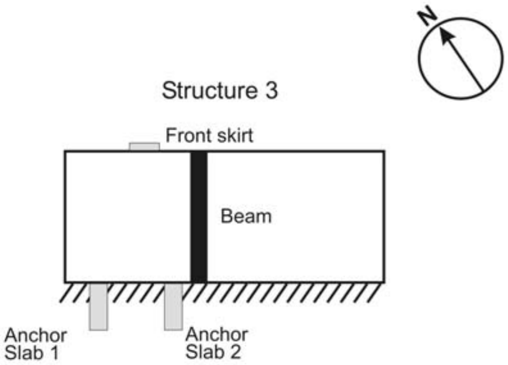


Figure 6.28 - Schematic representation of the wharf structure 3 and the elements assessed.

The dominant wind directions ranged from NE to NW. The top level of the deck was + 3.80 m above the mean water level. Figures 6.29 and 6.30 show the structure in question.

Figure 6.29 – Approach to Structure 3 from the fjord.

Figure 6.30 – View of the anchor slabs and beams under Structure 3.

6.3.6 Condition assessment of Structure 3

6.3.6.1 Concrete cover measurements

An extensive measurement routine of the concrete cover was performed on the central beam. The results are summarized in Table 6.10.

Due to erratic measurement values of the concrete cover of the anchor slabs, no conclusions were reached. These were prefabricated and indications on the project show a concrete cover of 60 mm. The reinforcement was exposed locally at a few points, and measurements made showed that the main reinforcement was located at depths of approximately 60 mm.

Table 6.10 – Concrete cover data for Structure 3.

Elements of S1	Concrete Cover (mm)		
	Average	Standard Deviation	Nº measurements
Beam	60.82	1.79	22
Anchor slabs	No values presented due to erratic reading		

Figure 6.31 shows the probability density curve (PDF) of the concrete cover measurements, in comparison with a normative requirement of 45 mm and an assumed standard deviation of 5 mm (EN 206-1, Eurocode 2), for the environmental exposure class XS3.

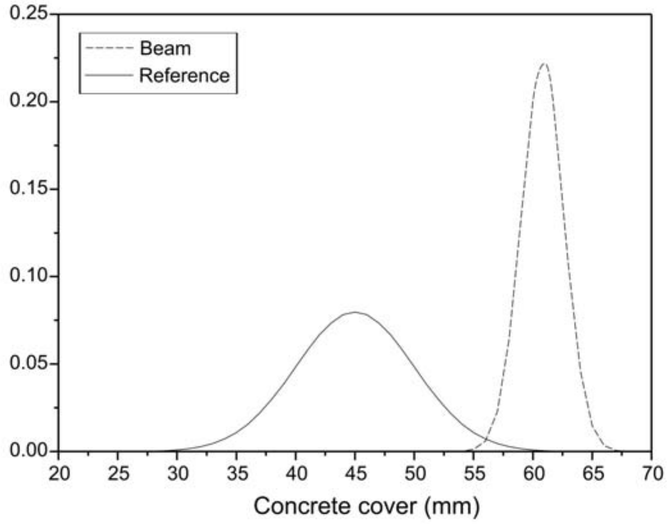


Figure 6.31 – PDF curve of concrete cover readings for the beam in comparison to the normative requirement.

It can be seen from the figure that the majority of the results obtained indicate a concrete cover larger than the minimum values.

6.3.6.2 Chloride profiles

Chloride profiles were determined on the north west surface of the central beam and on the underside of two anchor slabs. Due to the difficulty in access to this area, a limited number of profiles were collected. Figures 6.32 to 6.35 illustrate the measurements made and the average chloride profile curve.

When comparing the chloride profiles for the beam with the anchor slabs, the second, due to their closer proximity to the water level and the greater susceptibility to spray, have higher concentrations. Neither the beam nor the anchor slabs are in contact with rain water. On the other hand, the front skirt presents the highest concentration, since the samples where taken from the splash zone of the frontally exposed concrete elements.

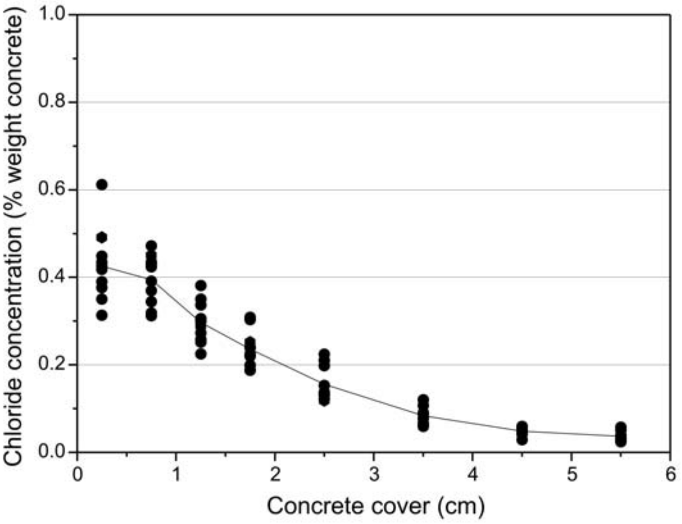


Figure 6.32 – Average chloride profile for the beam.

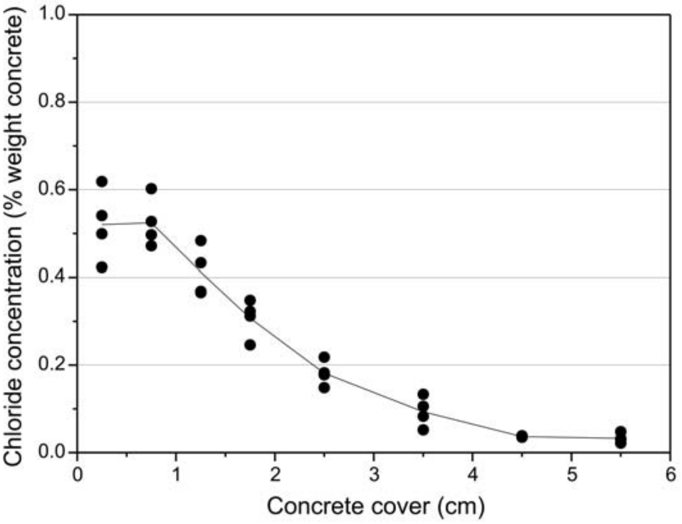


Figure 6.33 – Average chloride profile for anchor slab 1.

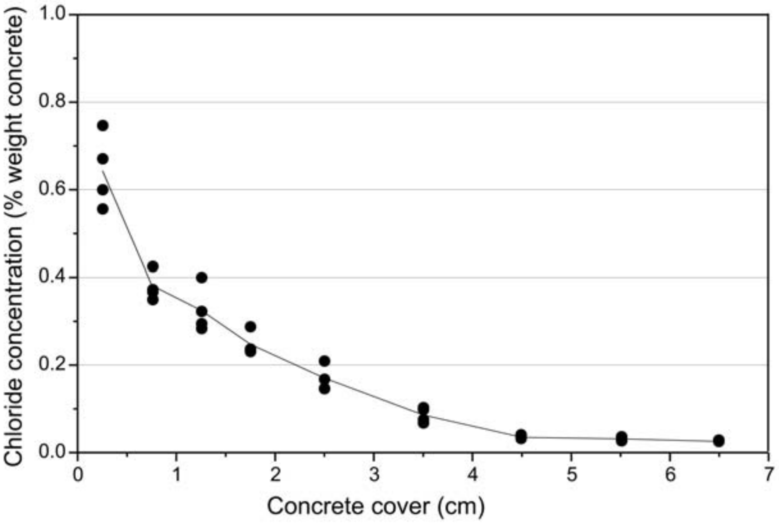


Figure 6.34 – Average chloride profile for anchor slab 2.

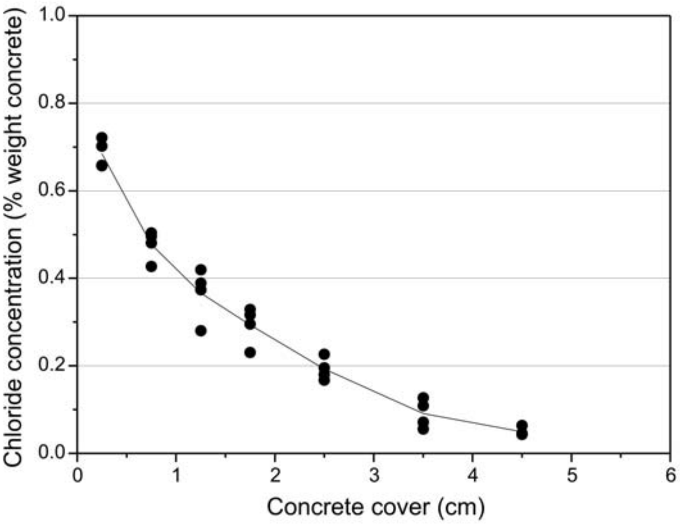


Figure 6.35 – Average chloride profile for the front skirt.

Assuming the reinforcement is located at an average depth of 60 mm, by analysing the profiles it can be seen that the chloride content is below 0.1 % by weight of concrete at the reinforcement depth. The concentration at that depth is considered to be insufficient to have initiated depassivation. Therefore, the reinforcement in these elements seems not to be in danger of corrosion.

A resume of the results from the chloride profile analysis is shown in table 6.10. The statistical quantification of the data is presented for the elements analysed. To obtain these results, the effect of the wash out of chlorides from the surface was taken into consideration.

Table 6.11 – Data calculated from profile curve fitting.

Elements	Chloride Diffusion (10^{-12} m^2/s)		Chloride Concentration (% weight of concrete)	
	Average	St.Deviation	Average	St.Deviation
Beam	1.589	0.432	0.518	0.070
Anchor slab 1	1.356	0.288	0.717	0.087
Anchor slab 2	1.327	0.527	0.581	0.103
Front skirt	1.262	0.271	0.710	0.025

6.3.6.3 Concrete cores

In order to assist in the evaluation of the concrete quality, several concrete cores were extracted from the surface concrete deck. These cores were tested according to the NT Build 492 (1999), the results of which are presented in Table 6.12.

Table 6.12 – Results of tests performed on cores

Reference	D (10^{-12} m^2/s)	ρ (Ω.m)	Observation
C7	10.22	63.01	---
C8	11.01	70.27	---
C9	8.94	70.31	---
C10	10.84	69.14	---
C11	---	69.55	Total penetration of specimen
C12	10.15	69.34	---

Although information made available by the owners confirms that the whole structure was executed with the same concrete type, the results indicate varying concrete qualities. The surface of the concrete deck is most probably covered by a superficial concrete layer of inferior quality.

6.3.6.4 Electro-chemical potential measurements

Figures 6.36 and 6.37 illustrate the results of the electro-chemical potential measurements made. These were performed using the BLOODHOUND® device.

While the condition of the beam does not seem to indicate corrosion initiation, the anchor slabs, namely the lower halves, show steep electric-chemical potential gradients . This is a strong indication of corrosion initiation.

More so, these tendency where confirmed by local visual examination of the reinforcement. The lower halves of the anchor slabs correspond to the most severely exposed extremity of the slab, subject to constant spray and wet/dry cycles of sea water.

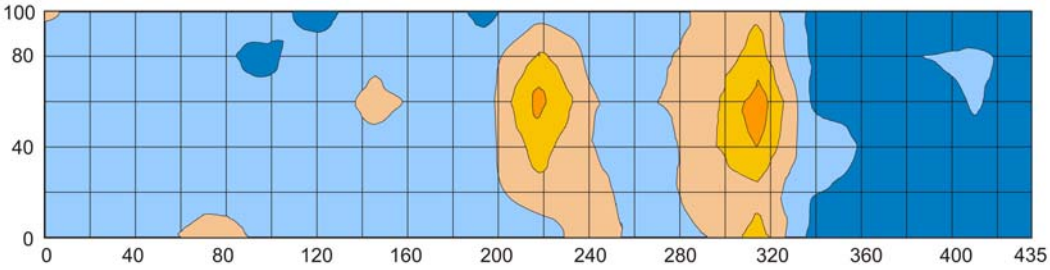


Figure 6.36 – Potential mapping of the surface of beam.

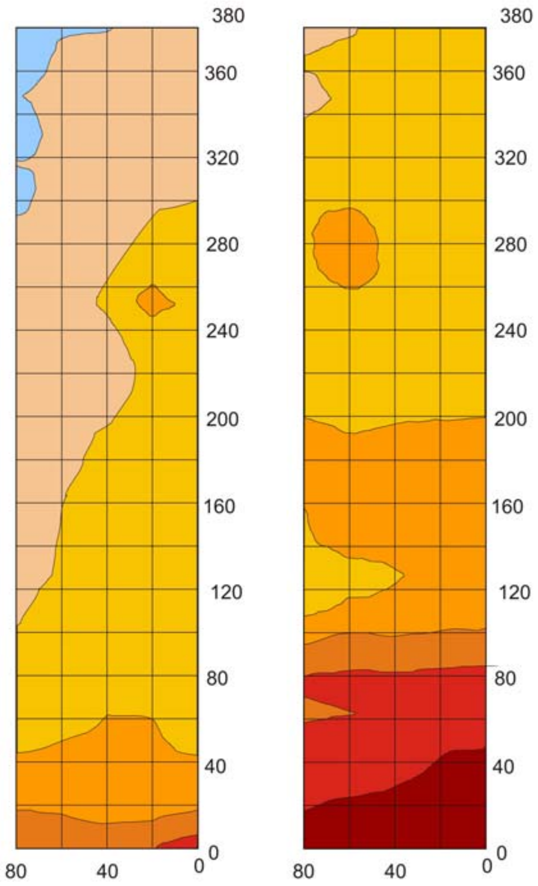


Figure 6.37 - Potential mapping of the surface of anchor slab 1 and 2, respectively.

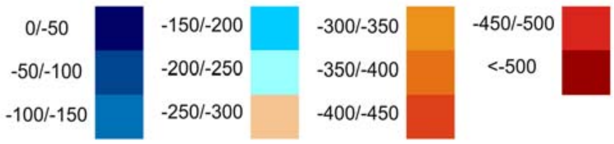


Figure 6.38 – Electrical potential gradient values (mV).

6.3.7 Structure 4

Structure 4 is an eight years old Norwegian concrete harbour structure. The structure has a water front of 131 m, and consisted of an open concrete deck on top of steel tubes filled with concrete.

The durability had been specified according to the Norwegian Concrete Code NS 3420 (2003) with a maximum w/b ratio of 0.45, a minimum Portland cement (PC) content of 300 kg/m^3 and a minimum concrete cover of 50 mm. However, more cement has been used as shown in Table 6.13. Hence, the durability requirements according to the new European Concrete Code EN 206-1 (2000) are also fulfilled.

Table 6.13 - Some data on the concrete structures investigated.

Cement (PC) (kg/m^3)	400
w/c	0.45
Concrete cover (mm)	50
Age (years)	8

Condition assessment was primarily based on a large number of chloride profile measurements, surface potential mapping and concrete cover measurements. Several 100 mm diameter concrete cores were also drilled out for testing of chloride diffusivity and carbonation depths. Since the deck beams represented the most exposed and vulnerable parts of the structures, all measurements were concentrated on a few representative elements of the structure.

On Structure 4, five separate elements were investigated. These were comprised of two beams (an exposed beam and a protected beam), two anchor slabs, and a front skirt. The locations of these are presented in figure 6.39.

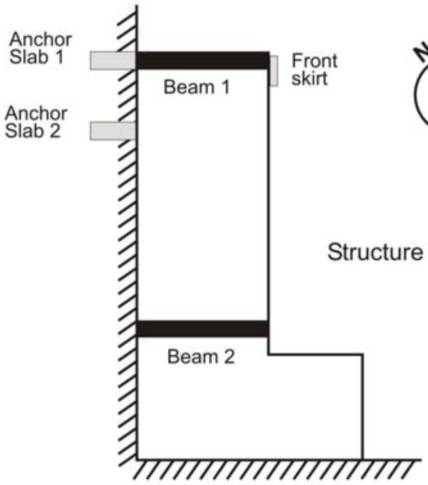


Figure 6.39 - Schematic representation of the wharf structure 4 and the elements assessed.

The dominant wind directions ranged from NE to NW. The top level of the deck was + 3.00 m above the mean water level. Figure 6.40 illustrates the construction in question.

Figure 6.40 – View of Structure 4 from the fjord.

6.3.8 Condition assessment of Structure 4

6.3.8.1 Concrete cover measurements

An extensive measurement routine of the concrete cover was performed on the both beams and both anchor slabs. The results are summarized in Table 6.14.

Due to erratic values of the concrete cover of the inner anchor slab measured, no conclusions could be drawn. The reinforcement was exposed locally both on the beams (up to the stirrups) and on the anchor slabs (main reinforcement). Measurements made showed that the reinforcement was located at depths of approximately 60 mm. The results are summarized in the following table.

Table 6.14 – Concrete cover data for Structure 4.

Elements of S1	**Concrete Cover (mm)**		
	Average	Standard Deviation	Nº measurements
Beam 1	53.33	6.09	21
Beam 2	57.39	4.14	28
Anchor Slab 1	49.17	8.11	18
Anchor Slab 2	No values presented due to erratic reading		

Figure 6.41 shows the probability density curve (PDF) of the concrete cover measurements, in comparison with a normative requirement of 45 mm and an assumed standard deviation of 5 mm (EN 206-1, Eurocode 2), for the environmental exposure class XS3.

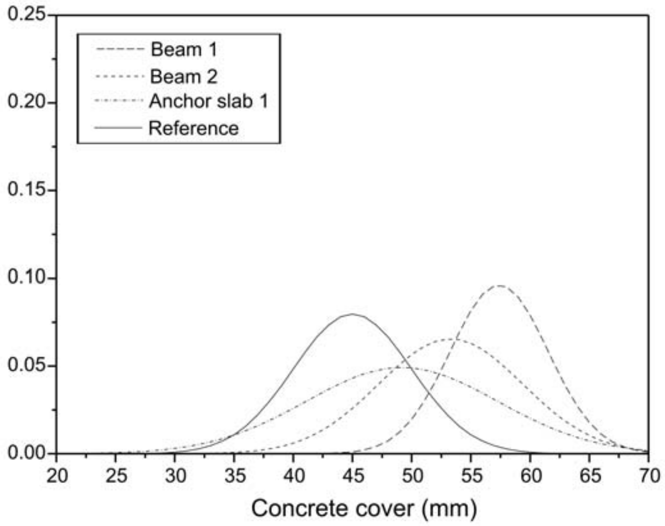


Figure 6.41– PDF curve of concrete cover readings from both beams and anchor slabs in comparison to the normative requirement.

It can be seen from Figure 6.41 that the majority of the measurements performed indicate a concrete cover larger than the minimum value required.

6.3.8.2 Chloride profiles

Chloride profiles were determined on the northeast surface of two beams, on the underside of the two anchor slabs and on the front skirt. Due to the difficulty in access to this area, a limited number of profiles were collected.

Figures 6.42 to 6.46 illustrate the measurements made and the average chloride profile curve.

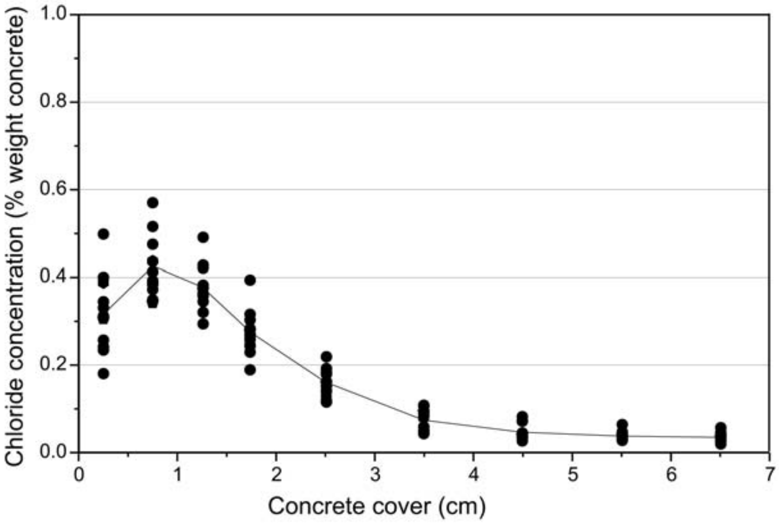


Figure 6.42 – Average chloride profile for beam 1 (side).

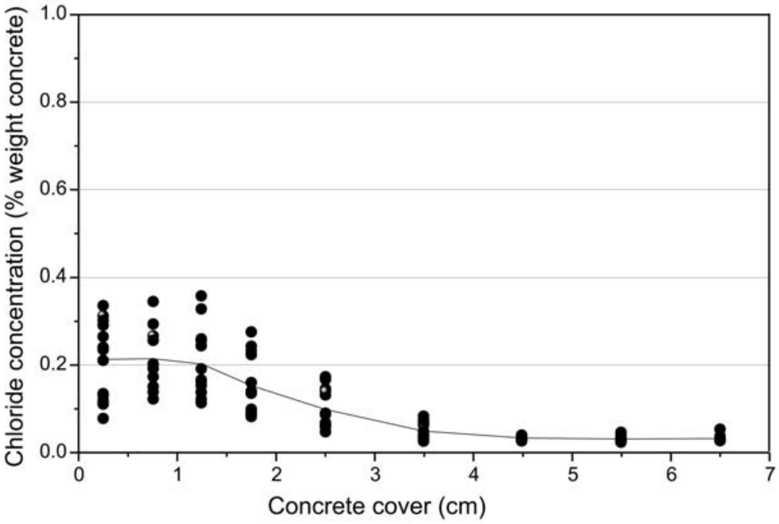


Figure 6.43 – Average chloride profile for beam 2 (side).

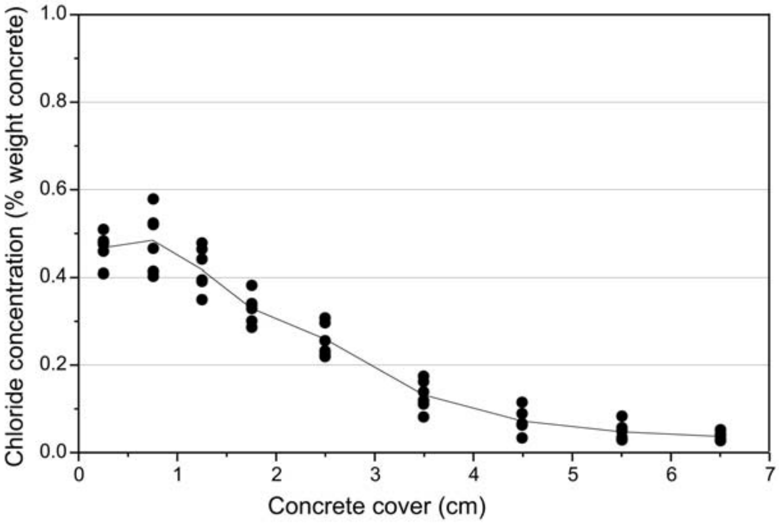


Figure 6.44 –Average chloride profile for anchor slab 1.

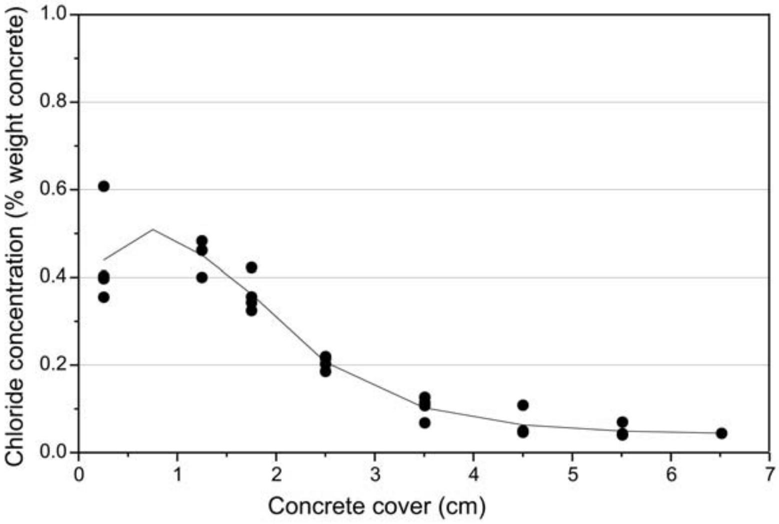


Figure 6.45 – Average chloride profile for anchor slab 2.

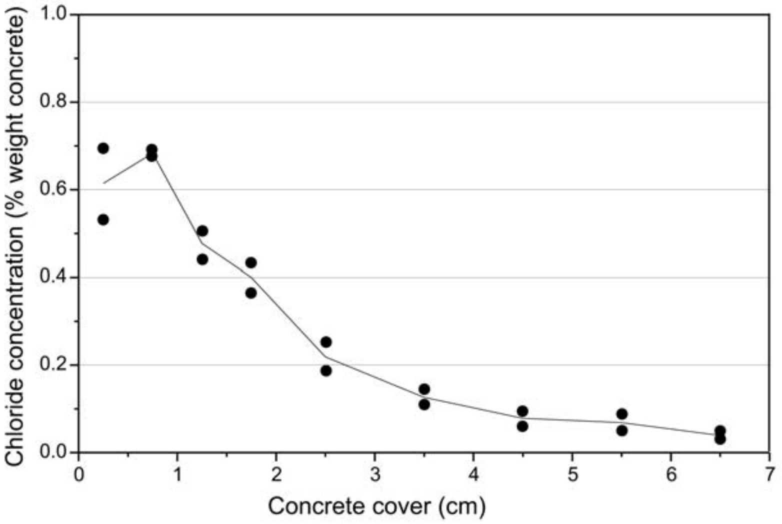


Figure 6.46 – Average chloride profile for the front skirt.

Comparing directly the chloride profiles from both beams, it is apparent that the outer beam is more exposed than the inner. However, the difference is not too high due to the slightly protective nature of the geographical location of the wharf structure, and, due to the relative height of the beam in relation to the average sea level. However, for the anchor slabs, due to their proximity to the tidal variations and constant splashing of the waves, the concentration are the highest found on the structure.

Assuming the reinforcement is located at an average depth of 60 mm, by analysing the profiles it can be seen that the chloride content is below 0.1 % by weight of concrete at the reinforcement depth. The concentration at this depth is considered to be insufficient to initiate depassivation. Therefore, the reinforcement in these elements may not be in danger of corrosion.

A resume of the results from the chloride profile analysis is shown in table 6.15. The statistical quantification of the data is presented for the elements analysed. To obtain these results, the effect of the wash out of chlorides from the surface was taken into consideration.

Table 6.15 – Data calculated from profile curve fitting

Elements	Chloride Diffusion (10^{-12} m^2/s)		Chloride Concentration (% weight of concrete)	
	Average	St.Deviation	Average	St.Deviation
Beam 1	1.23	0.28	0.620	0.123
Beam 2	1.53	0.83	0.303	0.112
Anchor slab 1	1.81	0.22	0.608	0.073
Anchor slab 2	1.66	0.55	0.703	0.054
Front skirt	1.23	0.20	0.879	0.037

6.3.8.3 Concrete cores

In order to assist in the evaluation of the concrete quality, several concrete cores were extracted from the surface concrete deck. These cores were tested according to the NT Build 492 (1999), the results of which are presented Table 6.16.

Table 6.16 – Results of tests performed on cores

Reference	D ($m^2/s10^{-12}$)	ρ (Ω.m)	Observation
C1	---	---	Cracked core
C2	---	---	Cracked core
C3	---	42.11	Total penetration
C4	22.85	39.76	---
C5	13.53	41.26	---
C6	13.40	45.43	---

Although indications made available by the owners of the structures confirm that the whole structure was executed with the same concrete type, results from Table 6.16 are too poor to be considered of the same concrete. The surface of the concrete deck is most probably covered with a superficial concrete layer of inferior quality.

6.3.8.4 Electro-chemical potential measurements

Figures 6.47 to 6.49 illustrate the results of the Electro Potential measurements made. These were performed with the use of the BLOODHOUND® device.

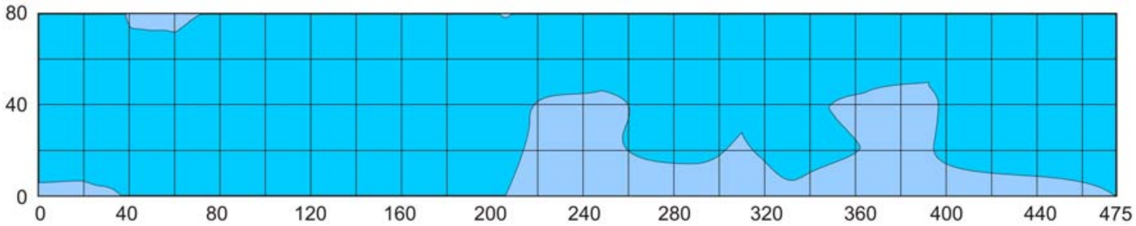


Figure 6.47 – Potential mapping of the surface of beam 1.

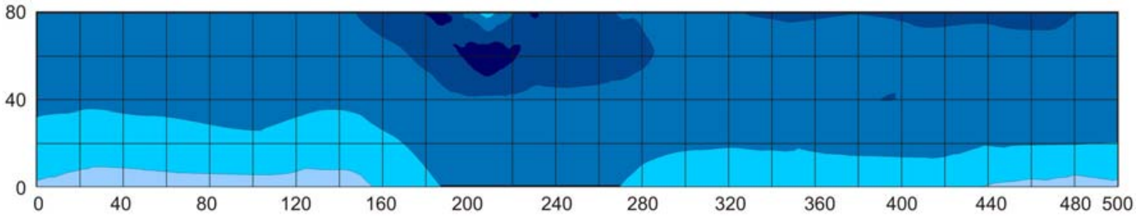


Figure 6.48 – Potential mapping of the surface of beam 2.

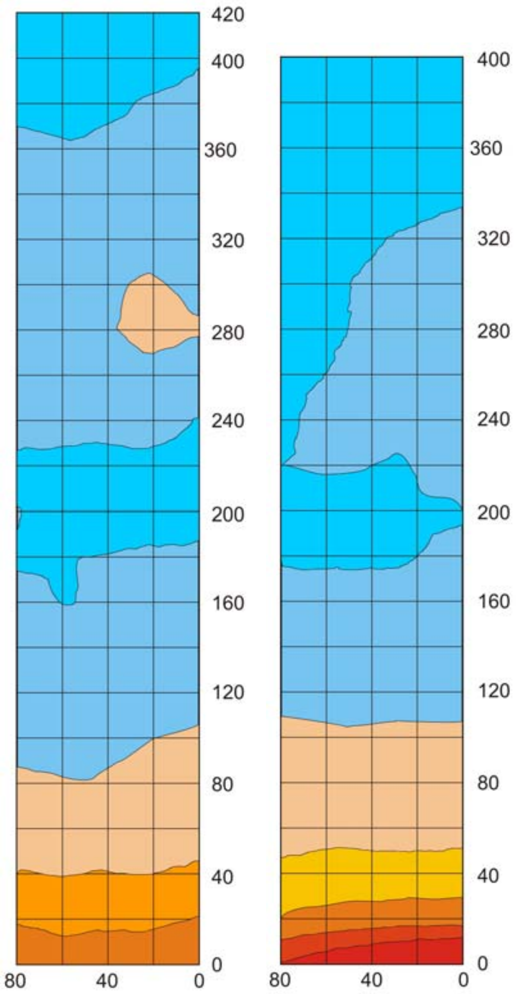


Figure 6.49 – Potential mapping of the surface of anchor slab 1 and 2, respectively.

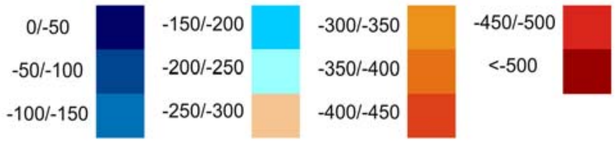


Figure 6.50 – Electrical potential gradient values (mV).

While the condition of the beam does not indicate corrosion initiation, the anchor slabs, namely the lower halves, show steep gradients in electrico-chemical potential. This is a strong indication of corrosion initiation.

These tendencies were confirmed by visual inspection of the exposed reinforcement at some locations. The lower halves of the anchor slabs correspond to the most severely exposed extremity of the slab, subject to constants spray and dry/wet cycles of sea water.

6.4 Summary

The inspection of the built quality of four structures was performed through the evaluation of the concrete cover depth, the chloride diffusivity and by electrical potential mapping of the concrete cover. The environmental response was evaluated by measuring the chloride penetration (profiles), and the corrosion activity through visual damage assessment.

The four structures examined in this investigation had ages ranging from 7 to 14 years. An effort was made to collect as much information as possible, from the most significant elements of these structures.

For most of the structures, owners had little or no information on the concrete quality of their structures. Furthermore, no documentation existed on the concrete mix design used. However, though contacts with the construction companies that built the structure and analysis of the structural designs, some information concerning the concrete properties was obtained.

Due to the specific nature of concrete wharf structures, the access to the underside of such structures is difficult and associated with tidal variations and wet/slippery rock fills, only a limited amount of tests were performed.

A general analysis of the results shows that the results obtained depend on the location of the element in the structure. This is important since the whole structure does suffer equally the effect of the environmental loading, i.e. the micro environment changes with the location of each element.

In terms of built quality, the concrete cover readings and the diffusion coefficients for the different elements of the structures show that the quality is generally uniform, whether above or below the prescribed requirements of the applicable standards.

Structure 2 is almost double the age of the other structures assessed. The longer period of exposure to the environment in conjunction with the poorer concrete quality make it the structure with the worse concrete quality of the structures assessed.

In Structures 2, 3 and 4, due to the presence of chlorides in the concrete, the overall electric-chemical potential readings normally suffer a shift in value of approximately -150 mV (Bertolini 2004). However, for localized (pitting) corrosion, it is the presence of the gradients that is of interest. The gradients that present a drop of -150 mV/m are of special interest (Bertolini 2004) and indicate that there is a strong possibility of localised corrosion.

CHAPTER 7
APLICATION OF MODEL TO THE CONCRETE STRUCTURES INVESTIGATED

7.1 General

Once the data from the assessed structures has been collected, the software (i.e. the proposed model) is used to assess the durability peformance of the four reinforced concrete wharf structures. A description of the data obtained from the assessment of the four structures is presented in Chapter 6.

To evaluate the durability performance of the structures, some basic information is needed regarding the service life of the concrete structures in question. Which were the limit state chosen (serviceability or ultimate) and, what events were chosen to represent the limit state violation?

7.2 Defining model parameters

The service life according to the various owners of the structures assessed was defined as being 50 years. However, no mention was made as to what "service life" represents: a serviceability limit state (initiation of corrosion, cracking, spalling, etc) or an ultimate limit state (collapse). Due to the lack of definition, and taking into account the characteristics of the model proposed, the serviceability limit state of depassivation of the reinforcement was chosen.

The various stochastic parameters presented are obtained from the data collected from the assessment of the structures. In relation to these parameters, x_C is measured directly on the structure, while D_C and C_S are obtained from the tests performed in the laboratory on samples taken from the structure. The age of the structures t, and the time of testing t_0 are also defined.

The analysis was performed at two separate temperatures: 21 ºC and 10 ºC. The choice of these temperatures is related to the fact that many design/assessment are made disregarding the effect the temperature has on the parameters measured. If the real average temperature is ignored, by default the model/software considers it to be the reference temperature (21 ºC). A more realistic temperature for the Norwegian climate, taking into account the influence that temperatures below zero have on chloride penetration (bringing it to a halt), is considered to be 10 ºC. In the model, the realistic effect of temperature is considered, hence, the 10 ºC is adopted in addition to the reference temperature.

In Chapter 6, the durability assessment has quantified the environmental loading (C_S), the concrete cover (x_C) and the diffusion coefficient of the concrete (D_C). Only the parameters C_{CR} and α remain to be defined for each structure.

According to NS 3490 (1999), which is the Norwegian Standard for requirements to reliability in design of structures, the probability of failure for a serviceability limit state should not exceed 10 %, and according to EN1990 (2002), the probability of failure for a serviceability limit state should not be more than 7 %, for a service life of 50 years. The relationship between the reliability index β and the probability of failure p_F is given in table 7.1.

Table 7.1 – Relationship between β anf the p_F (EN1990 2002).

p_F	10^{-1}	$7x10^{-2}$	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.28	1.5	2.32	3.09	3.72	4.27	4.75	5.20

For an irreversible serviceability limit state, the EN1990 (2002) establishes a minimum values for the reliability index at 1.5, for structures of Reliability Class 2 (associated with a medium consequence for loss of human life, and considerable consequence for economic, social or environmental factors) and a service life of 50 years.

For this analysis, the value of 10% probability of failure was chosen.

7.3 Probabilistic analysis of the durability performance of Structure 1

Table 7.2 presents a summary of the statistical parameters used for the probabilistic analysis of the various elements of Structure 1. Besides the deterministic parameters (t_0 and t), only normal distributions were used to characterise the data stochastically. The symbol *N(…)* indicate that the data has a normal distribution. The values between the bracket are the mean value and the standard deviation of the normal distribution. *D(…)* symbolises that the data is deterministic, hence only a single value is given.

The critical chloride concentration (C_{CR}) chosen took into account the total binder content, the w/c ratio and the fact that silica fume was used in the concrete mix. The value chosen corresponds approximately to 0.45 - 0.50 % by weight of binder (See Table 3.1, Chapter 3).

The time dependency of the diffusion coefficient, defined by the parameter α, was also based on the binder type (OPC + SF) and the environmental exposure conditions.

Figures 7.1 to 7.8 present the results of the probabilistic analysis performed on the elements of Structure 1 using the software described in chapter 4. Both the probability of failure versus time and the reliability versus time are presented, for both temperatures considered. The input data files and the result files for these simulations are presented in the Appendix 4.

Table 7.2 – Stochastic parameters for structure 1.

Elements/ Parameter	Beam 1 (under)	Beam 1 (side)	Beam 2 (under)	Beam 2 (side)
x_C (mm)	N(48.3, 5.9)	N(48.3, 5.9)	N(48.7, 5.3)	N(48.7, 5.3)
D_C (10^{-12} m^2/s)	N(1,140, 0.162)	N(0.948, 0.166)	N(1.028, 0.170)	N(1.338, 0.266)
C_{CR} (%/wt. conc.)*	N(0.08;0.01)	N(0.08;0.01)	N(0.08;0.01)	N(0.08;0.01)
C_S (%/wt conc.)	N(0.828, 0.079)	N(0.555, 0.158)	N(0.645, 0.161)	N(0.462, 0.084)
α (-)	N(0.39;0.07)	N(0.39;0.07)	N(0.39;0.07)	N(0.39;0.07)
t_0 (days)	D(2920.0)	D(2920.0)	D(2920.0)	D(2920.0)
t (years)	D(50.0)	D(50.0)	D(50.0)	D(50.0)

* 0.08 for a 380kg cement ≈ 0.45 - 0.50 % weight of cement.

The effect of temperature on the ingress of chlorides is apparent. For all the elements analysed, the time to the serviceability limit of 10% probability of failure to be reached increased more than double, when the simulation was performed at 10ºC.

Figures 7.1 to 7.4 represent results of evaluation of different surfaces tested of the same concrete element. The underside of beam 1 is more protected from direct rain fall, therefore, the concentration of chlorides on the surface is higher. Since the concrete properties are identical, it is the chloride concentration that affects the performance significantly. Due to the positioning of the surfaces of the element, at both 10ºC and 21ºC, double the time is necessary for the lateral surface to exceed the probability of failure limit of 10 % when compared to the underside.

Since the location of beam 2 is the most protected from the environmental exposure conditions of all the beams in the structure, both surfaces analysed present similar performances (see Figures 7.5 to 7.8).

Comparing the results of the analysis performed to beam 1 and beam 2, the favourable location of beam 2 results in better long term performance. At 10ºC, beam 1 would exceed the serviceability limit state of 10 % after 19 years, where as beam two would need 27 years. If we consider the reference temperature, a decrease in the time to depassivation occurs, with beam 1 exceeding the limit in 8 years and beam 2 in 12 years.

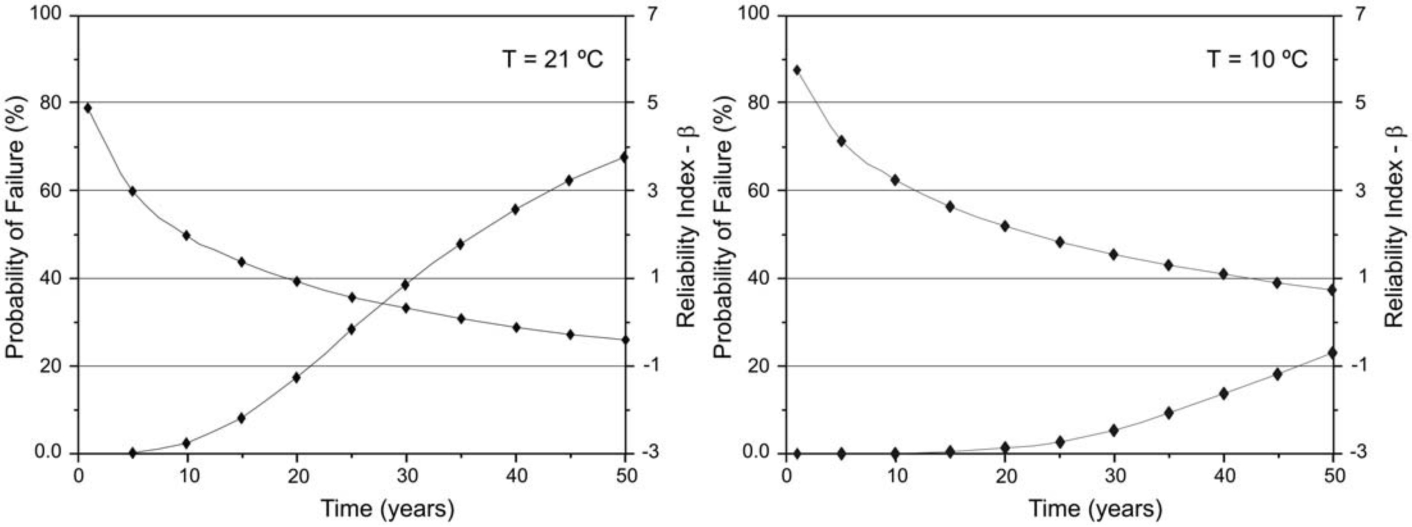


Figure 7.1 and 7.2 – p_F and β versus time for beam 1 (side).

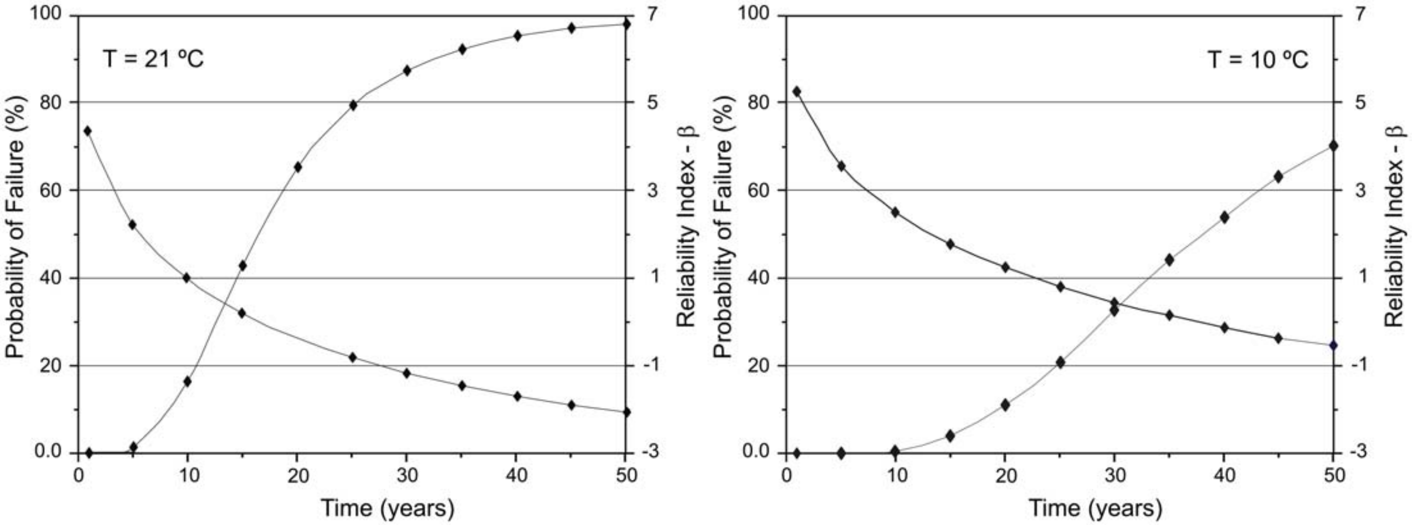


Figure 7.3 and 7.4 – p_F and β versus time for beam 1 (under).

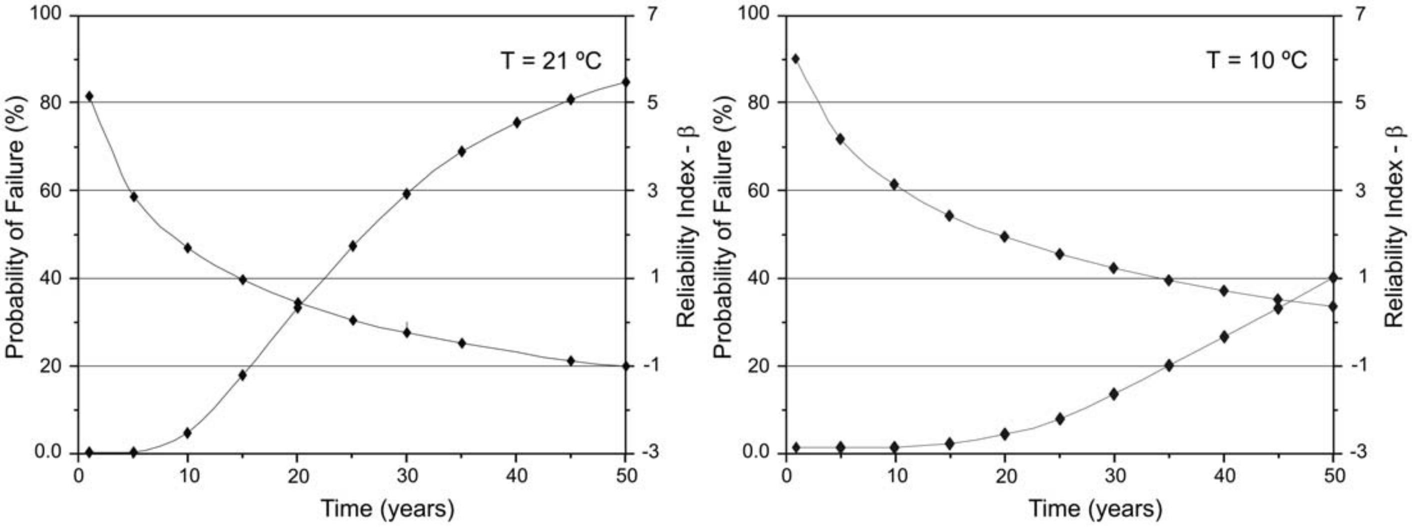


Figure 7.5 and 7.6 – p_F and β versus time for beam 2 (side).

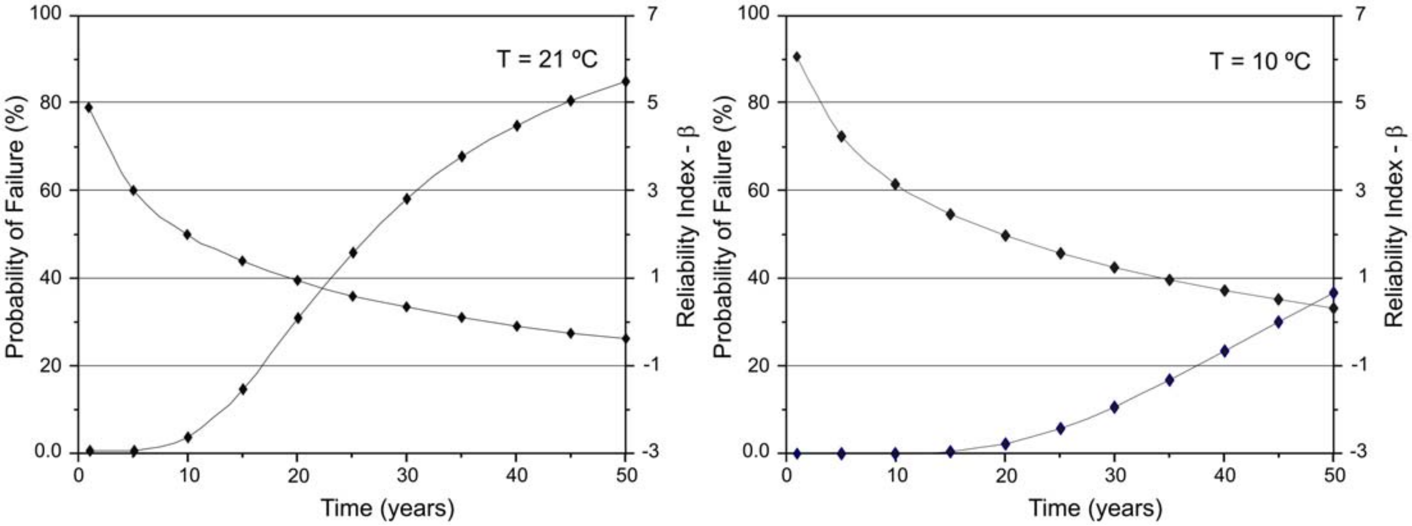


Figure 7.7 and 7.8 – p_F and β versus time for beam 2 (under).

7.4 Probabilistic analysis of the durability performance of Structure 2

In Table 7.3, the statistical parameters used for the probabilistic analysis of the various elements of the Structure 2 are given. Besides the deterministic parameters (t_0 and t), the normal and lognormal distributions were used to characterise the data stochastically. The symbol *N(…)* and *LN(…)* indicate that the information has a normal distribution and lognormal distribution, respectively. The values between the brackets are the mean value and the standard deviation of the normal distribution of the logarithm of these values for the lognormal distribution. *D(…)* symbolises that the data is deterministic, hence only a single value is given.

In this structure, ordinary Portland cement has been used, hence, for the choice of the critical chloride content, the total binder content and the water/binder ration were taken into account. This value corresponds approximately to 0.40 % by weight of binder (See Table 3.1, Chapter 3).

The time dependency of diffusion parameter was also based on the binder type and the environmental exposure conditions. Due to the scatter of the diffusion coefficient values, a log-normal distribution was used to represent stochastically these values. In this way, negative values are avoided.

Figures 7.9 to 7.14 present the results of the probabilistic analysis performed on the elements of Structure 2 using the software described in the previous chapter. Both the probability of failure versus time and the reliability versus time are presented, for both temperatures considered. The input data files and the result files for these simulations are presented in the Appendix 4.

Table 7.3 – Stochastic parameters for structure 2

Elements/ Parameter	Beam 1	Beam 2	Beam 3
x_C (mm)	N(43.4, 1.7)	N(42.0, 2.8)	N(38.7, 4.0)
D_C (10^{-12} m^2/s)	*LN(0.851, 0.396)*	*LN(0.869, 0.357)*	*LN(1.118, 0.398)*
C_{CR} (%/wt. conc.)*	N(0.06;0.01)	N(0.06;0.01)	N(0.06;0.01)
C_S (%/wt conc.)	N(0.491, 0.036)	N(0.575, 0.060)	N(0.377, 0.024)
α (-)	N(0.30;0.07)	N(0.30;0.07)	N(0.30;0.07)
t_0 (days)	D(5110.0)	D(5110.0)	D(5110.0)
t (years)	D(50.0)	D(50.0)	D(50.0)

* 0.06 for a 350kg cement more or less 0.4 % weight of cement
† see Table 6.7 for normal values

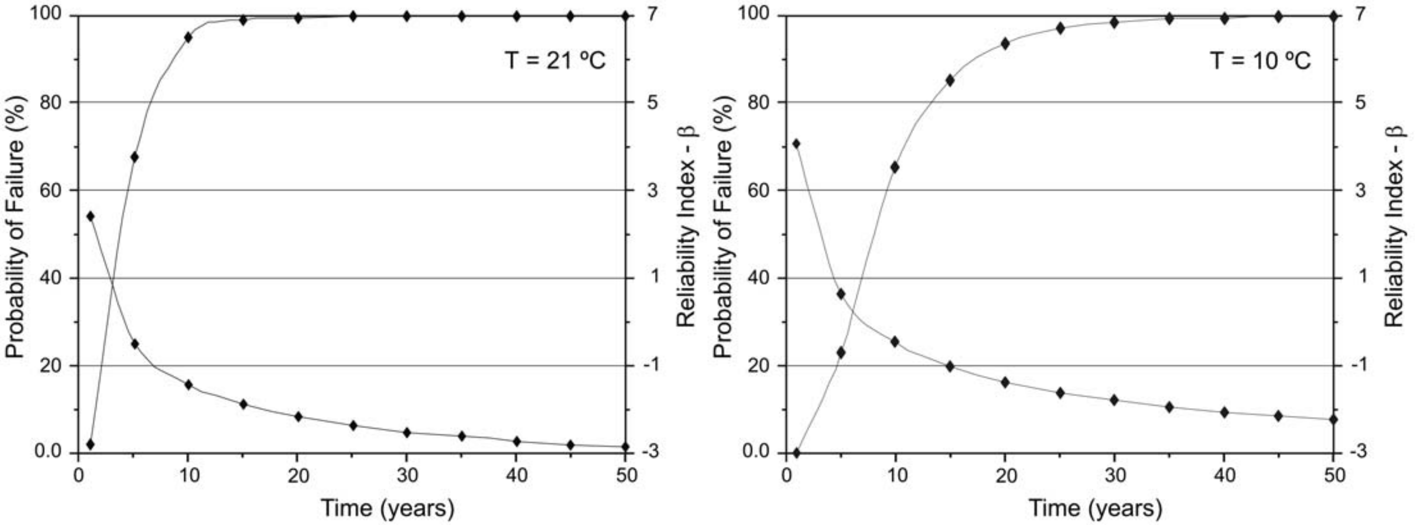


Figure 7.9 and 7.10 – p_F and β versus time for beam 1.

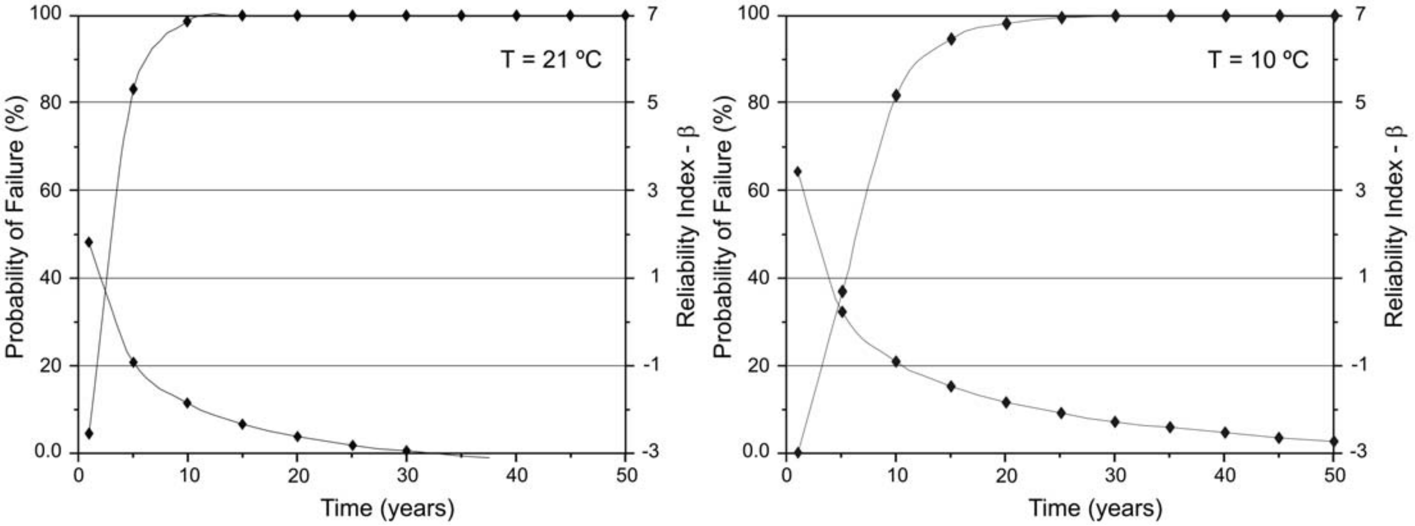


Figure 7.11 and 7.12 – p_F and β versus time for beam 2.

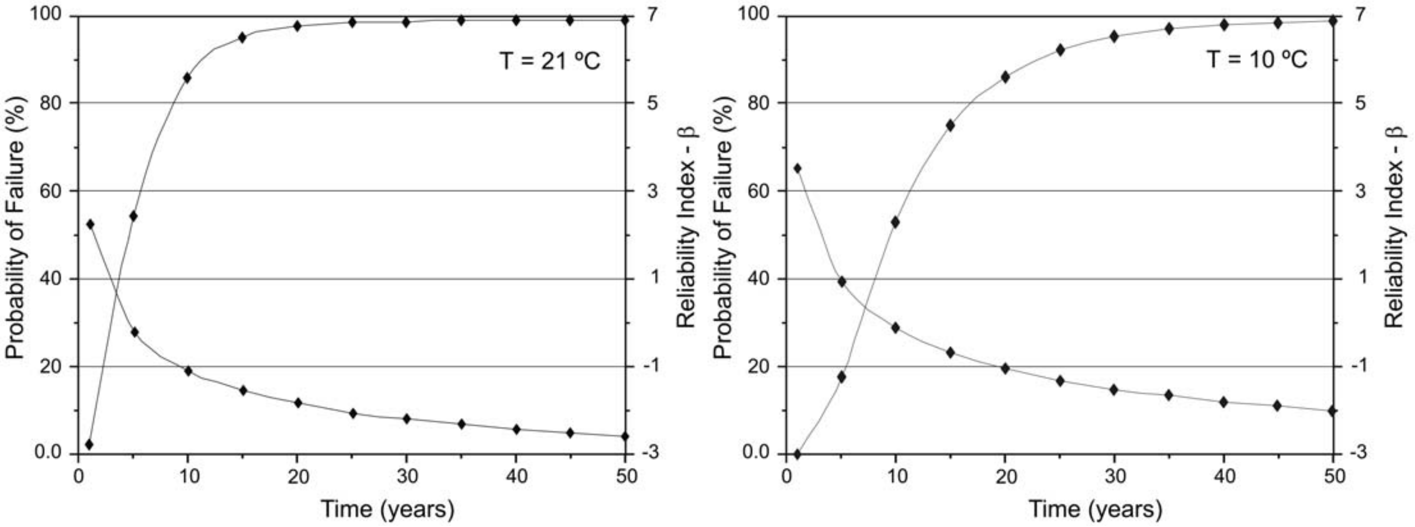


Figure 7.13 and 7.14 – p_F and β versus time for beam 3.

Although all the beams have rather poor concrete quality, beam 3 has a higher diffusion coefficient, and smaller concrete cover. Beam 2 has higher chloride content on the surface of the beam. Despite these differences, the overall conditions are unfavourable for all the beams evaluated, with too little concrete cover and too poor concrete quality. If we take into account the age of the structure (14 years) and the poor concrete quality, the visibility of corrosion signs are expected. This was confirmed in Chapter 6.

There is very little difference between the results of the analyses performed using different temperatures. At both 10ºC and 21ºC, the beams reach the probability of failure limit of 10 % in a period from 2 to 4 years.

7.5 Probabilistic analysis of the durability performance of Structure 3

In Table 7.4, the statistical parameters used for the probabilistic analysis of the various elements of the Structure 3 are given. Besides the deterministic parameters (t_0 and t), the normal and lognormal distributions were used to characterise the data stochastically. The symbols *N(…)* and *LN(…)* indicate that the information has a normal distribution and lognormal distribution, respectively. The values between the brackets are the mean value and the standard deviation of the normal distribution of the logarithm of these values for the lognormal distribution. *D(…)* symbolises that the data is deterministic, hence only a single value is given.

For structure 3, ordinary Portland cement was used. Therefore, for the choice of the critical chloride content, the total binder content and the w/c ratio were taken into account. This value corresponds approximately to 0.4 % by weight of binder (See Table 3.1, Chapter 3).

Table 7.4 – Stochastic parameters for structure 3

Elements/ Parameter	Beam	Anchor slab 1	Anchor slab 2
x_C (mm)	N(60.8, 1.8)	N(60.0, 5.0)	N(60.0, 5.0)
D_C (10^{-12} m^2/s)	N(1.589, 0.432)	N(1.356, 0.288)	*LN(0.210, 0.383)*
C_{CR} (%/wt. conc.)*	N(0.07;0.01)	N(0.07;0.01)	N(0.07;0.01)
C_S (%/wt conc.)	N(0.518, 0.070)	N(0.717, 0.087)	N(0.581, 0.103)
α (-)	N(0.37;0.07)	N(0.37;0.07)	N(0.37;0.07)
t_0 (days)	D(2555.0)	D(2555.0)	D(2555.0)
t (years)	D(50.0)	D(50.0)	D(50.0)

* 0.07 for a 400kg cement more or less 0.4 % weight of cement
† see Table 6.11 for normal values

The time dependency of diffusion parameter was also based on the binder type and the environmental exposure conditions (location of the structure in relation to the sea).

Figures 7.15 to 7.28 present the results of the probabilistic analysis performed on the elements of Structure 3 using the software described in the chapter 4. Both the probability of failure versus time and the reliability versus time are presented, for both temperatures considered. The input data files and the result files for these simulations are presented in the Appendix 4.

The effect that the temperature has on the ingress of chlorides is significant. For all the elements analysed, the time necessary to reach the serviceability limit of 10% probability of failure increased more than 100 %, when the simulation was performed at 10ºC.

The location of Anchor slab 2 is more protected from the environmental exposure than the other elements, but still subject to considerable splash and spray, therefore, it indicates a higher surface chloride content that influence slightly the performance.

Comparing the results of the analysis performed to the beam and anchor slabs 1 and 2, the slightly more protected location of anchor slab 2 results in favourable long term performance. At 10 ºC, the beam would exceed the serviceability limit state of 10 % after approximately 35 years, where as anchor slab 1 and 2 would need approximately 32 years. For a long term, the probability of failure curve of anchor slab 2 is less steep in the final years. If we consider the reference temperature, a decrease in the time to depassivation occurs, with all the elements

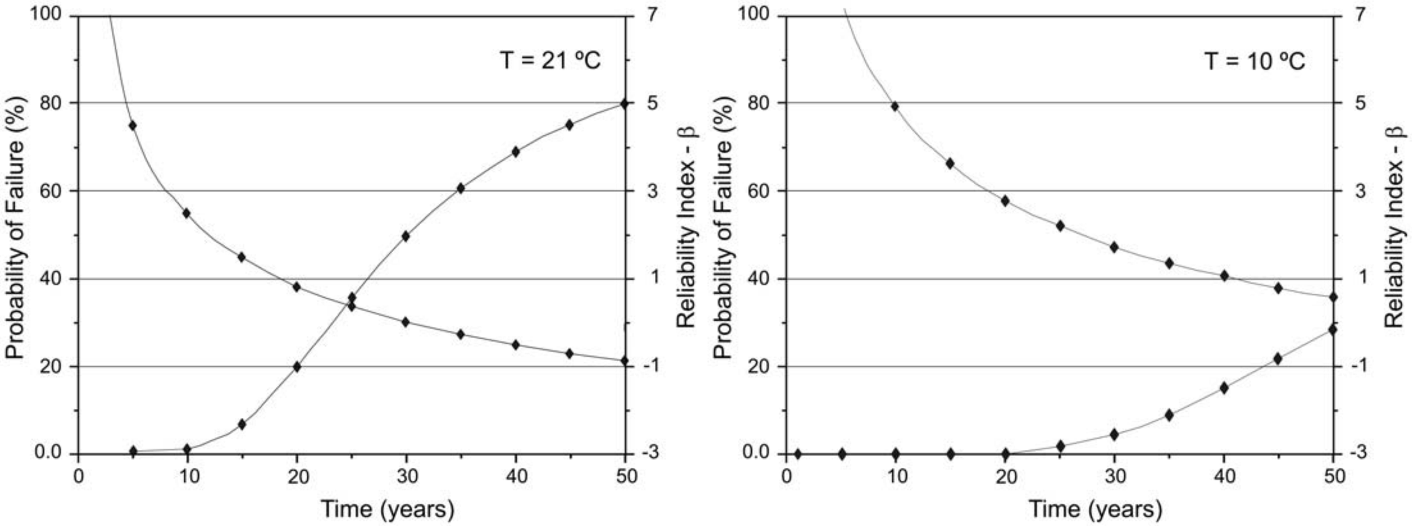


Figure 7.15 and 7.16 – p_F and β versus time for the beam.

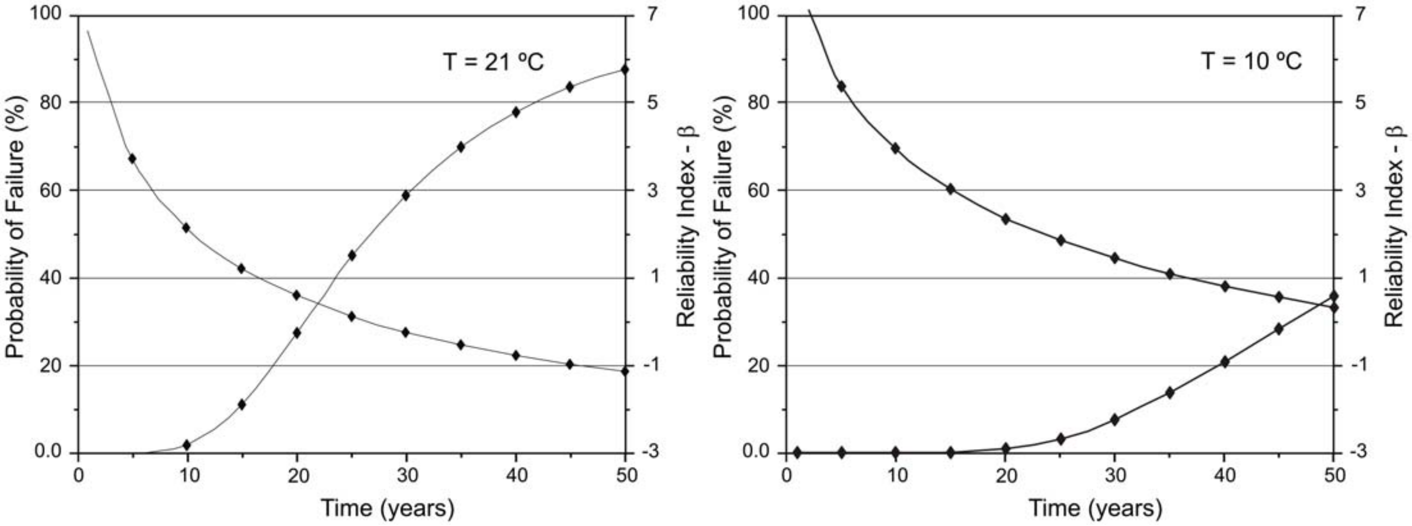


Figure 7.17 and 7.18 – p_F and β versus time for anchor slab 1.

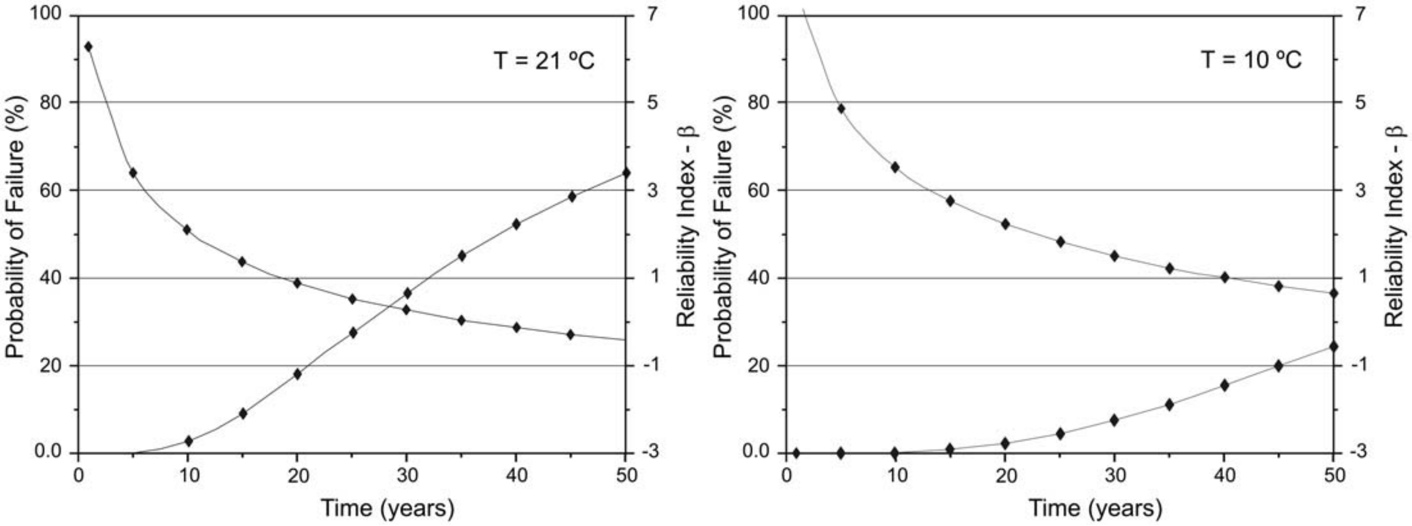


Figure 7.19 and 7.20 – p_F and β versus time for anchor slab 2.

analysed exceeding the limit in approximately 15 years. Despite the small difference related due to the natural stochastic nature of the parameters measured, all elements generally have similar performance.

7.6 Probabilistic analysis of the durability performance of Structure 4

In Table 7.5, the statistical parameters used for the probabilistic analysis of the various elements of Structure 4 are given. Besides the deterministic parameters (t_0 and t), the normal and lognormal distributions were used to characterise the data stochastically. The symbols *N(…)* and *LN(…)* indicate that the information has a normal distribution and lognormal distribution, respectively. The values between the brackets are the mean value and the standard deviation of the normal distribution of the logarithm of these values for the lognormal distribution. *D(…)* symbolises that the data is deterministic, hence only a single value is given.

For the choice of the critical chloride content, the total binder content and the water/binder ratio were taken into account. This value corresponds approximately to 0.40 % by weight of binder (See Table 3.1, Chapter 3), since ordinary Portland cement was used.

The time dependency of diffusion parameter was also based on the binder type and the environmental exposure conditions.

Table 7.5 – Stochastic parameters for structure 4.

Elements/ Parameter	Beam 1	Beam 2	Anchor slab 1	Anchor slab 2
x_C (mm)	N(53.3, 6.1)	N(57.4, 4.1)	N(49.2, 8.1)	N(50.0, 5.0)
D_C (10^{-12} m^2/s)	N(1.23, 0.28)	*LN(0.296, 0.580)*	N(1.81, 0.22)	*LN(0.455, 0.323)*
C_{CR} (%/wt. conc.)*	N(0.07;0.01)	N(0.07;0.01)	N(0.07;0.01)	N(0.07;0.01)
C_S (%/wt conc.)	N(0.620, 0.123)	N(0.303, 0.112)	N(0.608, 0.073)	N(0.703, 0.054)
α (-)	N(0.37;0.07)	N(0.37;0.07)	N(0.37;0.07)	N(0.37;0.07)
t_0 (days)	D(2920.0)	D(2920.0)	D(2920.0)	D(2920.0)
t (years)	D(50.0)	D(50.0)	D(50.0)	D(50.0)

* 0.07 for a 400kg cement more or less 0.4 % weight of cement
† see Table 6.15 for normal values

Figures 7.21 to 7.28 present the results of the probabilistic analysis performed on the elements of Structure 3 using the software described in the previous chapter. Both the probability of failure versus time and the reliability versus time are presented, for both temperatures

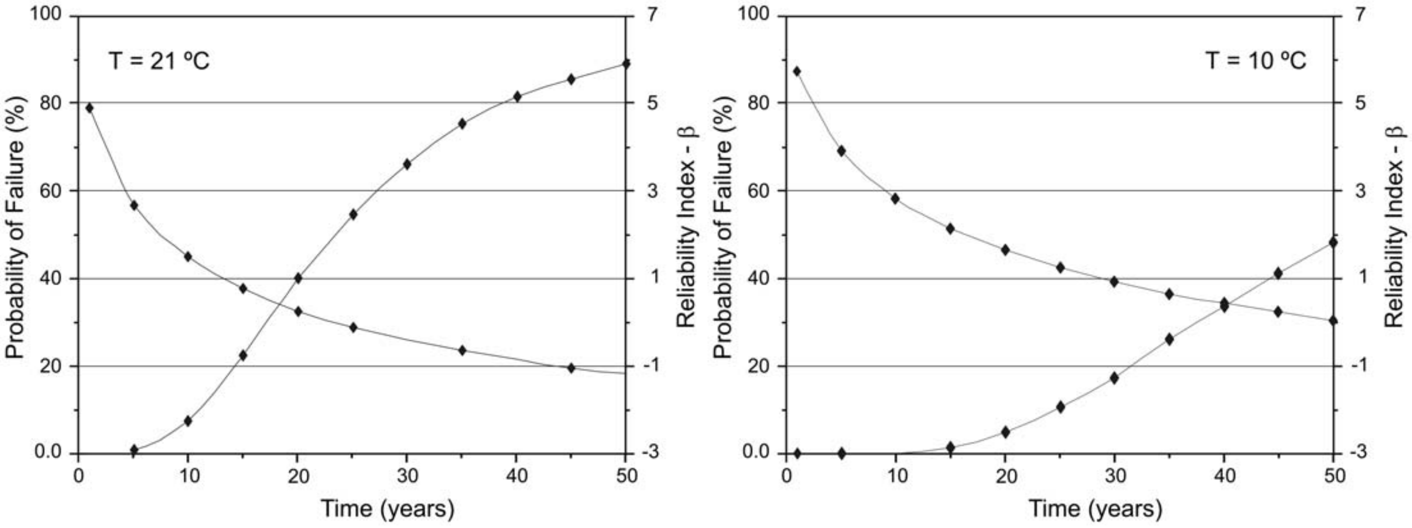


Figure 7.21 and 7.22 – p_F and β for beam 1.

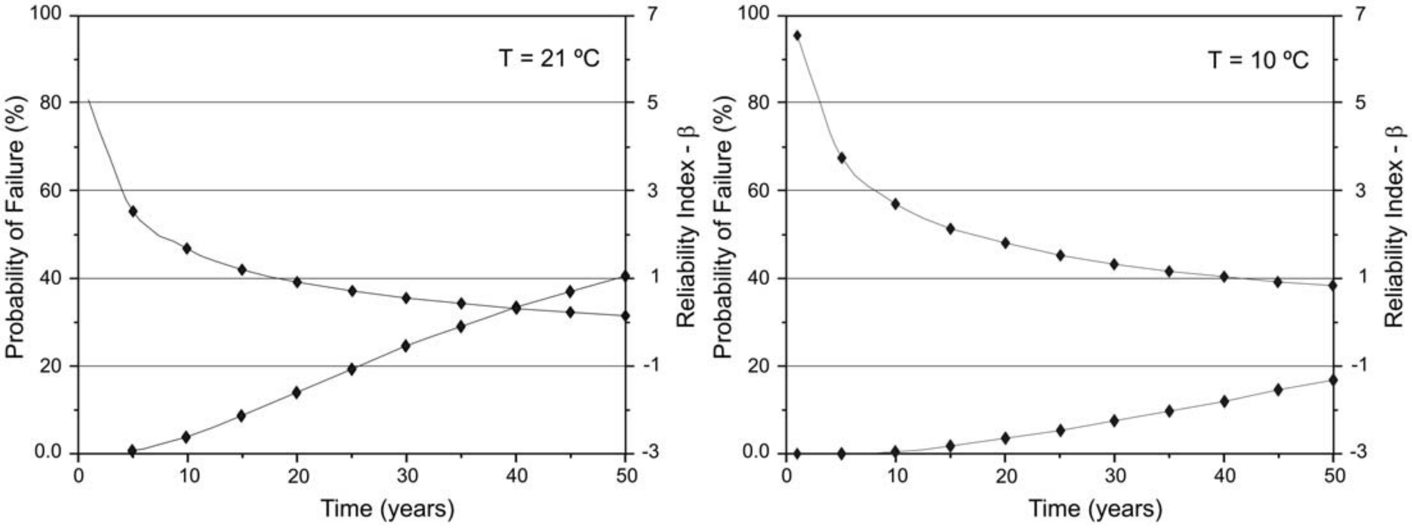


Figure 7.23 and 7.24 – p_F and β for beam 2.

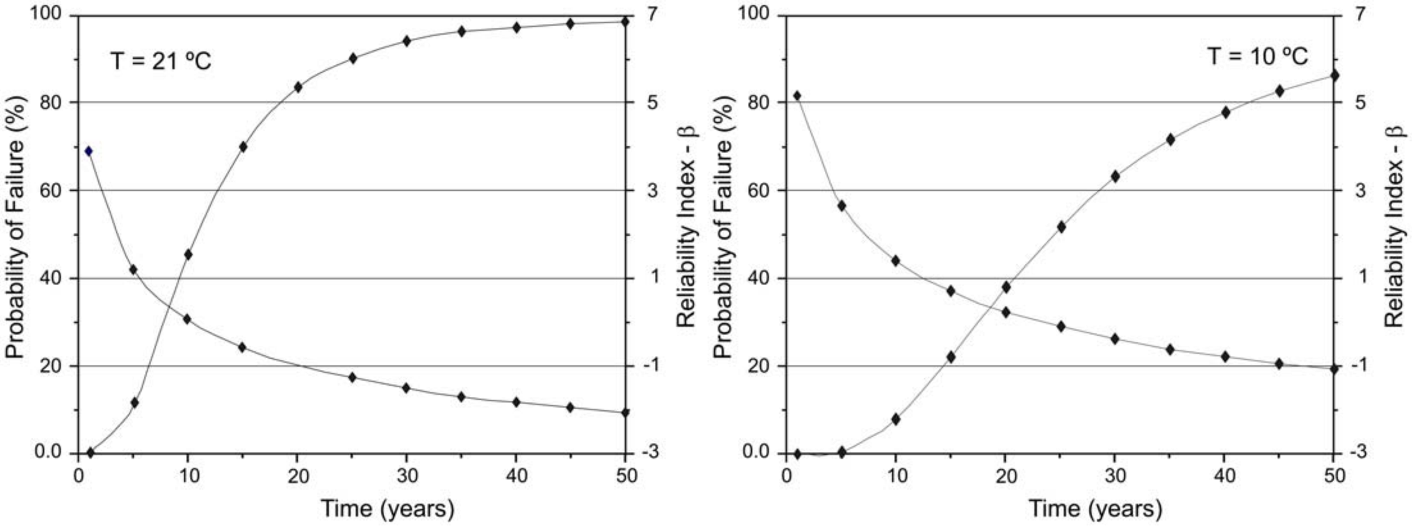


Figure 7.25 and 7.26 – p_F and β for anchor slab 1.

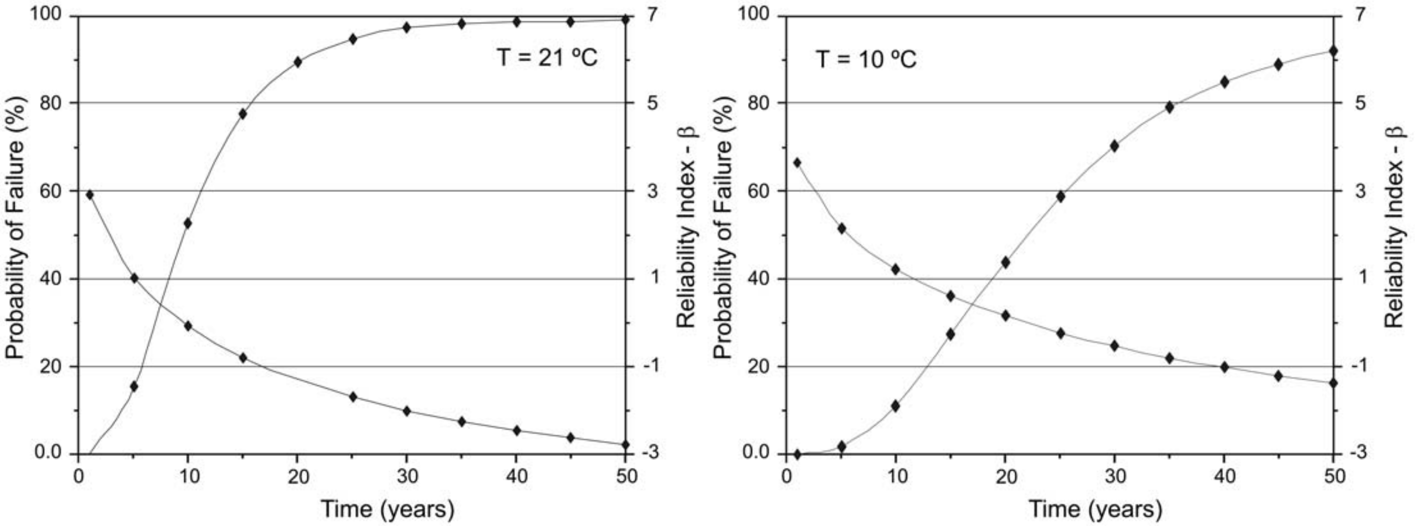


Figure 7.27 and 7.28 – p_F and β for anchor slab 2.

considered. The input data files and the result files for these simulations are presented in the Appendix 4.

The effect of temperature on the ingress of chlorides is significant. For all the elements analysed, the time for a serviceability limit of 10 % probability of failure to be reached increased more than double for the beams and more than triple for the anchor slabs, when the temperature decreased from 21 ºC to 10 ºC.

The two anchor slabs analysed have similar concrete cover, diffusion coefficients and surface chloride content, hence their performance is similar to each other. At 10 ºC, the anchor slabs would exceed the serviceability limit state of 10 % after approximately 10 years, where as, if we consider the reference temperature, a decrease in the time to depassivation occurs, with all the elements analysed exceeding the limit after approximately 5 years.

This does not apply to the beams analysed, since beam 2 is more protected from the environmental exposure and has a lower surface chloride concentration. This is because the height of the deck above the sea level is sufficiently high to avoid the splash and spraying. Comparing the results of the analysis performed for beams 1 and 2, the more protected location of beam 2 results in favourable long term performance. At 10 ºC, beam 2 would exceed the serviceability limit state of 10 % after approximately 35 years, where as beam 1 would need approximately 25 years. If we consider the reference temperature, a decrease in the time to depassivation occurs, with all the beams analysed exceeding the limit in approximately 10 and 15 years for beams 1 and 2, respectively.

7.7 Deterministic versus probabilistic analysis

The effect on design of the service life calculation is considerable when comparing deterministic with probabilistic modelling. This is illustrated in Figure 7.29 and 7.30, where the results of a durability performance analysis are presented for two ages: 25 years and 50 years.

The calculations are based on the data from beam 1 (Structure 1, lateral side) presented in table 7.2.

Table 7.6 – Parameter values for simulation.

Parameter	Value
x_C (mm)	Varied 10 mm to 100 mm
D_C (10^{-12} m^2/s)	N(1,140, 0.162)
C_{CR} (%/wt. conc.)	N(0.08;0.01)
C_S (%/wt conc.)	N(0.828, 0.079)
α (-)	N(0.39;0.07)
t_0 (days)	D(2920.0)
t (years)	D(50.0) and D(25.0)

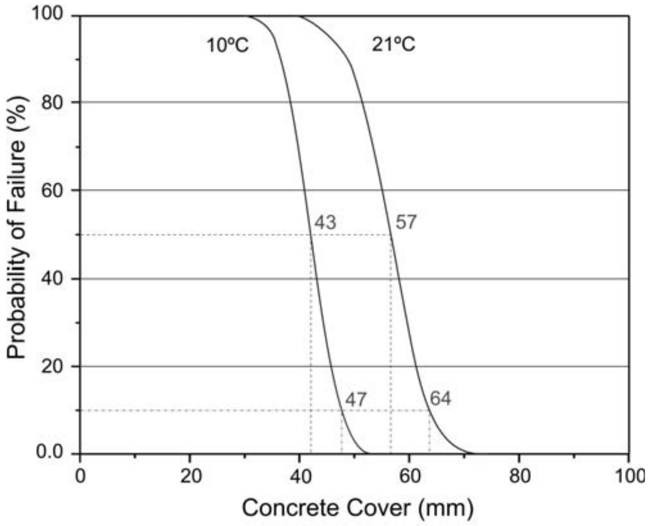


Figure 7.29 - Calculated concrete cover for different temperatures. Results simulated on 25 years of exposure.

Based on the data a probabilistic calculation of required concrete cover was performed. The results of these calculations are shown in Figures 7.29 and 7.30. With a probability of 50% for initiation of corrosion the needed concrete cover varies between 43, for an average environment temperature of 10 ºC, to 57 mm, for an average environment temperature of 21 ºC, for a service life of 25 years. The 50% probability corresponds to the deterministic results. For the serviceability limit state of 10 % (probability of corrosion initiation), the cover thickness needed for 25 years service life becomes 47 and 64 mm for an average environment temperature of 10 ºC and 21 ºC, respectively.

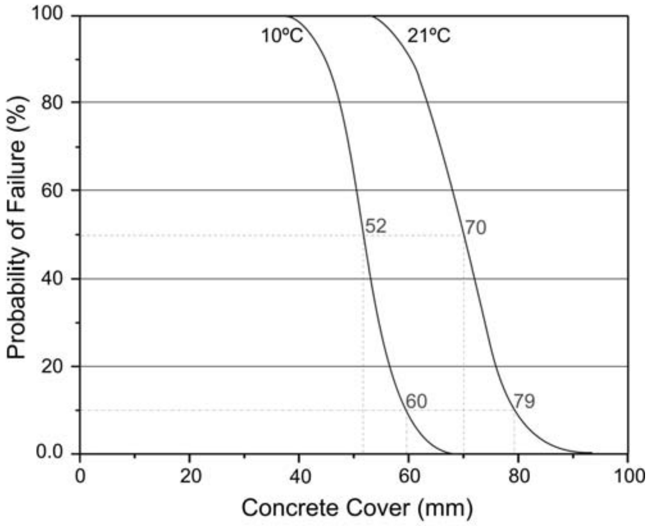


Figure 7.30 - Calculated concrete cover for different temperatures. Results simulated on 50 years of exposure.

With a probability of 50% for initiation of corrosion the needed concrete cover varies between 52 for an average environment temperature of 10ºC to 70 mm for an average environment temperature of 21ºC, for a service life of 50 years. The 50% probability corresponds to the deterministic results. For the serviceability limit state of 10 % (probability of corrosion initiation), the cover thickness needed for 50 years service life becomes 60 and 79 mm for an average environment temperature of 10ºC and 21ºC, respectively.

The comparison of the concrete cover values resulting from a probabilistic and deterministic analysis show that the first represent an increase between 10 to 15 % over the second. This increase of almost 10 mm, for a 50 years service life, has already be shown to be very important for the long term durability of the structure in marine environment (see Figure 4.6).

On the other hand, by designing a structure for a cold environment, such as Norway, the use of realistic average temperatures results in more economic designs, with a reduction of the necessary concrete cover by approximately 30%. However, if designing structures for warm climate with average temperatures over 21ºC, care should be take as the reference temperature is no longer represents a design on the safe side.

7.8 Summary

The results from analysis of the durability performance of the structures demonstrate how traditional condition assessment can be expanded and improved by use of proposed model. The data collected from the condition assessment can be used in the model to evaluate long term performance.

The analyses of the results indicate that the environmental loading changes with the local positioning of the elements in the structure. Some elements, although protected from the rain and wind, can be subjected to splashing and spraying. In such cases, there is an accumulation of the concentration of chlorides on the surface (Anchor slab 2, Structure 3). However, the elements, if placed high enough from the sea level, are not subject to the spray and splash and therefore have lower chloride content (Beam, Structure 3). Even for the same element, different surfaces can have different performance due to their orientation regarding the predominant wind and rain direction. This is seen from data for Beam 1, Structure 1.

This is an interesting observation as it may have some repercussion in the design procedure. Specific design procedures may be adopted of individual elements depending on their location in the structure.

Table 7.7 – Approximate time to reach serviceability limit (years).

Structure	Element	21ºC		10ºC	
		p_F *(10%)*	β *(1.5)*	p_F *(10%)*	β *(1.5)*
Structure 1	Beam 1 (side)	16	13	35	30
	Beam 1(under)	8	7	19	17
	Beam 2 (side)	12	11	27	25
	Beam 2(under)	13	13	28	26
Structure 2	Beam 1	2	2	3	4
	Beam 2	2	2	3	4
	Beam 3	2	3	3	4
Structure 3	Beam	16	15	36	33
	Anchor Slab 1	14	13	32	30
	Anchor Slab 2	15	13	33	30
Structure 4	Beam 1	11	10	25	23
	Beam 2	16	12	35	25
	Anchor Slab 1	5	5	10	10
	Anchor Slab 2	4	4	9	8

Table 7.7 presents the approximate time for the serviceability limit state violation (either the 10 % probability of failure or the reliability index of 1.5), for the average temperatures considered.

The values for probability of failure and reliability index are similar due to the mathematical manner in which they are determined.

The improvement in the performance of the structure while using a more realistic 10 ºC rather than the reference temperature of 20 ºC is apparent in the increase (more than double) in the time to reach the serviceability limit. The use of the reference temperature in such situations could result in the under estimation of performance anticipating the need for intervention on the structure.

The comparison of the concrete cover values resulting from a probabilistic and deterministic analysis show that the first represent an increase between 10 to 15 % over the second. This increase of almost 10 mm, for a 50 years service life, has already been shown to be very important for the long term durability of the structure in marine environment (see Figure 4.6).

On the other hand, by designing a structure for a cold environment, such as Norway, the use of realistic average temperatures results in more economic designs, with a reduction of the necessary concrete cover by approximately 30%. However, if designing structures for warm climate with average temperatures over 21ºC, care should be take as the reference temperature is no longer represents a design on the safe side.

8. CONCLUSIONS

8.1 General conclusions

The research work presented here compromises a literature review, development of a model for evaluation of marine structures, field investigation on four marine structures, and development of computer aided program for evaluation of existing or new marine concrete structures.

The literature review covered concrete durability and processes involved in its deterioration. Special attention was given to corrosion of steel reinforcement specifically in marine environment. The service life concept is detailed and mathematical models for the various phases of the corrosion induced degradation of concrete are discussed.

As a result of the literature review, it became clear that, in order to improve the accuracy of the predictions of the service life of reinforced concrete structures, a performance based methodology should be adopted associated with probabilistic methods. Using a performance based methodology the performance of the structure, defined by the limit state, can be evaluated for the service life. By using probabilistic methods, it is possible to take into consideration the variations of material properties, the environmental loading and the effect of the quality of execution during construction.

Application of the described probabilistic methodology is relevant for structures in marine environment that should have specified service lives, which is the case for structures such as bridges and wharfs. The prediction model developed in this research work had the prime objective of being a sufficiently simple model while maintaining the accuracy and dependability of results obtained. Many of the proposed models in the literature are too detailed and require too much data to be used. Furthermore, there are few engineers with sufficient knowledge of reliability analysis to be able to use a model in a probabilistic manner. The simplicity of the model proposed ensured its user-friendly condition for designers and practitioners in charge of design and evaluation of concrete structures. The proposed model is based on the same degradation mechanisms and laws as most others, i.e. Fick’s second Law, with essential alterations being introduced. The accuracy of the predictions was ensured by taking into account the improvements to the second Fick´s law including the time dependency of the model and the temperature effect on the diffusion coefficient. These changes increased the accuracy of the prediction based on Fick’s second law, while maintaining equilibrium between complexity and usability. It was ensured that the input data necessary could be easily obtained by standard test procedures, and hence reducing the dependency on tabled data based on few selected case studies.

Additional advantages of the model are the ability to use different diffusion coefficients, obtained either from the Rapid Chloride Migration or the Chloride Profile Method. The model can also perform the evaluation of the chloride profiles and the determination of the surface concentration of chlorides, as well as, the diffusion coefficient based on the best fit of the profile by Fick's second law and the prediction of future chloride profiles based on present profiles. From a probabilistic point of view, the use of several distribution types and the incorporation of a variable to account for model variability are also advantageous.

The use of the model, incorporated in the software, enables an accurate evaluation of the concrete structure performance by providing valuable information on the effect of different parameter options on the performance outcome. This is of special importance in decision making process, especially when design options are considered.

The sensitivity analysis performed indicated the weight the various parameters of the proposed model had on the structural performance. The analysis helps the user of the software to understand how each variable influences the performance of the concrete structure, and how susceptible to change each variable is. The most influential parameters were the concrete cover (x_C), the diffusion coefficient (D_0) and the age factor of the diffusion coefficient (α), where small changes in the average value resulted in significant performance variations.

The model was also tested on existing structures. The inspection of the built quality of the structures was performed through the evaluation of the concrete cover depth, the chloride diffusivity and by electrical potential mapping of the concrete cover. The environmental response was evaluated by measuring the chloride penetration (profiles), the carbonation depth, and the corrosion activity through visual damage assessment.

The four structures examined in this investigation had ages ranging from 7 years up to 14 years of age. An effort was made to collect as much information as possible, from the most significant elements of these structures.

All of the structures' owners had no documentation concerning the concrete quality of their structures. Furthermore, few documentation existed on the concrete mix design used. However, some information concerning the concrete properties was obtained through contacts with the construction companies that built the structures and analysis of the structures design projects.

This scenario, while common, illustrates the current state of awareness towards concrete durability.

In terms of built quality, the concrete cover readings and the diffusion coefficients for the different elements of the structure show that the quality is generally uniform, whether above or below the prescribed.

Structure 2 is almost double the age of the other structures assessed. The longer period exposed to the environment in conjunction with the poorer concrete quality make it the structure with the poorest concrete quality of the structures assessed.

In Structures 2, 3 and 4, due to the presence of chlorides in the concrete, the overall electro-chemical potential readings normally suffer a shift in value of approximately -150 mV. However, for localized (pitting) corrosion, it is the presence of the gradients that is of interest. The gradients that present a drop of -150 mV/m are of special interest.

The proposed model permits reliability based inspection planning for a predefined safety level of structures. By combining the present model with information about costs and the classical decision theory, calculation of the overall life cycle cost for structures can be performed. This would provide the owner with a decision basis for evaluating the economic consequences of alternative solutions where the reliability is made clearly visible.

The presented methodology has a significant potential for the future. It is foreseen that within the next decade the reliability based overall service life design may well have replaced today's deemed to satisfy design procedures.

A general analysis of the structures concrete durability assessment results shows that the different elements of the structures possess a generally uniform quality, whether above or below the prescribed. Furthermore, large variations in the exposure environment may occur even on a single structure. This is vital in that, different parts of the same structure should be designed according to their location, i.e. local exposure environment.

The model proposed adopts the serviceability limit state of depassivation. It is important to note that, although a useful point for the service life design of a new structure, it is evident that many structures continue to function, with or without repair, once corrosion has started. Therefore, it is important to understand the concept of service life design appropriately, and in this way avoid misinterpretations. The serviceability limit state chosen does not represent a limit state in which structural damage has occurred. It is a limit state that provides important information as to the condition of the structure and an indication of the time for planning adequate maintenance procedure.

Due to the overall lack of adequate design, many structures in marine environment have serious corrosion problems within the first few years of service. Through the use of the model, a designer can realize the consequences of design options concerning the concrete material and structural dimensions. The validation of the proposed model can be seen by comparing the results from the assessments of the structures with the probabilistic simulation of the service life of the structures. The effect of changes in model parameters on the overall performance can now be rapidly evaluated.

The analyses of the structural assessment results indicate that the environmental loading changes with the local positioning of the elements in the structure. Some elements, although protected from the rain and wind, can be subjected to splashing and spraying. In such cases, there is an accumulation of the concentration of chlorides on the surface (Anchor slab 2, Structure 3). However, the elements, if placed high enough above the sea level, are not subject to the spray and splash and therefore have lower chloride content (Beam, Structure 3). Even for the same element, different surfaces can have different performances due to their orientation regarding the predominant wind and rain direction. This is seen from data for Beam 1, Structure 1. This is an interesting result as it may have repercussion in the design procedure. Specific design procedures may be adopted for individual elements depending on their location in the structure.

In the case of structure 2, visual inspection shows rust staining and cracks indicating that corrosion has already reached a critical point. This is confirmed by the simulation where, in a period of less than 3-4 years, the structure should have reached depassivation and corrosion initiated. The results of the simulation performed regarding the concrete degradation due to corrosion are in accordance with the results from visual inspection of the other structures.

The results from the analysis of the durability performance of the structures demonstrate how traditional condition assessment can be expanded and improved by using the proposed model. The data collected from the condition assessment can be used in the model to evaluate long term performance.

It is important to emphasize the importance of temperature in the model. The improvement in the performance of the structure while using 10 ºC instead of a temperature of 21 ºC is apparent in the increase (more than double) in the time to reach the serviceability limit. The use of the reference temperature, i.e. 21 ºC, in such situations results in an underestimation of adequate performance.

The comparison of the concrete cover values resulting from a probabilistic and deterministic analysis show that the first represent an increase between 10 to 15 % over the second. This increase of almost 10 mm, for a 50 years service life, is seen to be very important for the long term durability of structures in marine environment (see Figure 4.6).

8.2 Future research

Computer aided design makes space for continuous improvements as resources and new utilizations are made available with time. However, a line has to be drawn between the academic application of knowledge and the in-situ use of this knowledge. Often, the knowledge accumulated through research is not easily passed on to the construction industry in time and in an adequate manner.

As recommendations for future research work, improvements can be made on both the model and the software. Further research work is needed in enhancing the user-friendliness of the software. Simultaneous assessments and graph facilities could be introduced. Newer models should include other limit state criteria such as cracking and spalling.

There are many areas in which further research are needed to improve the current knowledge of the parameters of the model. Some of these are: the chloride binding and the time dependency of cement types in various environments to better quantify the related parameters; the parameters that influence the cracking of concrete due to corrosion, and the development of models that are able to simulate this behaviour adequately.

9. REFERENCES

1991 **ACI Committee 201** - Guide to durable concrete. ACI Mater J 1991;88(5).

1997 **Ahmad**, S.; Bhattacharjee, B.; Assessment of service lives of reinforced concrete structures subjected to chloride-induced rebar corrosion. Journal Structural Eng 1997;23(4): 177-82.

1997a **Ahmad,** S.; Bhattacharjee, B.; Wason, R.; Experimental service life prediction of rebar corroded reinforced concrete structure. ACI Materials Journal 1997;94(4):311-6.

1995 **Akita, H.**; Fujiwara, T.: Water and salt movement within mortar partially submerged in salty water. Proc. of CONSEC 95, Sapporo, Japan, E&FN Spon, London.

1985 **Al-Amoudi** OSB. Studies on the evaluation of permeability and corrosion resisting characteristics of portland pozzolan concretes. M.S. Thesis, King Fahd University of Petroleum and Minerals, Dhahran; 1985.

1992 **Al-Saadoun** SS, Rasheeduzzafar, Al-Gahtani AS. Mix design considerations for durable concrete in the Arabian Gulf environment. The Arab J Sci Eng 1992;17(1):17-33.

1996 **Amey, S.L.;** Johnson, M.A.; et al; A methodology for predicting service life of concrete structures exposed to marine environments. ACI SP-163, Detroit, USA.

1997 **Andrade, C.;** Arteaga, A.; et al. Models for predicting corrosion rates. Brussels-Brite-EuRam, 1997, Project nº BE95-1347.

1996 **Andrade**, C. and Alonso, C.; Corrosion rate monitoring in the laboratory and on site, *Construction and Building Materials*, 1996, Vol. 10, No. 5, 1996, pp. 315-328.

1990 **Andrade**, C.; Alonso, M.C.; Gonzalez, J.A.; An initial effort to use the corrosion rate measurements for estimating rebar durability. Corrosion Rates of Steel in Concrete, ASTM STP 1065, N.S. Berke, V. Chaker and D. Whiting, Eds., American Society for Testing and Materials, Philadephia (1990) 29-37.

1987 **Anonnymus**., Concrete Durability: A Multi Billion Dollar Opportunity, Rep. NMAB-437 Defense Technical Information Center. Cameron Station. Alexandria. 1987.

2004 **Årskog**, V., Ferreira, M. and Gjørv, O.E, ”Practical Guidelines for Probability-Based Durability Design of Concrete Structures in Severe Environment”, Report BML 200403, Department of Structural Engineering, Norwegian University of Science and Technology, NTNU, Trondheim, Norway, 2004.

1985 **Arup**, H., Electrochemical monitoring of the corrosion state of steel in concrete, 1 st Int. Conf. on Deterioration & Repair of Reinforced Concrete in the Arabian Gulf, 26-29 Oct. 1985.

1993 **Arup,** H.; The Study of Transportation Mechanisms Using Migration Experiments. Some Caveats and Ideas for Future Research. Presented at "Chloride Penetration into Concrete Structures", a Nordic Miniseminar in January 1993, Goteborg.

1987 **Arya** C.; Buenfeld, N.R.; Newman, J.B.; Assessment of simple methods of determining the free chloride ion content of cement pastes. Cement Concrete Res 1987;17:907-18

2003 **ASTM C227-03**, Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method), ASTM, 2003

2003 **ASTM C289-03**, Standard Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method), ASTM, 2003

2003 **ASTM C666M-03**, Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing, ASTM, 2003

1994 **ASTM C671-94**, Standard Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing (Withdrawn 2003), ASTM, 1994

1983 **ASTM Special Technical Publication 818**, Corrosion of metals in association with concrete, ASTM, 1983.

1994 **Bamforth**, P. B. and Chapman-Andrews, J., Long term performance of r.c. elements under UK coastal exposure conditions, *in Corrosion and Corrosion Protection of Steel in Concrete,* Sheffield Academic Press, Ed Swamy, 1994, pp. 139-156.

1990 **Bamforth**, P. B. and Pocock, D. C, (1990) Minimising the risk of chloride induced corrosion by selection of concreting materials, in *Proc. 3rd Int. Symp. "Corrosion of Reinforcement in Concrete Construction* , SCI, Wishaw, May 1990, pp. 119-131.

1993 **Bamforth**, P. B., (1993) Concrete classification for r.c. structures exposed to marine and other salt laden environments, in *Structural Faults and Repair 93, Vol II*, Edinburgh, June/July 1993, pp. 31-40.

1995 **Bamforth**, P. B., (1995) A new approach to the analysis of time-dependent changes in chloride profiles to determine effective diffusion coefficients for use in modelling of chloride ingress, in *RILEM International Workshop on Chloride Penetration into Concrete,* St Remy-les-Chevreuse, France, Oct 1995, Paper 21,pp.195-205.

1996 **Bamforth**, P. B., Predicting the risk of reinforcement corrosion in marine structures, O E Gjorv Symposium 'Concrete for Marine Structures, in *3rd CANMET/ACI International Conference on Performance of Concrete in Marine Environment*, New Brunswick, 1996, pp. 207-233

1999 **Bamforth**, P. B., *The derivation of input for modeling chloride ingress from eight-year UK coastal exposure trials*, Magazine of Concrete Research, 51-n^{o}2, pp.87-96, 1999

1999 **Bamforth**, P. B.; The derivation of input data for modelling chloride ingress from 8 year UK coastal exposure trials. *Magazine of Concrete Research. 1999*

1998 **Bamforth**, P. Double Standards in Design. CONCRETE, December 1998.

1994 **Bamforth,** P.B.; Chapman-Andrews, J.F.: Long term performance of R. C. elements under UK coastal exposure conditions, in Corrosion and Corrosion Protection of Steel in Concrete, edited by R.N. Swamy (Sheffield Academic Press, Sheffield), Vol. 1, p. 139-156.

2001 **Basheer, L**., Kropp, J. & Cleland, D.J. (2001), Assessment of the durability of concrete from its permeation properties: a review, Construction and Building Materials, No. 15, pp. 93-103, 2001.

1979 **Bazant**; Z. P.; Physical model for steel corrosion in concrete sea structures - Theory. ASCE J Structural Division 1979;105:1 137-53.

1979a **Bazant**; Z. P.; Physical model for steel corrosion in concrete sea structures - Application. ASCE J Structural Division 1979;105:1 155-66.

1993 **Beeby**, A W, 'Design for life', Concrete 2000 - Economic and durable concrete through excellence, Vol 1, Ed. R K Dhir and M R Jones, E & F N Spon, 1993. pp.37-50.

1978 **Beeby** AW. Corrosion of reinforcing steel in concrete and its relation to cracking. The Struct Engr 1978;56A(3):77-81.

1994 **Berke**, N.S., Hicks, M.C., *Predicting Chloride Profiles in Concrete*, Corrosion, 50, pp.234-239, 1994.

1990 **Berkely,** K.G.C., Pathmanaban S.; Cathodic protection of reinforcement steel in concrete. London: Butterworths & Co. Ltd.; 1990.

2004 **Bertolini, L.;** Elsener, B.; et al; Corrosion of Steel in Concrete. Prevention, Diagnosis, Repair. Wiley-VCH, p.392. 2004.

1997 **Beslac**, J., Hranilovic, M., Maric, Z. and Sesar, P.: The Krk Bridge: chloride corrosion and protection, Proceedings, International Conference on Repair of Concrete Structures – From Theory to Practice ina Martine Environment, Ed. by A. Blankvoll, Norwegian Public Roads Administration, Oslo, pp. 501-506, 1997.

1994 **Betonghandboken** - Material (Concrete handbook - Material), (1994), edited by Ljungkrantz, C., Möller, N. & Petersons, N., Svensk Byggtjänst, Stockholm, 1994. (in Swedish).

1964 **Biczock,** L.; Concrete corrosion, Concrete Protection, Hungarian Academy of Science, Budapest, 117, 1964

--- **BLOODHOUND,** Potential and Resistance Mapping Device, Cyberdan A/S, Skovlytoften 26, DK-2840 Holte, Denmark

1999 **Boddy**, A., E. Bentz, et al. An overview and sensitivity study of a multimechanistic chloride transport model. Cement and Concrete Research, 29(6), 1999, 827-837.

1983 **Brately**, P., Fox, B.L., Schrage, E.L., *A Guide to Simulation*, Springer Verlag, New York, 1983

1997 **Breit**, W., *Untersuchungen zum kritischen korrosionsauslösenden Chloridgehalt für Stahl in Beton.*, in Aacher Beitrage zur Bauforschung, 8, Aachen, Technische Hochschule, Thesis, 1997.

1994 **BRE Report 216** - Durability of pfa concrete', Thomas, M.; Matthews, J.D.; BR216 (1994).

1999 **BRE Digest 444** - Corrosion of steel in concrete (in 3 parts), (1999).

1997 **Broomfield, J. P**. (1997), Rebar corrosion – What do we know for sure, Proceedings International Conference on Repair of concrete structures – From Theory to Practice in a Marine Environment, Svolvær, 1997. pp. 35-47.

1982 **Browne, R.D.:** Design prediction of the life for reinforced concrete in marine and other chloride environments. Durability of Building Materials. Elsevier Scientific Publishing Company. Amsterdam Holland. No. 1, pp. 113-125.

1982 **BS 5075-1**, Concrete admixtures: Part 1 - Specification for accelerating admixtures, retarding admixtures and water-reducing admixtures, BSI, 1982.

2002 **BS 5328**, Concrete. British Standards, 2002.

1997 **BS 8110-1,** Structural use of concrete - Part 1: Code of practice for design and construction. British Standards Institute, 1997

2002 **BS 8500,** Concrete - Complementary British Standard to BS EN 206-1. British Standards, 2002.

1996 **Bukowski, R.W.;** Risk and Performance standards. NIST Building and Fire Research Laboratory, USA, 1996

1993 **Byfors**, K., "Chloride-initiated reinforcement corrosion, Chloride binding", CBI Report 1:90, Swedish Cement and Concrete Research Institute, 1990.

1990 **Cady, P.D.; Weyers, R.E.:** Chloride penetration and deterioration of concrete bridge decks. Cement, Concrete and Aggregates, CCAGDP, Vol. 5, No. 2, pp. 81-87. American Society for Testing and Materials. USA.

1990 **Cady**, P. D., University of Pennsylvania, Private Communications, 1990.

2003 **Cairns, J.; Law, D.;** Prediction of the ultimate state of degradation of concrete structures. Proceeding of ILCDES 2003, Kuopio, pp169-174, 2003.

1995 **Carlsen** J.E., Chloride profile based on sampling of dust by hand drill with vacuum attachment. Proceedings of the Nordic Seminar in Lund, February 1-2, 1995 on corrosion of reinforcement. Field and Laboratory Studies for Modelling and Service Life. University of Lund, Sweden. Report TVBM-3064, 1995.

1988 **Chen**, Z. Y., Wang, N. Z., "The effect of the cement paste-steel interracial zone on the electrochemical properties of reinforcing rods", Mat. Res Soo Svmo Proc, Vol. 114, Materials Research Society 1988 V P'

1970 **Collepardi** M., Marcialis A., Turriziani R.: The Kinetics of Chloride Ions Penetration in Concrete, *Il cemento*, Vol. 67, pp. 157-164, 19??.

2000 **Concrete Society,** Technical Report nº. 54 – Diagnosis of deterioration in Concrete Structures. Identification of defects, evaluation and development of remedial action. P.76, 2000.

1963 **Cordon** WA, Gillespie HA. Variables in concrete aggregates and portland cement which influence the strength of concrete. ACI J Proc 1963;60:1029.

1997 **Costa**, A.J.S., *Durabilidade de Estruturas de Betão Armando em Ambiente Marítimo*, Universidade Técnica de Lisboa, Instituto Superior Técnico, Doctural Thesis, p.545, 1997.

1981 **Coull**, W.A.: Characteristics and service record of commonly used South African aggregates, Proceedings, Fifth International Conference on Alkali-Aggregate Reaction, Cape Town, 1981.

1986 **Crank**, J.: The mathematics of diffusion. Second edition. Clarendon Press. Oxford, UK

1993 **Croce**, S, Luccini, A, Samuelsson, S and Soronis, G, 'CIB W94 design for durability, general report 1', Durability of Building Materials and Components 6, Ed. S Nagataki, T Nireki and F Tomosawa, E & F N Spon, 1993. pp. 1392-1402

1995 **CSEP**; *Introduction to monte carlo methods*. Computational Science Projects, 1995.

1987 **Curie**, R. J., Carbonation depths in structural quality concrete; an assessment of evidence from investigations of structures and from other sources. Building Res. Establishment Rep., 1987, 16.

1992 **Dagher**, H.J.; Kulendran, S.; Finite element model of corrosion damage in concrete structures. ACI Structural Journal 1992;89(6):699-708.

1975 **Dakhil** FH, Cady PD, Carrier RE. Proc J, Am Concr Inst 1975;72(8):421.

1995 **Dhir, R.K**.; Jones, M.R.; McCarthy, M.J.; Pfa concrete: chloride-induced reinforcement corrosion. Mag. Conc. Res, 46, 269-277 (1995).

1983 **Dolar-Mantuani**, L.: Handbook of concrete aggregates – A petrographic and technological evaluation, Noyes Publication, USA, 345 p., 1983.

1997 **DuraCrete**, *Design Framework*, Document BE95-1347/R1, The European Union – Brite EuRam III, Contract BRPR-CT95-0132, Project BE95-1347, CUR, Gouda, 1997.

2000 **DuraCrete**, *General Guidelines for Durability Design and Redesign*, The European Union - Brite EuRam III, Project nºBE95-1347/R15, *Probabilistic Performance based Durability Design of Concrete Structures*, 2000.

1998 **DuraCrete**, *Modelling of degradation*, The European Union – Brite EuRam III, Project No. BE95-1347, *Probabilistic performance based durability design of concrete structures*, No. R 4-5, 1998.

1999 **DuraCrete**, *Probabilistic Methods for Durability Design, Document* BE95- 1347/R0, The European Union – Brite EuRam III, Contract BRPR-CT95-0132, Project BE95-1347, CUR, Gouda, 1999.

2000a **Duracrete**, *Statistical Quantification of the Variables in the Limit State Functions*, The European Union - Brite EuRam III, Project nºBE95-1347/R9, *Probabilistic Performance based Durability Design of Concrete Structures*, 2000.

1986 **Elsener, B.; Böhni, H.:** Corrosion of steel in mortar studied by impedance measurements. In Electrochemical Methods in Corrosion Research, edited by M. Duprat (Trans tech Publications LtD, CH), Vol. 8, Ch. 55, pp. 363-372.

2000 **EN 12350-1**, Testing fresh concrete – Part 1 Sampling, CEN, Brussels, 2000.

2000 **EN 197-1,** Cement - Composition, specifications and conformity criteria Part 1: Common cements, CEN, Brussels, 2000.

2002 **EN 1991-1,** Eurocode 1 : Basis of design and actions on structures - Part 1: Basis of design, CEN (European Committee for Standardization), Brussels, 2002.

2000 **EN 206-1**, Concrete - Part 1: Specification, performance, production and conformity; CEN, Brussels, 2000.

1998 **EuroLightCon**; Chloride penetration into concrete with lightweight aggregates. Document BE96-3942/R2, Project BE96-3942, 1998.

1960 **Ewenson**. E. G.and Gillot, J. E.. Characteristics of Kingston Carbonate Rock Reaction. Highway Res. Board Bui. 275. Washington. D.C., 1960. 23.

2003 **Ferreira,** R.M., “Recommendations for probability-based durability design of concrete structures”, Report BML 200305, Department of Structural Engineering, Norwegian University of Science and Technology, NTNU, Trondheim, Norway, 2003.

2000 **Ferreira**, R.M.; *Avaliação dos ensaios de durabilidade do betão.* Masters Thesis, Universidade do Minho, Guimarães, 246 pp, (2000).

2002 **Ferreira,** R.M.; Relationship between chloride concentrations. Internal report. DEC, University of Minho, 2002.

1999 **fib**: "Structural Concrete. Textbook on Behaviour, Design and Performance. Updated knowledge of the CEB/FIP Model Code 1990". **fib** Bulletins 1-3. Specially: Steen Rostam: Chapter 5: "Durability", and Chapter 8: "Assessment, Maintenance and Repair". 1999.

2000 **fib -** TG 5.6 Model code for service life design of concrete structures, 2000.

1997 **Fluge**, F.: Environmental loads on coastal bridges, Proceedings, International Conference on Repair of Concrete Sturctures – From Theory to Practice in a Marine Environment, ed. by A. Blankvoll, Norwegian Public roads Administration, Oslo, pp. 89-98, 1997.

2001 **Fluge**, F., *Marine chlorides. A probabilistic approach to derive provisions for EN206-1.*, DuraNet (http://www.duranetwork.com/), Tromsø, Norway, 2001, 48-68.

1988 **Foy**, C., Pigeon, M. and Bauthia, N.: Freeze-thaw durability and deicer salt scaling resistance of a 0.25 water-cement ratio concrete. Cement and Concrete Research, Vol. 18, pp. 604-614, 1988.

1997 **Frederiksen, J.M. ;** HETEK, Chloride penetration into concrete, relevant test methods, The Danish Road Directorate, Report No.94, 1997.

2002 **Frederiksen, J.M.;** Methods for determination of chloride threshold values for steel in concrete. NT TECHN REPORT 500, ISSN: 0283-7234, pg64.

1992 **Frederiksen, J.M**.; Sorensen, H.: Chloridindtrcsngning i beton under last (in Danish). High Performance Concretes in the 90'ies, Report No. 2.3, 19 p. AEC Consulting Engineers (Ltd) A/S. Vedbiek, Denmark.

1996 **Frederiksen, J.M.;** Sørensen, H.E.; et al; HETEK: The effect of w/c ratio on chloride transport into concrete – Immersion, migration and resistivity tests. The Danish Road Directorate, Report 54.

1974 **French**, W. J, and Poole, A. B., Deleterious reactions between dolomites from Bahrain and cement pastes. Cement Cone. Res.. 46, 1974.

1990 **Gagne**, R., Pigeon, M. and Aitcin, P.C.: Durabilité au gel bétons de hautes performances mécaniques, Materials and Structures, RILEM Vol. 23, pp. 103-109, 1990 (in French).

1993 **Gautefall**, O.: Experience from nine years exposure of concrete in the tidal/splash zone. Proc. Nordic Seminar on Chloride Transport into Concrete, Chalmers, Gothenburg, Jan. 13-14, 1993.

1975 **Gilliott**, J.E.: Practical implications of the mechanisms of alkali-aggregate reactions, Proceedings, Third International conference on Alkali-Aggregate Reaction, Reykjavik, 1975.

1996 **Gjørv**, O.E. - Odd E. Gjørv Symposium on concrete for marine structures, Proceedings, ed. by P.K. Mehta, CANMET/ACI, pp. 259-279, 1996.

1987 **Gjørv**, O.E. and Bathen, E.: Quality control of the air void system in hardened concrete, Nordic Concrete Research, Publication No. 6, pp. 95-110, 1987.

2000 **Gjørv**, O.E.: Controlled service life of concrete structures and environmental consciousness, Proceedings, International Workshop on Concrete Technology for a Sustainable Development in the 21st Century, ed. by O.E. Gjørv and K. Sakai, E & FN Spon, London and New York, pp. 1-13, 2000

1968 **Gjørv**, O.E.: Durability of reinforced concrete wharves in Norwegian habours, Ingeniørforlaget, Oslo, 208 p., 1968

2002 **Gjørv,** O.E., *Durability and Service Life of Concrete Structures*, Proceedings, The First fib Congress 2002, Session 8, V. 6, Japan Prestressed Concrete Engineering Association, Tokyo, 2002, pp. 1-16.

2002a **Gjørv,** O.E.; Ferreira, R.M.; Liu, G.; Betongflåter for maritime formal. Rapport BML 200125. NTNU, Trondheim, 2002.

2002b **Gjørv, O.E.;** Ferreira, R.M.; Liu, G.; Blast-furnace slag for concrete. Report BML 200204, NTNU, Trondheim, 2002.

1997 **Glanville**, J.; Neville, A.; Prediction of Concrete Durability. E&FN SPON, London, 1997, 185 p.

1997 **Glanville,** J.; Neville, A.;(Editors) Prediction of concrete durability. Proceedings of STATS 21st Aniversary Conference. E&FN Spon, London UK, pg 185.

1995b **Glass, G.K**.; Buenfeld, N.R.: Chloride threshold levels for corrosion induced deterioration of steel in concrete. Proc. Int. RILEM Workshop on Chloride Penetration into Concrete, Saint-Remy-Les-Chevreuse, October 15-18.

1995a **Glass, G.K**.; Buenfeld, N.R.: The determination of chloride binding relationships. Paper presented at the RILEM International Workshop on Chloride Penetration into Concrete, October 15-18, Saint-Remy-les-Chevreuse, France

1991 **Glass**, G.K.; Page, C.L.; Short, N.R.; *Factors Affecting the Corrosion Rate of Steel in Carbonated Mortars* Corrosion Science, **32**, 1283-1294 (1991)

1991 **Goni** S, Andrade C and Page C L. Corrosion behaviour of steel in high alumina cement mortar samples: effects of chloride. Cement and Concrete Research, Vol 21, No 4, 635-46.

1981 **Goto** S, Roy DM. The effect of w/c ratio and curing temperature on the permeability of hardened cement paste. Cement Concr Res 1981;575.

1993 **González**, J.A., López, W., and Rodríguez, P., *Effects of Moisture Availability on Corrosion Kinetics of Steel in Concrete*, Corrosion, 49, pp.1004-1010, 1993

2004 **Gonzalez**, J.; Jalali, S.; Development of Negative Pore Water Pressure During Cement Hydration. (To be published).

1988 **Grigg**, N.S.: Infrastructure engineering and management, John Wiley and Sons, New York, 1988.

1977 **Gutt**, W.H.; Chemical resistance of concrete, Concrete, 12, 35, 1977

1990 **Hammer**, T.A. and Sellevold, E.J.: Frost resistance of high strength concrete, ACI SP-121, pp. 457-87, 1990.

1990 **Hansson**, C.M., Sørensen, B., *The threshold concentration of chloride in concrete for the initiation of reinforcement corrosion*. ASTM 1065, 1990. in Corrosion Rates of Steel in Concrete (Berke, N.S.; Chaker, V. et al, p 3-16)

1967 **Haussmann,** D. A.; Steel corrosion in concrete - how does it occur?, Material Protection, Vol. 6, Nov. pp 19-23 (1967).

2001 **Helland, S.**; Basis of design. Structural and service life design, a common approach. DuraNet - Third Workshop “Service Life Design of Concrete Structures – From Theory to Standardisation.”, Tromsø, pp16-23, 2001.

1988 **Henderson**. J., The Repair and Protection of Concrete Structures. Concrete Construction Exhibitions.
Leicestershire. U.K.. 1988, 8.

1993 **Henriksen, C.F.:** Prediction of service life and choice of repair strategy. Proceedings of the 6th Conference on Durability of Building Materials and Components, Japan.

1996 **HETEK 53**, Chloride penetration into concrete. State of the Art. Transport processes, corrosion initiation, test methods and prediction models, Report No. 53, 1996

1997 **HETEK 83**, – Theoretical background: A System for estimation of Chloride ingress into Concrete, Report No. 83, 1997

1981 **Higgins**, D. D., Diagnosing the cause of defects or deterioration in concrete structures. Concrete, British Standards Current Practice Sheet No. 69, 1981, 33.

1987 **Ho,** D.W.S., Lewis, R.K. Carbonation of concrete and its prediction. Cement Concr Res 1987; 17:489-504.

1995 **Hoogenboom**, R.; The relevance of durability research. Proceeding of the International workshop on new developments in Performance test methods, Hørsholm, Denmark, 10-11 April, pg.61-70

1987 **Hope**, B.B.; Alan, I.; Chloride corrosion threshold in concrete. ACI Mater J 1987;84:306-14.

1990 **Jaegermann** C. Effect of water-cement ratio and curing on chloride penetration into concrete exposed to Mediterranean sea climate. ACI Mater J 1990;87(4):333-9

1970 **Kalousek**, G. L. and Benton, E. J., Mechanism of Sea Water Attack on Cement Pastes. ACI Journal. 187. 364, 1970.

1980 **Klieger**, P.: Something for Nothing – Almost, Concrete International: Design & Construction, Vol. 2, No. 1, pp. 15-23, 1980.

1991 **Kobayashi** K, Suttoh K. Oxygen diffusivity of various cementitious materials. Cement Concr Res 1991;21:273-84.

1999 **Lahus**, O.: An analysis of the condition and condition development of concrete wharves in Norwegian fishing harbours, Dr.ing.Thesis 1999:23, Department of Building Materials, Norwegian University of Science and Technology, NTNU, Trondheim 1999 (in Norwegian).

1991 **Lambert, P.; Page, C.L.; Vassie, P.R.W.:** Investigations of reinforcement corrosion. 2. Electrochemical monitoring of steel in chloride-contaminated concrete. Materials and Structures, Vol. 24, No. 143, pp. 351-358.

1995 **Larsen, C.K.:** Effect of type of aggregate, temperature and drying/rewetting on chloride binding and pore solution composition. Paper presented at the RJLEM International Workshop on Chloride Penetration into Concrete, October 15-18, Saint-Remy-les-Chevreuse, France.

1998 **Larsson**, J., "The enrichment of chlorides in expressed concrete pore solution submerged in saline solution", Proceedings Nordic Seminar on Field Studies of Chloride Initiated Reinforcement

Corrosion in Concrete, Lund, Sweden, February 2-3, 1995.

2000 **Leira, B. J.**, Hynne, T., Lindgård, J., 'Marine Concrete Structures Subjected to Chlorid Attack: Probabilistic Lifetime Assessment', Proceedings of the ETCE/OMAE2000 Joint Conference: Energy for the New Millennium. American Society of Mechanical Engineers (ASME), New Orleans, USA, 2000.

2003 **Li**, C.Q.; Lawanwisut, W.; Serviceability assessment of reinforced concrete structures in marine environments. Proceedings PRO 29 RILEM, 2nd International Workshop on Life Prediction and Aging Management of Concrete Structures, Paris, France, p.127-136, 2003.

2001a **Life-365**: Service Life Prediction Model. Computer program for predicting the service life and Life-cycle costs of reinforced concrete exposed to chlorides. Silica Fume Association, 2001.

2001 **LIFE-365**: The Life-365 Model, *Proceedings of Nordic Mini Seminar* (Nilsson, L.O. (Ed.)), Publication P-01:6, Department of Building Materials, Chalmers University of Technology, Göteborg, 2001

2003 **LIFECON**; Models for the Prediction of the Residual Service Life. State of the Art Report. Deliverable D3.2, Project G1RD-CT-2000-00378, 2003.

1998 **Lindmark, S.** (1998), Mechanisms of Salt Frost Scaling of Portland Cement-bound Materials: Studies and Hypothesis, Report TVBM 1017, Division of Building Materials, Lund Institute of Technology, Lund, 1998.

1998 **Liu**, Y.; Weyers, R.E.; Modeling the time-to-corrosion cracking in chloride contaminated reinforced concrete structures. ACI Mat. Journal. 95 (6) (1998) 675-681.

1996 **LNEC E-378,** *Betões. Guia para a utilização de ligantes hidráulicos*. LNEC, Lisboa, 1996.

2004 **LNEC E-464,** *Betões. Metodologia prescritiva para a vida útil de projecto de 50 anos face às acções ambientais*. LNEC, Lisboa, 2004.

2004 **LNEC E-465,** *Betão. Metodologia para estimar as propriedades de desempenho do betão armado ou pré-esforçado que permitem satisfazer a vida útil de projecto sob as exposições ambientais XC ou XS*. LNEC, Lisboa, 2004.

1980 **Locke, C.E.; Siman, A.;** Electrochemistry of reinforcing steel in salt contaminated concrete. Material Strucutres, Vol. 24, Nº. 143, pp. 351-358, 1980.

2000 **Long, A.E.,** Henderson, G.D. & Montgomery, F.R. (2000), Why assess the properties of 309 near-surface concrete?

2001 **Lounis**, Z.; Mirza, M.S.; Reliability-based service life predictions of deteriorating concrete structures. Concrete Under Severe Conditions, Proceeding of Third International Conference, Vancouver, 2001, 965-972.

1996 **Luping**, T.; *Chloride transport in concrete - Measurement and Prediction*. Doctoral Thesis, Chalmers Tekniska Hogskola, pp. 88 + 12 papers, 1996.

1985 **Lukas, W.;** Relationship between chloride content in concrete and corrosion in untensioned reinforcement on Australian bridges and concrete roads surfings. Betongwerk und Fertigeil-Technik, Vol. 51, nº. 11, pp. 730-734, 1985.

1994a **Maage**, M.; Helland, S.; Carlsen, J.E.: Chloride penetration in high performance concrete exposed to marine environment. Proc of RILEM international Workshop on Durability of High Performance Concrete, Wien, February 14-15 1994, ed. by H. Sommer, pp. 194-207.

1995 **Maage**, M.; Poulsen, E.; Vennesland, O.; Carlsen, J.E.: Service life model for concrete structures exposed to marine environment - Initiation period. LIGHTCON Report No. 2.4, STF70 A94082 SINTEF, Trondheim, Norway.

1996 **Macias** A, Kindness A and Glasser F P. Corrosion behaviour of steel in high alumina cement mortar cured at 5, 25 and 55 °C: chemical and physical factors. Journal of Materials Science 31 (1996), 2279—89.

2000 **Malhorta**, V.M.*Durability of Concrete*, Corrosion Handbook Second Edition, R.W. Revie Ed., Wiley, 2000

1987 **Malhotra**, V.M., Paiter, K. and Bilodeau, A.: Mechanical properties and freezing and thawing resistance of high-strength concrete incorporating silica fume, Proceedings, CANMET – ACI International Workshop on Condensed Silica Fume in Concrete, Montréal, Canada, 25 p., 1987.

1994 **Mangat**, P.S., Molloy, B.T.: Predicting of long term chloride concentration in concrete. Materials and Structures, Vol. 27, pp. 338-346.

1995 **Mangat,** P.S.; Molloy, B.T.: Chloride binding in concrete containing PFA, gbs or silica fume under sea water exposure.

1980 **Manning**, D.G. : Where have all the bubles gone? Concrete International: Desigh & Construction, Vol. 2, No. 8, Aug. pp. 99-102, 1980.

2003 **Martín-Pérez**, B.; Lounis, Z.; Numerical Modelling of Service Life of Reinforced Concrete Structures. Proceedings PRO 29 RILEM, 2nd International Workshop on Life Prediction and Aging Management of Concrete Structures, Paris, France, p.71-79, 2003.

2001 **Martín-Pérez**, B.; Pantazopoulou, S.J.; Thomas, M.D.A.; Numerical solution of mass transport equations in concrete structures. Computers and Structures, 79(13), 2001, 1251-1264.

1999 **Martín-Pérez**, B.; Service Life Modelling of R.C. Highway Structures exposed to chlorides. PhD Dissertation, University of Toronto, 1999.

1981 **Maslehuddin** M. Optimization of Concrete Mix Design for Durability in the Eastern Province of Saudi Arabia. M.Sc. Thesis, Department of Civil Engineering, King Fahd University of Petroleum and Minerals, Dhahran; 1981.

1987 **Masters**. L. W. and Brandt, E., Prediction of Service Life of Building Materials and Components. CIBW80/RILEM 71-PSL Final Report. 1987. 20.

1999 **McGee**, R., *Modelling of durability performance of Tasmanian bridges*, Proceedings of the 8th International Conference on the Application of Statistics and Probability, Sydney, 1999.

1996 **Mejlbro**, L.; The complete solution to Fick's second law of diffusion with time dependant diffusion coefficient and surface concentration. Proceeding of CEMENTA's workshop on Durability of Concrete inSaline Environments. Danderyd, Sweden

1999 **Melchers,** R.E.; *Structural reliability: Analysis and prediction*. John Wiley and Sons Ltd., 1999

1975 **Mehta**, P. K., Effect of cement composition on corrosion of steel in concrete. Proc. Symp. Corrosion of Steel in Concrete. ASTM, 1975.

1980 **Mehta**, P. K.. Durability of Concrete in Marine Environment — A Review, ACI Publication SP-65. 1980.

1994 **Möller,** J.S. (1994), Measurement of carbonation in cement based material, Publication P-93:11, Department of Building Materials, Chalmers University of Technology, Göteborg, 1994.

1947 **Mielenz** et al.: Cement-aggregate reaction in concrete, Journal of American Concrete Institute, Detroit, Vol. 19, No. 2, pp. 93-129, 1947.

1990 **Morinaga**, S.; Prediction of service lives of reinforced concrete buildings based on the corrosion rate of reinforcing steel. Proceedings of Building Materials and Components, Brighton, UK, 7-9 November 1990:5-16.

1965 **Mozer**, J.D.; et al.; Corrosion of reinforcing bars in concrete. J Am Concr Inst 1965: 909-31.

2001 **Munch-Petersen**, C., Riv ned, byg op eller reparer – den optimale strategi (Demolish, construct or repair – the optimal strategy), Presented at Dansk Brodag Mars 2001, Taastrup, 2001.

1995 **Naess, A.;** *Structural reliability*. Lecture Notes Fag 37078, NTNU, Norway 1995.

1995 **Naess**, A.; *Structural reliability*. Lecture Notes Fag 37078, NTNU, Norway 1995.

1995 **Nagataki,** S-; Otsuki, N.; Wee, T.H.; Nakashita, K.: Condensation of chloride ion in hardened cement matrix materials and on embedded steel bars. AC1 Materials Journal, July-August, pp. 323-332.

1997 **Neville**, A.M.; Brooks, J.J.; Concrete Technology, Longman, 438p, (1997).

1995 **Neville,** A.M.; Properties of concrete. Longman, England, 844 pp. 1995.

1995 **Neville**, A.; *Chloride attack of reinforced concrete: an overview* Materials and Structures, **28**, 63-70 (1995)

1990 **Nilsson**, L.; Repairs of the Øland Bridge, experience and results of the first six columns carried out in 1990, Report, NCC, Malmø, Sweden, 1991 (In Swedish).

1998 **Nilsson,** L.; Gehlen,C.; Extension of resistivity model. Oral presentation at Duracrete Steering

Committee Meeting, Madrid, 1998

1997 **Nilsson,** L.; P. Sandberg, E. Poulsen, et al. From HETEK, A system for estimation of chloride ingress into concrete, Theoretical background, 1997.

1998 **Nilsson**, L., Ngo, M.H. and Gjørv, O.E., 'High-Performance Repair Materials for Concrete Structures in the Port of Gothenburg', Proceedings, Second International Conference on Concrete Under Severe Conditions – Environment and Loading, ed. by O.E. Gjørv, K. Sakai and N. Banthia, E & FN Spon, London and New York, 1998, pp. 1193-1198.

2000 **Nilsson, L.-O.,** Andersen, A., Luping, T. & Utgennant, P. (2000), Chloride ingress data from field exposure in a Swedish road environment, Publication P-00:5, Department of Buildings Materials, Chalmers University of Technology, Göteborg, 2000.

1999 **Nordtest 492**, *Concrete, Mortar and Cement Based Repair Materials: Chloride Migration Coefficient from Non-steady State Migration Experiments*, NT BUILD 492, Finland, 1999.

1999 **Norwegian Standard-3490**, "Design of structures – Requirements to reliability", Oslo, 1999.

2002 **NP ENV 13670-1**, Execution of concrete structures - Part 1: Common Rules; CEN 2002.

2003 **NS 3420**, *Beskrivelsestekster for bygg, anlegg, innstallasjoner*. Norsk Standard, 2003.

2003 **NS 3465,** Execution of concrete structures – Common rules. Norsk Standard, 2003.

2003 **NS 3473**, Design of concrete strucutres – Design and detailing rules. Norsk Standard, 2003.

1999 **NS 3490,** Basis of structural design, Requirements to reliability. Norsk Standard, 1999.

1995 **NT BUILD 443**, Concrete, hardened: Accelerated chloride penetration. NordTest, VTT, Finland, 1995.

1999 **NTNU** - Norwegian University of Science and Technology, NTNU, Department of Building Materials (unpublished data).

1981 **Okada**, E., Hisaka, M., Kazama, Y. and Hattori, K.: Freeze-thaw resistance of superplasticized concretes, Developments in the Use of Superplasticizers, ACI SP-68, pp. 269-82, 1981.

1992 **Okkenhaug**, K. and Gjørv, O.E.: Effect of delayed addition of air-entraining admixtures to concrete, Concrete International, Vol. 14, No. 10, pp. 37-41, 1992.

2000 **Okochi, H.,** Kameda, H., Hasegawa, S.-I., Saito, N., Kubota, K. & Igawa, M. (2000), Deterioration of concrete structures by acid deposition – an assessment of the role of rainwater on deterioration by laboratory and field exposure experiments using mortar specimens, Atmospheric Environment, No. 34, pp. 2937-2945, 2000.

1998 **Park**, S.K., Miller, K.W., *Minimal random number generator*, Communications of the ACM, vol31, pp1192-1201, 1998.

1987 **Parrott** L J. A review of carbonation in reinforced concrete. BRE/Cement & Concrete Association Report. Slough, C&CA, 1987.

1992 **Pettersson**, K., "Corrosion threshold value and corrosion rate in reinforced concrete", CBI Report 2:92, Swedish Cement and Concrete Research Institute, Stockholm 1992, pp 19-20

1992 **Pettersson**, K.,"Corrosion of Steel in High Performance Concrete", Proceedings 3rd Int. Symp. on Utilization of High Strength Concrete, Lillehammer Norway 1993.

1993 **Pettersson, K.:** Chloride threshold value and corrosion rate in reinforced concrete. Proc. of conference "Concrete 2000", (ed. R.K. Dhir & M.R. Jones), E&FN Spon, London, UK. Vol. 1, pp. 461-471.

1992 **Pettersson, K.:** Corrosion threshold value and corrosion rate in reinforced concrete. Swedish Cement and Concrete Research Institute, Stockholm, CBI Report 2:92, 43p.

1966 **Porbaix**, M., Atlas of electrochemical equilibria in aqueous solutions. Pergamon Press, New York 1966.

1993 **Poulsen**, E.: On a model of chloride ingress into concrete having time dependent diffusion coefficient. Proceedings of the Nordic Miniseminar in Gothenburg. Chalmers University of Technology. Gothenburg, Sweden. Report P-93:l.

1999 **prEN 10080-1**, Steel for the reinforcement of concrete - Weldable reinforcing steel – Part 1 – General requirements, CEN, Brussels, 1999.

1999 **prEN 10138-1**, Prestressing steels - Part 1: General requirements, CEN, Brussels, 1999.

2001 **prEN 1990**, Eurocode: Basis of structural Design—Stage 34, April2001, CEN, Brussels, 2001.

2002 **prEN 1992-1-1**, Eurocode 2: Design of concrete structures - Part 1: General rules and rules for buildings. CEN, Brussels, 2002.

2001 **Pruckner**, F., *Corrosion and protection of reinforcement in concrete measurements and interpretation*, University of Vienna, Faculty of Natural Sciences and Mathematics, Doctoral Thesis, p.232, 2001.

1994 **Purvis**, R.L.; Babaei, P.E.K.; Clear, K.C.J Markow, M J.: Life-cycle cost analysis for protection and rehabilitation of concrete bridges relaive to reinforcement corrosion. Strategic Highway Research Program, National Research Council. Washington DC, USA. Report SHRP-S-377, pp.79-82.

1995 **Petersson**, P.-E.: The salt frost durability of concrete – field tests, SP Report 1995: 73, Swedish National Testing and Research Institute, Borås, 64 p., 1995 (in Swedish).

1984 **Petersson**, P.E.: Effect of salt containing environment on frost resistance of concrete, Technical Report SP-RAPP 1984: 34 ISSN 0280-2503. National Testing Institute, Borås, Sweden, 1984 (in Swedish).

1955 **Powers**, T. C. and Steinour, H. H., An interpretation of some published research on alkali aggregate reaction. Part I. Chemical reactions and mechanisms of expansion, ACI Journal, 497. 1955.

1987 **Pullar-Strecker**, P.; Corrosion damaged concrete: assessment and repair. London: Butterworths; 1987.

1985 **Rasheeduzzafar**, Dakhil FH, Al-Gahtani AS. Corrosion of reinforcement in concrete structures in the Middle East. Concr Int: Design Construct 1985:48-55.

1989 **Rasheeduzzafar**, Al-Gahtani AS, Al-Saadoun SS. Influence of construction practices on concrete durability. ACI Mater J 1989;86(6):566-75.

1990 **Rasheeduzzafar**, Dakhil FH, Al-Gahtani AS, Al-Saadoun SS, Bader MA. Influence of cement composition on the corrosion of reinforcement and sulfate resistance of concrete. ACI Mater J 1990;87(2):1 14-22.

1986 **Rasheeduzzafar**, Dakhil FH, Bader MA. Toward solving the concrete deterioration problem in the Arabian Gulf region. The Arab J Sci Eng 1986;11(2):129–46.

1992 **Rasheeduzzafar**. Influence of cement composition on concrete durability. ACI Mater J 1992;89(6):574-86.

1996 **Raupach**, M.; *Investigations on the influence of oxygen on corrosion of steel in concrete - Part I,* Materials and Structures, **29**, 174-184 (1996)

1978 **Regourd**, M., Hosnairn, H., Morlureaux, B. Bisseny, P. and Evers. G., Ettringite and Thausmasite in
Mortar of the Offshore Dike in the Chebourg Port. No. 358, 1978.

1996 **Rodriguez, J.; Ortega, L.M.; et al;** Assessing structural conditions of concrete structures with corroded reinforcement. 4th International Congress on Concrete in Service of Mankind, Dundee, UK, 1996.

1994 **Rostam**, S.; Schiessl, P.; Service Life Design in Practice - Today and Tomorrow. Proceedings of the International Conference "Concrete Across Boarders", Odense Denmark 1994.

2001 **Rostam**, S.; Service Life Design of Concrete Structures - The potentials of the DuraCrete Methodology. DuraNet - Third Workshop Service Life Design of Concrete Structures. Tromsø 2001, 48-68.

1981 **Rubinstein**, R.Y.; *Simulation and monte carlo method.* John Wiley & Sons, New York, 1981.

1993 **Saetta**, A.V., Scotta, R.V., Vitaliani, R.V., *Analysis of Chloride Diffusion into Partially Saturated Concrete*, ACI Materials Journal, 90, pp.441-451, 1993.

1993 **Sandberg**, P., Larsson, J., "Chloride binding in cement pastes in equilibrium with synthetic pore solutions", Proceedings Nordic Seminar on Chloride Initiated Reinforcement Corrosion in Concrete, Gothenburg, Sweden, January 13-14, 1993.

1995 **Sandberg**, P., Pore solution chemistry in concrete", Proceedings Nordic Seminar on Field Studies of Chloride Initiated Reinforcement Corrosion in Concrete, Lund, Sweden, February 2-3, 1995

1995 **Sandberg**, P.: Critical evaluation of factors affecting chloride initiated reinforcement corrosion in concrete. Division of Building Materials, Lund Institute of Technology, Sweden. Report TVBM-7088, pp.24-25.

1993 **Sandvik**, M. and Wick, S.O.: Chloride penetration into concrete platforms in the North Sea, Proceedings, Workshop on Chloride Penetration into Concrete Structures. Department of Building Materials, Chalmers University of Technolog, Gothenburg, ed. by L.O. Nilsson, 7 p., 1993.

1994 **Sandvik**, M., Haug, A.K. and Erlien O.: Chloride permeability of high-strength concrete platforms in the North Sea, ACI SP-145, ed. V.M. Malhtora Detroit, pp. 121-130, 1994.

1998 **Schiessl**, P. (editor): Corrosion of steel in concrete, report of the technical committee 60-CSC R1LEM. Chapman and Hall. London, UK.

1998 **Schiessl**, P.; Corrosion of Steel in Concrete. Report of the TC60-CSC RILEM, Chapman and Hall London, 1988.

1990 **Schiessl,** P.; Raupach, M.; The influence of concrete composition and microclimate on the critical chloride content in concrete. In Corrosion of Reinforcement in Concrete, edited by C.L. Page, K.W.J. Threadaway and P.B. Bamforth (Elsevier Applied Science, London), pp. 49-58.

1975 **Schiessl** P.; Admissible crack width in reinforced concrete structures. Contribution II, 3-17. International Colloquium on the Behavior in Service of Concrete Structures; 1975

1979 **Schrage**, L., *ACM Transaction on Mathematical Software*, Vol. 5, n.2, pp.132-138, 1979

1995 **Sergi**, G., Yu, S.W., Page, C.L, "Diffusion of chloride and hydroxyl ions in cementitious materials exposed to a saline environment", Magazine of Concrete Research, 44, No. 158, pp. 63-69, March 1992

1990 **Sharobim, K.G.** (1990). Steel corrosion and its prevention in reinforced concrete, Doctor Thesis, Department of Civil Engineering, Hiroshima University, Hiroshima, 1990.

1980 **Slater**. S. E.. Corrosion of Metals in Association with Concrete, ASTM STP818, 1980

1990 **Sorensen**, B., "The corrosion properties of stainless steel in concrete", Proceedings 3rd Int. Symposium on Corrosion of Reinforcement in Concrete, May 21-24, 1990, pp. 601-610.

1994 **Stoltzner**, E. and Sørensen, B.: Investigation of chloride penetration into the Farø Bridges, Dansk Beton, Vol. 11, No. 1, pp. 16-18, 1994 (In Danish).

1975 **Stratful,** R.F.; Jurkovich, W.J.; Spellman, D.I.; Corrosion testing of bridge decks. Transportation research record nº539, pp50-59.

1995 **Swamy**, R.N.; Hamada, H.; Fukute, T.; Tanikawa, S.; Laiw, J.C.: Chloride penetration into concrete incorporating mineral admixtures or protected with surface coating material under chloride environments. Proc of CONSEC 95, Sapporo, Japan, E& FN Spon, London, UK

1988 **Takewaka**, K. and Mastumoto, S., *Quality and Cover Thickness of Concrete Based on the Estimation of Chloride Penetration in Marine Environments*, ACI SP 109-17, American Concrete Institute, 1988, pp. 381-400.

1996 **Tang**, L. Chloride transport in concrete – Measurment and prediction. Department Building Mateirals, Chalmers University of Technology. Pub-96.6

1993 **Tang, L**.; Nilsson, L.O.: Chloride binding capacity and binding isotherms of OPCpastes and mortars. Cement and Concrete Research, Vol. 23, No. 2, pp.347-353.

1996 **Tang, L**.; Nilsson, L.O.; Service life prediction for concrete structures under seawater by a numerical approach. in Durability of Building Materials and Components 7, Proc. 7th International Conf. on Durability of building materials and components, Stockholm, pp 97 - 106 (1996).

1997 **Taylor,** H.F.W. (1997), Cement Chemistry, Thomas Telford Services, Second edition, London, 1997.

1979 **Tepfers**, R., Cracking of concrete cover along anchored deformed reinforcing bars, Magazine of Concrete Research. 31 (106) (1979) 3-12.

1982 **Thoft-Christensen**, P. & Baker, M. J. (1982), Structural Reliability Theory and Its Applications, Springer Verlag, Berlin, 1982.

1996 **Thomas**, M. D. A., Chloride thresholds in marine concrete, *Cement and Concrete Research*, Vol. 26, No. 4, April, 1996, pp. 513-519.

1990 **Thomas,** M.D.A.; Matthews, J.D.; Haynes, C.A.: Chloride diffusion and reinforcement corrosion in marine exposed concrete containing pulverised fuel ash. In Corrosion of Reinforcement in Concrete, ed. C.L. Page, K.W.J. Treadaway & P.B. Bamforth (Elsevier Applied Science, London, UK), pp. 198-212.

1970 **Timoshenko**, S.P. and Goodier, J.N., Theory of Elasticity. McGraw-Hill Book Company, New York, 1970.

1982 **Tuutti**, K., "Corrosion of steel in concrete", CBI Research, fo 4:82, Swedish Cement and Concrete Research Institute, Stockholm 1982.

1993 **Tuutti, K.:** Effect of cement type and different additions on service life. Proc. Conference "Concrete 2000", ed. R.K. Dhir & M.R. Jones, E&FN Spon, London, UK, Vol. 2, pp. 1285-1295.

1982 **Tuutti, K.;** Corrosion of Steel in Concrete. Swedish Cement and Concrete Research Institute, Stockholm. Report No. CBI Research FO 4:82. 1982b Tuutti, K.: Service life of structures with regard to corrosion of embedded steel. Performance of Concrete in Marine Environment, pp. 223-236.

1999 **Tuutti, K.;** Repair philosophy for concrete structures, pp 159-169 in Concrete durability and repair technology. Proc. Int. Conf. *Creating with Concrete.* Dundee, 1999

1980 **TWRL** - Concrete in the Oceans, Tech. Rep. No. 5, Taylor Woodrow Research Labs, Cement and Concrete Association. Wexham Springs. U.K., 1980. 67.

1960 **Tyler** IL. Long-time study of cement performance in concrete, chapter 12, concrete exposed to seawater and fresh water. J Am Concr Inst Proc 1960;57:825-36.

1986 **Ugural**, A.C., Advanced Strength and Applied Elasticity. Elsevier Applied Science, London, 1986.

1971 **Uhlig,** H.H. Corrosion and corrosion control——an introduction to corrosion science and engineering. New York: John Wiley & Sons, Inc.; 1971.

1983 **Uhlig,** H.H.; Corrosion and corrosion control. New York: John Wiley and Sons; 1983.

1990 **Uji**, K.; Matsuoka, V.; Maruya, T.: Formulation of an equation for surface chloride content due to permeation of chloride. Proceedings of the Third International symposium on "Corrosion of Reinforcement in Concrete Construction". Elsevier Applied Science. London, UK. pp.258-267.

1984 **Vassie**, P.; Reinforcement corrosion and the durability of concrete bridges. Proceeding Lnstn Civil Engineers, Part 1. 76, 713, 1984.

1968 **Verbeck** GJ. Field and laboratory studies of the sulphate resistance of concrete. In: Performance of concrete. University of Toronto Press; 1968. p. 113.

1975 **Verbeck** G. J., Mechanism of corrosion in concrete. Corrosion of Metals in Concrete. ACI SP-49, 1975. 21.

1993 **Vidarsson**, H., Personal communication, Bergsoe Ami Corrosion, Landskrona, Durability of Marine Concrete Structures, Danish Concrete Institute, Aalborg Portland, Cementa, Denmark - Sweden 1993, pp. 149-173.

1978 **Vivian**, H. E.. Alkali Aggregate Reaction, Division of Building Res. CS1RO Rep. Australia. 1978, 1.

1993 **Wang**, X.M.; Zhao, H.Y.; The residual service life prediction of R.C. structures. Durability of building materials and components 6. E & FN Spon; 1993. p. 1107-14.

1985 **West,** R.E.; Hime, W.G.: Chloride profiles in salty concrete. Materials Performance, July.

1917 **Wig**, R.J. and Ferguson, L.R.: What is the trouble with concrete in seawater? Engineering News Record, Vol. 79, pp. 532, 641, 689, 737 and 794, 1917.

1993 **Wilson**, K R and Wallbank, E J, 'Design for durability', Concrete 2000 – Economic and durable concrete through excellence, Vol 1, Ed. R K Dhir and M R Jones, E & F N Spon, 1993. pp. 85-97.

1997 **Wood**, J.G.M. and Crerar, J.: Tay Road Bridge: Analysis of chloride ingress variability and prediction of long term deteroration, Construction and Building Materials, Vol. 11, No. 4, pp. 249-254, 1997.

1988 **Yonezawa**, T., Pore Solution Comnositinn and Chloride-Induced Cnrrnsinn nf Steel in Concrete. Brtish Ph D Thesis, Victoria University of Manchester Corrosion and Protection Centre, 1988

10. APPENDIXES

APPENDIX 1
PROBABILISTIC BACKGROUND FOR MODEL IMPLEMENTATION

A.1 Introduction

Most observable phenomena in the world contain a certain amount of uncertainty. That is, they cannot be predicted with certainty. In general, repeated measurements of physical phenomena generate multiple outcomes. Among these multiple outcomes, some are more frequent than others. The occurrence of multiple outcomes without any pattern is described by terms such as *uncertainty, randomness,* and *stochasticity.* The word *stochasticity* comes from the Greek word *stochos,* meaning uncertain. For example, if several identical specimens of steel reinforcement were loaded until failure in a laboratory, each specimen would fail at a different value of the load. The load bearing capacity of the bar is therefore a random quantity and known as a random variable. In general, all the parameters of interest in engineering analysis and design have some degree of uncertainty and thus may be considered to be random variables.

In view of the uncertainties in engineering analysis and design, satisfactory performance cannot be absolutely ensured. Instead, assurance can only be given in terms of the probability of success in satisfying some performance criterion. In engineering terminology, this probabilistic assurance of performance is referred to as *reliability*. The term reliability is commonly defined as the complement of the probability of failure, but more properly it is the probability of adequate performance of the system over a given period of time (Melchers 1999). *Probability* denotes the chance that a particular, predefined event occurs.

An alternative way to look at reliability is to consider unsatisfactory performance of the system. In that case, one might measure the probability of failure to satisfy some performance criterion, referred to as *risk.* Thus, risk and reliability are complementary terms. In some references, the term risk is not just the probability of failure but includes the consequence of failure. For example, if the cost of failure is to be included in risk assessment, then risk is defined as the product of the probability of failure and the cost of failure (Haldar 2000).

Reliability or risk assessment of engineering systems uses the methods of probability and statistics, between which a distinction needs to be drawn here. Statistics is the mathematical quantification of uncertainty (mean, standard deviation, etc., of a variable), whereas probability theory uses the information from statistics to determine the likelihood of specific events.

The analysis and design of most engineering systems utilize the basic concept that the capacity, resistance, or supply should at least satisfy the demand. No matter how supply and demand are modelled or described, most engineering problems must satisfy this concept. However, most of the parameters related to supply and demand are random quantities. The primary task of planning and design is to ensure satisfactory performance, that is, to ensure that the capacity or resistance is greater than demand during the system's useful life (Haldar 2000).

Engineers have always recognized the presence of uncertainty in the analysis and design of engineering systems. However, traditional approaches simplify the problem by considering the uncertain parameters to be deterministic and accounting for the uncertainties through the use of empirical safety factors. Safety factors are derived based on past experience but do not absolutely guarantee safety or satisfactory performance. Also, they do not provide any information on how the different parameters of the system influence safety. Therefore, it is difficult to design a system with a uniform distribution of safety levels among the different components using empirical safety factors.

Engineering design is usually a trade-off between maximizing safety levels and minimizing cost. A design methodology that accomplishes both of these goals is highly desirable. Deterministic safety factors do not provide adequate information to achieve optimal use of the available resources to maximize safety. On the other hand, probabilistic analysis does provide the required information for optimum design. While probabilistic analysis brings rationality to the consideration of uncertainty in design, it does not discount the experience or expertise gathered from a particular system. In fact, the probabilistic methodology includes a professional factor, which incorporates the expert opinions of experienced designers about different uncertain quantities in the system (Haldar 2000)].

A.2 Measures of reliability – Limit States

When an engineering system is loaded in some way it will respond in a manner which depends on the type and magnitude of the load and the resistance of the system. Whether the response is considered satisfactory depends on the requirements which must be satisfied. Such requirements might include safety of the system against collapse, limitations on damage, or on deflections or any of a range of other criteria. Each of these requirements may be termed a *limit state*. The violation of a limit state can then be defined as the attainment of an undesirable condition for the structure. Limit states are represented by the Limit State Equations, generally of the form $g(x) = 0$. There are two main categories of limit states according to the seriousness of the damage that occurs from the inability to fulfil the requirements (Melchers 1999). These are:

- Ultimate Limit State - the limit between the state where the system is able to carry loads acting on it and the state where the system has collapsed;
- Serviceability Limit State - the limit between the state where the performance of the system is acceptable and the state where the system is no longer serviceable.

The study of reliability is concerned with the calculation and prediction of the probability of limit state violation for engineered systems at any stage during their life. In particular, the study of safety is concerned with the violation of the ultimate or serviceability limit states for the system. The probability of occurrence of an event such as limit state violation is a numerical measure of the chance of its occurring (Melchers 1999). This measure may either be obtained from measurements of the long-term frequency of occurrence of the event for generally similar systems, or may be simply a subjective estimation of the numerical value. In practice it is not usually possible to observe for a sufficiently long period of time, and a combination of subjective estimation and frequency observations for system components and properties is used to predict the probability of limit state violation for the system as a whole (Naess 1995).

Generally, methods to measure the reliability of a structure can be divided into four groups (Madsen 1986):

- Level I methods: The uncertain parameters are modelled by one characteristic value, as for example in codes based on the partial coefficients concept;
- Level II methods: The uncertain parameters are modelled by the mean values and the standard deviations, and by the correlation coefficients between the stochastic variables. The stochastic variables are implicitly assumed to be normally distributed. The reliability index method is an example of a level II method;
- Level III methods: The uncertain quantities are modelled by their joint distribution functions. The probability of failure is estimated as a measure of the reliability;
- Level IV methods: In these methods the consequences (cost) of failure are also taken into account and the risk (consequence multiplied by the probability of failure) is used as a measure of the reliability. In this way different designs can be compared on an economic basis taking into account uncertainty, costs and benefits.

If the reliability methods are used in design, they have to be calibrated so that consistent reliability levels are obtained. Level I methods can, for example, be calibrated using level II methods, level II methods can be calibrated using level III methods, etc. Several techniques can be used to estimate the reliability for level II and III methods:

- Simulation techniques: Samples of the stochastic variables are generated and the relative number of samples corresponding to failure is used to estimate the probability of failure. The simulation techniques are different in the way the samples are generated;
- FORM techniques: In First Order Reliability Methods the limit state function is linearized and the reliability is estimated using level II or III methods;
- SORM techniques: In Second Order Reliability Methods a quadratic approximation to the failure function is determined and the probability of failure for the quadratic failure surface is estimated.

A.2.1 Factors affecting reliability

Identification of the uncertainties in reliability assessment for complex systems may be difficult. It may be advantageous to use a systematic scheme to help to enumerate all operational and environmental loading states and for each consider possible combinations of error or malfunction. This is essentially *event-tree analysis* (Henley 1981). Rather similarly, the systematic development of all possible forms of hazard to which a structure might be subjected has been termed *hazard scenario analysis* (Schneider 1981). More generally, techniques such as *brain storming* (Osborne 1957) may be of use. Various other techniques have also been suggested (Henley 1981). All amount essentially to a critical analysis of the problem to be analysed, consideration of all imaginable consequences and all imaginable possibilities and retaining only those with some finite probability of occurrence. Furthermore, all techniques rely on having available expert opinion for the various assessments to be made and up-to-date information on which to base assessments.*)*

Reliability analysis requires information about uncertainties in the system. Before collecting such uncertainty information and proceeding with the reliability analysis, the engineer needs to understand that there are different types of uncertainty in engineering. In a broad sense, uncertainties in a system may come from cognitive or qualitative sources and noncognitive or quantitative sources (Haldar 2000).

Noncognitive or quantitative sources of uncertainty or randomness can be classified into three types: physical, statistical and modelling uncertainties:

- Physical uncertainty is that identified with the inherent random nature of a basic variable. Physical uncertainty may be reduced but not eliminated with greater availability of data, or greater effort in quality control. It is a fundamental property of the basic variable. The physical uncertainty for any basic variable is generally not known a priori and must be estimated from observations of the variable or be subjectively assessed;

- Statistical estimators such as the sample mean and higher moments can he determined from available data and then used to suggest an appropriate probability density function and associated parameters. Generally the observations of the variable do not perfectly represent it and as a result there may be bias in the data as recorded;
- Modelling uncertainty is caused by the use of a simplified relationship between the basic variables to represent the real relationship or phenomenon of interest. In its simplest form, modelling uncertainty concerns the uncertainty of physical models, such as limit state equations.

Cognitive or qualitative sources of uncertainty relate to the vagueness of the problem arising from intellectual abstractions of reality. They may come from phenomenological, decision, prediction and uncertainties due to human factors:

- Phenomenological uncertainty may be considered to arise whenever the form of construction or the design technique generates uncertainty about any aspect of the possible behaviour of the structure under construction, service and extreme conditions. It is therefore of particular importance for novel projects, or those which attempt to extend the state of the art (Pugsley 1962). It should be quite clear that for such projects only subjective estimates of the effect of this type of uncertainty can be given;
- Decision uncertainty arises in connection with the decision as to whether a particular phenomenon has occurred. In terms of limit states it is concerned purely with the decision as to whether a limit state violation has occurred. Many problems in structural reliability assessment involve the prediction of some future state of affairs; in this case the prediction of the reliability of some structure at some time $t > 0$ in the future. An estimate of structural reliability depends on the state of knowledge available to the analysist. As new knowledge related to the structure becomes available, the estimate will become more refined, with, usually but not necessarily, a concomitant reduction in uncertainty.
- The uncertainties resulting from human involvement in the design, construction, use, etc., of structures may, for convenience, be considered in two categories: human errors and human intervention. Human errors can be divided, roughly, into errors

due to natural variation in task performance and gross errors. Gross error might be considered again in two categories: those errors which occur in the normal processes of design, documentation, construction and use of the structure within accepted procedures, and those which are a direct result of ignorance or oversight of fundamental structural or service requirements. Some forms of human intervention are institutionalised, e.g. design checking to obtain approval, and sanctions for violations of the law. Intervention may also be informal, such as may result from the observation that ‘something is wrong’. Human intervention strategies include measures such as education, complexity reduction and personnel selection. External measures such as self-checking, external checking and inspection and finally sanctions.

A.3 Probabilistic approach to Reliability Analysis

In general, engineering design consists of proportioning the elements of a system so that it satisfies various criteria of performance, safety, serviceability, and durability under various demands. The probability of occurrence of a limit state violation is a numerical measure of the chances of its occurrence, from long term observations or subjective estimates.

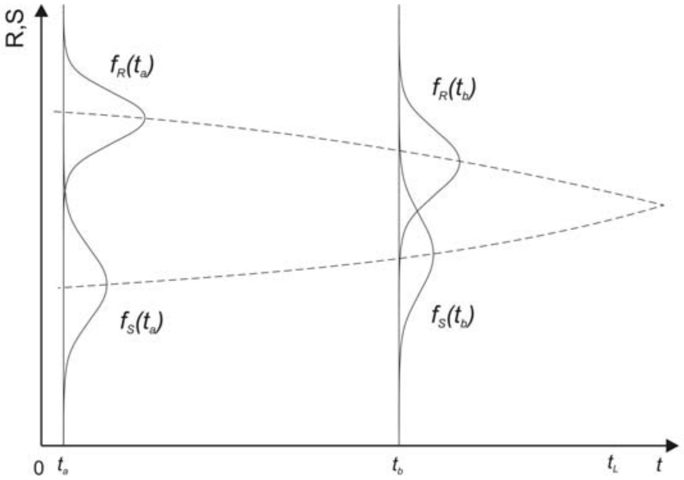


Figure A.1 Time-variant reliability problem (Melchers 1999).

Figure A.1 represents the general reliability problem, where the limit state function is violated if, at any time

$$R(t) - S(t) < 0 \quad \text{or} \quad \frac{R(t)}{S(t)} < 1 \tag{A.1}$$

The probability that it occurs is the probability of failure. It can be roughly represented by, but is not actually equal to the overlap of the probability density functions.

A.3.1 Basic Reliability Problem

The basic structural reliability problem considers only one load effect *S* resisted by one resistance *R*. Each of *S* and *R* are described by a known probability density function (PDF), $f_s(s)$ and $f_R(r)$ respectively. As noted, *S* may be obtained from the applied loading *Q* through a structural analysis (either deterministic or with random components).

For convenience, it is considered a *R* and a *S* are not function of time. If *R* and *S* have known PDF $f_R(r)$ and $f_S(s)$ respectively and expressed in the same units, then,

$$p_f = P(R - S \leq 0) \quad \text{or} \quad P[g(R,S) \leq 0] \tag{A.2}$$

where *g(...)* is termed the limit state function, and the probability of failure is identical with the probability of limit state violation. The probability of failure of the joint (bivariate) density f_{RS}(r,s) is given by

$$p_f = P(R - S \leq 0) = \iint_D f_{RS}(r,s)dr.ds \tag{A.3}$$

If *R* and *S* are independent, then

$$f_{RS}(r,s) = f_R(r).f_S(s) \tag{A.4}$$

then we obtain

$$p_f = P(R - S \leq 0) = \int_{-\infty}^{\infty}\int_{-\infty}^{s \geq r} f_R(r).f_S(s)dr.ds \tag{A.5}$$

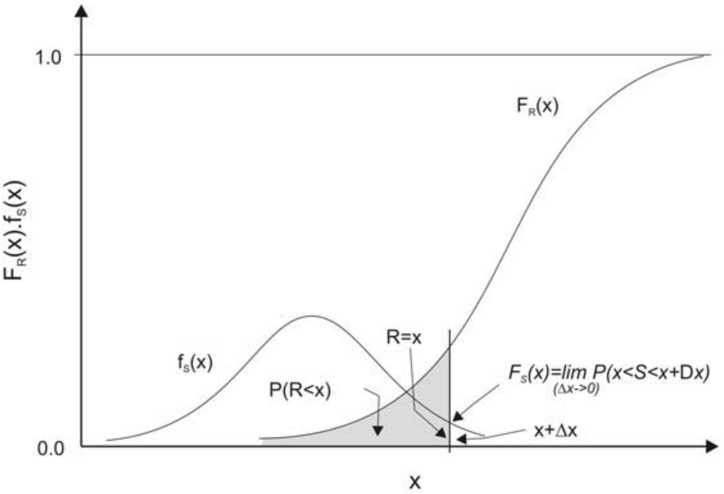


Figure A.2 Basic *R*-*S* problem: $F_R(\,).f_S(\,)$ representation (Naess 1995).

For a random variable *X*, the cumulative distribution function is given by

$$F_X(x) = P(X \leq x) = \int_{-\infty}^{x} f_X(y)dy \tag{A.6}$$

If R and S are independent, and if $x \leq y$, the solution of the probability of failure for bivariate density functions can de written in a single integral form

$$p_f = P(R - s \leq 0) = \int_{-\infty}^{\infty} F_R(x).f_S(x)dx \tag{A.7}$$

known as the convolution integral. $F_R(x)$ is the probability $R \leq x$ of the probability that the actual resistance R of the member is less than some value x, representing failure. The terms $f_S(x)$ represents the probability that the load effect S acting on the element has a value between x and $x + \Delta x$ in the limit as $\Delta x \rightarrow 0$. By calculating the integral over all x, the total failure probability is obtained. This can be seen in figure A.2.

For a few distributions of R and S it is possible to integrate the convolution integral analytically. If both R and S are normally random variables with means μ_R and μ_S and variances σ_R^2 and σ_S^2 respectively, the safety margin $Z = R - S$ then has the following mean and variance:

$$\mu_Z = \mu_R - \mu_S \tag{A.8}$$

or

$$\sigma_Z^2 = \sigma_R^2 + \sigma_S^2 \tag{A.9}$$

then (A.3) becomes

$$p_f = P(R - S \leq 0) = P(Z \leq 0) = \Phi\left(\frac{0 - \mu_Z}{\sigma_Z}\right) \tag{A.10}$$

where $\Phi(\ )$ is the standard normal distribution function, with expected value zero and unit standard deviation $(\mu_U = 0, \sigma_U = l)$. In figure A.2 the failure region is shown shaded.

Using (A.8) and (A.9) in (A.10), it follows that (Cornell 1969):

$$p_f = \Phi\left[\frac{-(\mu_R - \mu_S)}{\sqrt{(\sigma_S^2 + \sigma_R^2)}}\right] = \Phi(-\beta) \tag{A.11}$$

and

$$\beta = -\Phi^{-1}(p_f) \tag{A.12}$$

where $\beta = \mu_Z / \sigma_Z^2$ is defined as the safety or reliability index, as in figure A.5.

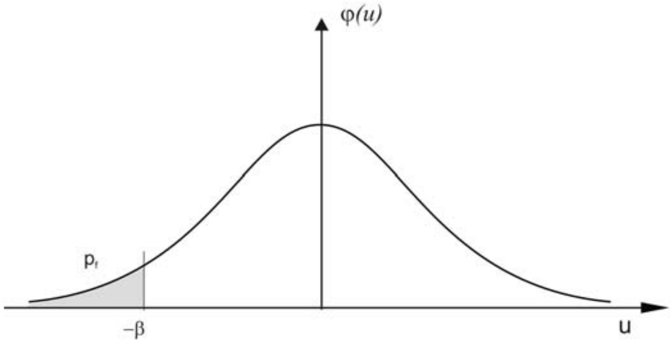


Figure A.5 Reliability index an probability of failure. φ is the standard normal density function (Cornell 1966).

If either σ_S or σ_R or both are increased, the term in square brackets in (A.11), will become smaller and hence p_f will increase, as might be expected. Similarly, if the difference between the mean of the load effect and the resistance is reduced, p_f increases. These observations may also be deduced from figure A.1, taking the amount of overlap of $f_R(\ )$ and $f_S(\ )$ as a rough indicator of p_f at any point in time.

A.3.2 Generalized limit state function

A.3.2.1 Basic variables

The fundamental variables which define and characterize the behaviour and safety of a structure may be termed the *basic variables*. They are usually the variables employed in conventional structural analysis and design. Typical examples are dimensions, densities or unit weights, materials, loads, material strengths. The compressive strength of concrete would be considered a basic variable even though it can be related to more fundamental variables such as cement content, water-to-cement ratio, aggregate size, grading and strength, etc. However, structural engineers do not normally use these latter variables in strength or safety calculations. It is often possible to choose the basic variables such that they are independent. However, this is not always the case. Thus the compressive and tensile strengths and the elastic modulus of concrete are related; yet in a particular analysis they might each be treated as a basic variable. Dependence between basic variables usually adds complication to a reliability analysis. It is important that the dependence structure between dependent variables be known and expressible in some form. This will usually be in the form of a correlation matrix (Melchers 1999).

The probability distributions for the basic variables can be obtained by direct inference from observed data, by subjective assessment or by some combination of these techniques. In practice some subjective influence is nearly always present since there arc seldom sufficient data available to identify unambiguously only one distribution as appropriate. All this assumes furthermore that past observations and experience for similar structures can be validly used for the structure under assessment.

Sometimes physical reasoning may be used to suggest an appropriate probability distribution. Thus, where the basic variable consists of the sum of many other variables (which are not explicitly considered), the central limit theorem can be invoked to suppose that a normal distribution is appropriate. This reasoning would be appropriate for the compressive strength of concrete (many component strengths) and for the dead load of a beam or slab (again many components of weight and several dimensions).

The parameters of the distribution may be estimated from the data using one of the usual methods, e.g. methods of moments, maximum likelihood, order statistics. However, it must be emphasized that such techniques should not be used blindly. Critical examination of the data for trends and outliers is always necessary and the reasons for these phenomena established. It is quite possible for such behaviour to be the result of data recording and storage procedures rather than the behaviour of the variable itself.

Finally, when model parameters have been selected, the model should be compared with the data if at all possible. A graphical plot on appropriate probability paper is often very revealing, but analytical *goodness of fit* tests can also be used (Melchers 1999).

It will not always be possible to describe each basic variable by an appropriate probability distribution. The required information may not be available. In such circumstances a point estimate of the value of the basic variable might be used, i.e. the best estimate given the known information. If some uncertainty information about the variable is also available, it might be appropriate to represent it by an estimate of its mean and its variance only. This is then known as a second moment representation. One way in which such a representation might be interpreted is that in the absence of more precise data, the variable might be assumed to have a normal distribution (as this is completely described by the mean and variance, i.e. the first two moments). However, other probability distributions might be more appropriate, even if only the first two moments are known. When more than second-moment, but less than full distribution information is available, (e.g. some correlation data) the incorporation of this in the reliability analysis becomes rather more difficult. Much of the difficulty can be avoided, however, by choosing independent basic variables (Melchers 1999).

A.3.2.1 Generalized limit state equations

The probability distributions for the basic variables can be obtained by direct inference from observed data, by subjective assessment or by some combination of these techniques.

Furthermore, it is assumed that it is possible to give a mathematical formulation of this failure mode. An important step in a reliability analysis is to decide which quantities should be modelled by stochastic variables and which should be modelled by deterministic parameters.

The basic variables $\overline{X}$ are stochastic variables. The realisations of the basic variables are denoted $\overline{x} = \left(x_1, x_2, ..., x_n\right)$ i.e. $\overline{x}$ is a point in the *n*-dimensional basic variable space. The joint density function for the stochastic variables $\overline{X}$ is $f_{\overline{X}}\left(\overline{x}\right)$. The elements in the vector of expected values and the covariance vector are

$$\mu_i = E\left[\overline{X}_i\right],\ i = 1,2,...,n \tag{A.13}$$

$$C_{ij} = Cov\left\lfloor X_i, X_j \right\rfloor,\ i,j = 1,2,...,n \tag{A.14}$$

The standard deviation of X_i is σ_i. The variance of X_i is $\sigma_\mathrm{i}^2 = \mathrm{C_{ij}}$. The coefficient of correlation between is X_i and X_j.

$$\rho_{ij} = \frac{C_{ij}}{\sigma_i \sigma_j},\ i,j = 1,2,...,n \tag{A.15}$$

where $-1 \leq \rho_{ij} \leq 1$.

Application of FORM, SORM and simulation methods requires that it is possible for given realisations *i* of the basic variables to state whether the structure (or component/failure mode) is in a safe state or in a failure state. The basic variable space is thus divided into two sets, the safe set ω_S and the failure set ω_F. The two sets are separated by the failure surface (limit state surface). It is assumed that the failure surface can be described by the equation

$$g\left(\overline{x}\right) = g\left(x_1, ..., x_n\right) = 0 \tag{A.16}$$

where *g(x)* is denoted the failure function.

Being $\overline{X}$ the vector of basic variables, then the resistance *R* could be expressed as $R=g_R(X)$ and the loading as $S = g_S\ (X)$. The functions g_R and g_S may be non-linear, and in general the cumulative density function, CDF, F_R, can be obtained by multiple integration over the relevant basic variables

$$F_R(r) = \int_{g_R(x)\leq r} \cdots \int f_X(x) dx \tag{A.17}$$

It follows that, with the limit state function expressed as *g(x)*, the generalization of (A.3)

$$p_f = P[g(x) \leq 0] = \int\limits_{g_R(x) \leq r} \dots \int f_X(x) dx \tag{A.18}$$

Here $f_x(x)$ is the joint probability density function for the n vector of basic variables. Note that the resistance R and the load effect S need no longer be independent, as each is now described in terms of the basic variables X.

It is important to note that the failure surface does not define a unique failure function, i.e. the failure surface can be described by a number of equivalent failure functions. However, whenever possible differentiable failure functions should be used. In structural reliability the failure function usually results from a mechanical analysis of the structure (Naess 1995).

The region of integration of $g(x) \leq 0$ in expression (A.18) denotes the space of limit state violation and is directly analogous to that for simple R-S limit state. Except for some special cases, the integration over the failure domain $g(x) \leq 0$ cannot be performed analytically. Methods to deal with the integration are essentially of two types (Annis 2003):

- multidimensional integration of the original problem using numerical approximations such as simulation;
- transformation of the original problem such that the probability density function for each variable is approximated by a normal distribution. The (multi)normal distribution has some remarkable characteristics which may then be used to determine, approximately, the probability of failure.

A.3.3 The indicator function

The limit state violation problem (A.18) can be further extended if the limit state function is generalized. For convenience, the use of the indicator function I defined in such a way that:

$$g(x) = \begin{cases} > 0 \Rightarrow \bar{x} \in \omega_S \Rightarrow I\left[g(x)\right] = 1 \\ \leq 0 \Rightarrow \bar{x} \in \omega_F \Rightarrow I\left[g(x)\right] = 0 \end{cases} \tag{A.19}$$

Usually the failure function is defined such that positive values of $g(x)$ correspond to safe states and negative values correspond to failure states, see figure A.3. It follows that $I[g(x)]$ may then be interpreted as a utility function with the failure state $g(x) \leq 0$ denoting a utility of zero, and the safe state $g(x) \leq 0$ having unit utility.

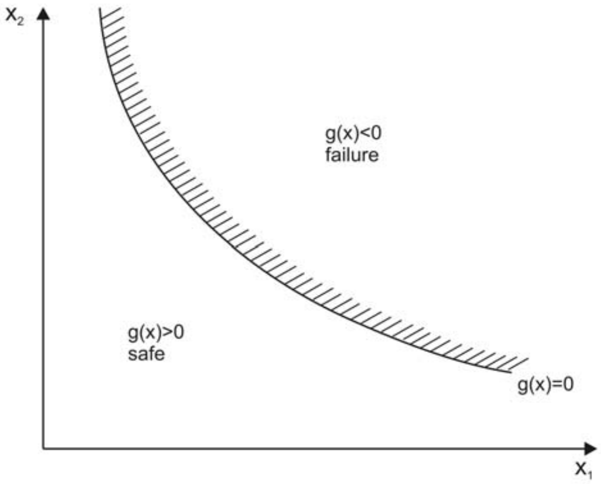


Figure A.3 Failure function *g(x)*.

In practice, and in particular in questions of lack of serviceability, the distinction between full utility and zero utility may not always be clear cut, and values between zero and unity may be appropriate, see figure A.4.

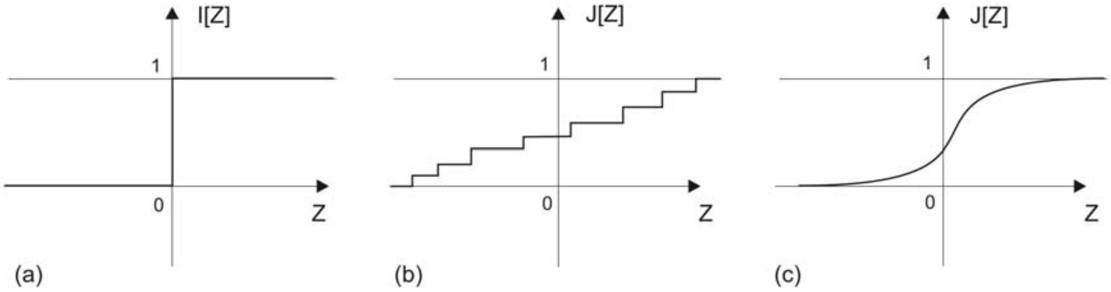


Fig A.4 Limit state violation indicators (3.3).

It may be that utility depends inversely on, for example, concrete crack size: with no cracks having a utility of 1, cracks < 10^{-3} mm a utility of 0.5 and greater cracks zero utility. Clearly many other possibilities exist. If now *J* denotes this more general interpretation of the indicator function (see figure A.4(b)), and if $J^C = 1 - J$ defines the complement of *J,* the generalization of (A.18) becomes

$$p_f = P\left(J^C\left[g\left(X\right)\right]\right) = \int_x \cdots \int J^C\left[g\left(X\right)\right] f_X\left(x\right) dx \tag{A.20}$$

As might be imagined, evaluation of (A.20) is not a simple matter. In addition, the function *J[g(x)]* must be known. Some initial work to establish *J|g(x)|* has been reported (Reid 1980).

A.4 Simulation Techniques

In the simplest form of the basic simulation, each random variable in a problem is sampled several times to represent its real distribution according to its probabilistic characteristics. Considering each realization of all the random variables in the problem produces a set of numbers that indicates one realization of the problem itself. Solving the problem deterministically for each realization is known as a simulation cycle, trial, or run. Using many simulation cycles gives the overall probabilistic characteristics of the problem, particularly when the number of cycles N tends to infinity. The simulation technique using a computer is an inexpensive way (compared to laboratory testing) to study the uncertainty in the problem.

The availability of personal computers and software makes the process simple. In fact, to evaluate the accuracy of these sophisticated techniques or to verify a new technique, simulation is routinely used to independently evaluate the underlying probability of failure (Naess 1995).

A.4.1 Monte Carlo simulation technique

The method commonly used for this purpose is called the Monte Carlo simulation technique. The name itself has no significance, except that it was used first by von Neumann during World War II as a code word for nuclear weapons work at the Los Alamos National Laboratory in New Mexico (CSEP 1995). Most commonly the name Monte Carlo is associated with a place where gamblers take risks. This technique has evolved as a very powerful tool for engineers with only a basic working knowledge of probability and statistics for evaluating the risk or reliability of complicated engineering systems. In many applications of Monte Carlo, the physical process is simulated directly, and there is no need to even write down the differential equations that describe the behaviour of the system. The only requirement is that the physical (or mathematical) system be described by probability density functions (PDF's). The Monte Carlo simulation technique has seven essential elements (CSEP 1995):

(1) *Problem definition:* defining the problem in terms of all the random variables;
(2) *Probability density functions:* quantifying the probabilistic characteristics of all the random variables in terms of their PDFs or PMFs and the corresponding parameters;
(3) *Random Number Generators:* generating the values of these random variables;
(4) *Sampling Rules:* evaluating the problem deterministically for each set of realizations of all the random variables, that is, numerical experimentation;

(5) *Scoring:* extracting probabilistic information from N such realizations;
(6) *Error estimation:* determining the accuracy and efficiency of the simulation;
(7) *Variance reduction techniques:* methods for reducing the variance in the estimated solution to reduce the computational time for Monte Carlo Simulation.

Initially, all the random variables are considered to be uncorrelated.

Monte Carlo simulation techniques involve *sampling* at *random* to simulate artificially a large number of experiments and to observe the result. In the case of analysis for structural reliability, this means, in the simplest approach, sampling each random variable X_i, randomly to give a sample value $\hat{x}_i$. The limit state function $G(\hat{x}) = 0$ is then checked. If the limit state is violated (i.e. $G(\hat{x}) < 0$), the structure or structural element has *failed*. The experiment is repeated many times, each time with a randomly chosen vector $\overline{x}$ of $\hat{x}_i$ values. If N trials are conducted, the probability of failure is given approximately by

$$p_f = \frac{n\left(G\left(\hat{x}_i\right) \leq 0\right)}{N} \tag{A.21}$$

where $n(G(\hat{x}) \leq 0)$ denotes the number of trials n for which $(G(\hat{x}) \leq 0)$. Obviously the number N of trials required is related to the desired accuracy for p_f.

It is clear that in the Monte Carlo method a game of chance is constructed from known probabilistic properties in order to solve the problem many times over and from that to deduce the required result, i.e. the failure probability.

A.4.2 Generation of random numbers

A necessary tool in simulation techniques for estimation of the probability of failure is to simulate outcome of stochastic variables with an arbitrary distributions. In a physical experiment it might be possible to select a sample value of each basic variable by means of some arbitrary random selection process, such as putting a sequence of numbers in a lot and selecting one. Provided that the lot size is large and the interval between numbers small, the probability distribution for these numbers would be the uniform or rectangular distribution, given by

$$F_V(v) = \begin{cases} v & if \quad 0 \leq v \leq 1 \\ 0 & if \quad else \end{cases} \tag{A.22}$$

It is possible to generate uniformly distributed random numbers through automated roulettes or the noise properties of electronic circuits. These generators tend to be slow and non-reproducible, so that an experiment can never be checked. Tables of random numbers can be stored in computer systems, but their recovery for use is also very slow (Melchers 1999).

The most common practical approach is to employ a *pseudo random number generator* (PRNG) such as is available on virtually all computer systems. They are termed *pseudo* since they use a formula to generate a sequence of numbers. Although this sequence is reproducible and repeats after (normally) a long cycle interval, for many practical purposes it is indistinguishable from a sequence of strictly true random numbers (Rubinstein 1981). The production of a reproducible sequence has an advantage in certain problems and in research work. If required, reproducibility can be destroyed simply by changing (randomly) the *seed number* required as input for most PRNGs. A simple device is to use the local time as a seed value (Naess 1995).

The generation of random numbers according to a specific distribution is the heart of Monte Carlo simulation.

A.4.3 Generation of random variates

Basic variables only seldom have a uniform distribution. A sample value for a basic variable with a given (nonuniform) distribution is called a *random variate* and can be obtained by a number of mathematical techniques. The most general of these is the *Inverse Transform* method. For a general stochastic variable X the distribution function if $F_X(x)$. In the inverse method two steps are needed to generate an outcome $\hat{x}$ of X:

1) generate an outcome $\hat{v}$ of V (example: using a multiplicative congruence generator)

2) determine the outcome of $\hat{x}$ by;

$$\hat{x} = F_X^{-1}\left(F_V\left(\hat{v}\right)\right) = F_X^{-1}\left(\hat{v}\right) \tag{A.23}$$

The method is illustrated in figure A.5. It is seen that the distribution function for $\hat{X}$ with outcomes generated by the procedure is

$$F_{\hat{X}}\left(x\right) = P\left(\hat{X} \leq x\right) = P\left(F_X^{-1}\left(V\right) \leq x\right) = P\left(V \leq F_X\left(x\right)\right) = F_X\left(x\right) \tag{A.24}$$

Specialized techniques for generating random variates from specific distributions often are computationally more efficient than the inverse transform method. Most computer systems have standard subroutines available. One such procedure (Box 1958) produces a pair of exact independent standardized normal variates, u_1 and u_2 given by

$$u_1 = \left(-2\ln r_1\right)^{1/2} \sin\left(2\pi r_2\right) \tag{A.25}$$

and

$$u_2 = \left(-2\ln r_1\right)^{1/2} cos\left(2\pi r_2\right) \tag{A.26}$$

where r_1, r_2 are realizations of uniformly distributed independent random variables R in the interval (0,1). Lognormally distributed random variables v_i may be obtained directly from expressions (A.25) and (A.26) since, for V lognormally distributed, $v_i = ln(u_i)$.

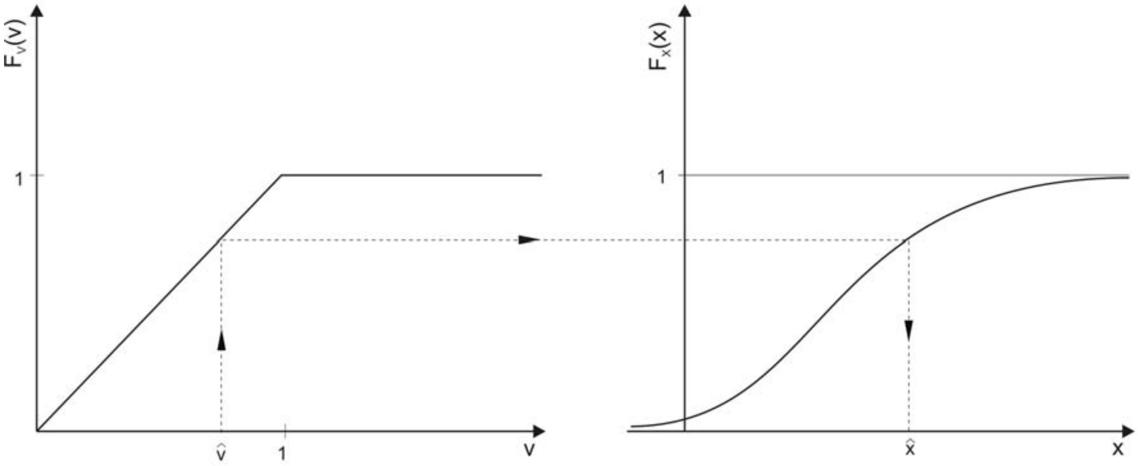


Figure A.5 - Illustration of inverse method.

The next task is to transform the uniform random numbers between 0 and 1, either generated by a computer or obtained from a table, to a random numbers with the appropriate characteristics. This is commonly known as the *Inverse Transformation* method or *Inverse CDF* method.

If X is normally distributed, that is, $N(\mu_X, \sigma_X)$, then $S = (X - \mu_X)/ \sigma_X$ is a standard normal variate, that is, $N(0, 1)$. It can be shown that

$$u_i = F_X\left(x_i\right) = \Phi\left(s_i\right) = \Phi\left(\frac{x_i - \mu_X}{\sigma_X}\right) \tag{A.27}$$

or

$$s_i = \frac{x_i - \mu_X}{\sigma_X} \tag{A.28}$$

thus,

$$x_i = \mu_X + \sigma_X s_i = \mu_X + \sigma_X \Phi^{-1}\left(u_i\right) \tag{A.29}$$

Equation (A.29) suggests that in this case, the u_i values first need to be transformed to s_i, that is, $s_i = \Phi^{1}(u_i)$, and Φ^{1} is the inverse of the CDF of a standard normal variable.

The x_i values can be calculated from the information on the s_i values. For $u_1 = 0.86061$, $s_1 =$

$\Phi^{-1}(0.86061) = 1.08306$; with the information on μ_X and σ_X, the corresponding x_1 can be calculated.

A computer program can be written to generate random numbers according to any distribution. In fact, many available computer programs can generate random numbers for commonly used distributions. If the computer cannot generate a specific distribution, equation (A.30) can be used to obtain it.

$$\hat{x}_i = F_X^{-1}(u_i) \tag{A.30}$$

A.4.4 Direct sampling (*simple* Monte Carlo)

The *Inverse Transform* method is the simplest Monte Carlo approach for reliability problems but not the most efficient. The basis for its application is as follows. The probability of limit state violation may be expressed as (Naess 1995)

$$p_f = J = \int \ldots \int I\left[G(x) \leq 0\right] f_X . dx \tag{A.31}$$

where *I[ ]* is an 'indicator function' which equals *1* if *[ ]* is true and *0* if *[ ]* is false. Thus the indicator function identifies the integration domain. If $\hat{x}_j$ represents the *jth* vector of random observations from f_x *( )*, it follows directly from sample statistics that

$$p_f = J_1 = \frac{1}{N} \sum_{j=1}^{N} I\left[G\left(\hat{x}_j\right) \leq 0\right] \tag{A.32}$$

is an unbiased estimator of *the* p_f. Thus expression (A.32) provides a direct estimate of p_f. It is known that if the value of *g( )* is less than zero, it indicates failure. Let N_f be the number of simulation cycles when *g( )* is less than zero and let *N* be the total number of simulation cycles. Therefore, an estimate of the probability of failure can be expressed as

$$p_f = \frac{N_f}{N} \tag{A.33}$$

In exploiting this procedure, three matters are of interest: how to extract most information from the simulation points, how many simulation points are needed for a given accuracy, or conversely, how to improve the sampling technique to obtain greater accuracy for the same or fewer sample points. One way that the results of the above sampling technique may be represented is as a cumulative distribution function $F_G(g)$. Obviously, much of the range of $F_G(g)$ is of little interest, since the structure or member is (usually) amply safe. To estimate the failure probability the region for which $G( ) \leq 0$, which represent 'failure', is clearly of most interest.

The estimate of p_f in equation (A.32) may be improved by fitting an appropriate distribution function through the points for which $G(\ )\leq 0$. However, the choice of an appropriate distribution function may be difficult. Rather than fitting a single distribution to the sample points, it is possible to fit a sequence of distributions and to optimize their parameters to give a *best fit* to the sample points (Grigoriu 1983).

N random numbers for each of the random variables in the problem will give N sets of random numbers, each set representing a realization of the problem. Thus, solving the problem N times deterministically will give N sample points, essentially generating information on the randomness in the output or response of the system to each set of input variables. The N generated sample points for the output or response can then be used to calculate all the required sample statistics, the histogram, the frequency diagram, the PDF and the corresponding CDF, and the probability of failure considering various performance criteria. The accuracy of the evaluation will increase as the number of simulations N increases.

A.4.5 Accuracy and Efficiency of Simulation

The ability of (A.33) to accurately estimate the probability of failure is a matter of concern. Obviously, the accuracy of the estimate will depend on the number of simulation cycles. For a small failure probability and/or small N, the estimate of p_f given by (A.33) may be subject to considerable error. The estimate of the probability of failure would approach the true value as N approaches infinity. The accuracy of (A.33) can be studied in several ways. One way would be to evaluate the variance or COV of the estimated probability of failure (Ayyub 1985). The variance or COV can be estimated by assuming each simulation cycle to constitute a Bernoulli trial, and the number of failures in N trials can be considered to follow a binomial distribution. Then, the COV of p_f can be expressed as

$$COV\left(p_f\right)=\delta_{p_f}\approx\frac{\sqrt{\dfrac{p_f\left(1-p_f\right)}{N}}}{p_f} \tag{A.34}$$

A smaller value of δ_{Pf} is desirable. Equation (A.34) indicates that δ_{Pf} approaches zero as N approaches infinity.

The standard error of p_f is estimated by

$$s\left(p_f\right)\approx\sqrt{\frac{p_f\left(1-p_f\right)}{N}} \tag{A.35}$$

Another way to study the error associated with the number of simulation cycles is by approximating the binomial distribution with a normal distribution and estimating the 95% confidence interval of the estimated probability of failure (Shooman 1968). It can be shown that

$$P\left[-2\sqrt{\frac{p_f\left(1-p_f^T\right)p_f^T}{N}}<\frac{N_f}{N}<2\sqrt{\frac{p_f\left(1-p_f^T\right)p_f^T}{N}}\right]=0.95 \tag{A.36}$$

where p_f is the true probability of failure. The percentage error can be defined as

$$\varepsilon\%=\frac{\frac{N_f}{N}-p_f^T}{p_f^T}\cdot 100\% \tag{A.37}$$

Combining (A.36) and (A.37), we obtain

$$\varepsilon\%=\sqrt{\frac{1-p_f^T}{N.p_f^T}}\cdot 200\% \tag{A.38}$$

Equation (A.38) indicates that there will be about 20% error if p_f^T is 0.01 and if 10,000 trials were used in the simulation. It can also be stated that there is 95% probability that the probability of failure will be in the range of 0.01 ± 0.002 with 10,000 simulations. Conversely, if the desired error is 10% and p_f^T is 0.01, then from (A.38), the required number of simulations N = 39,600.

Both (A.34) and (A.38) indicate that the number of simulation cycles to achieve a certain level of accuracy depends on the unknown probability of failure. In many engineering problems, the probability of failure could be smaller than 10^{-5}. Therefore, on average, only 1 out of 100,000 trials would show a failure. Thus, at least 100,000 simulation cycles are required to predict this behaviour. For a reliable estimate, at least 10 times this minimum (i.e., 1 million simulation cycles) is usually recommended. If the problem has *n* random variables, then *n* million random numbers are necessary if the Monte Carlo simulation is to successfully estimate the probability of failure (Haldar 2000).

Another way to evaluate the number of simulations required for a given confidence level may be made as follows. Since *G(X)* is a random variable in *X*, the indicator function *I[G(X) ≤ 0]* is also a random variable, albeit with only two possible outcomes. It follows from the central limit theorem that the distribution of J_1, given by the sum of independent sample functions (A.83) approaches a normal distribution as $N\to\infty$. The mean of this distribution is

$$E(J_1)=\sum_{j=1}^{N}\frac{1}{N}I[G\le 0]=E\big[I[G\le 0]\big] \tag{A.39}$$

which is equal to J (see A.83), while the variance is given by

$$\sigma_{J_1}^2=\sum_{j=1}^{N}\frac{1}{N^2}\mathrm{var}\big[I[G\le 0]\big]=\frac{\sigma^2_{I[G\le 0]}}{N} \tag{A.40}$$

This shows that the standard deviation of J_1 and hence of the Monte Carlo estimate (A.83) varies directly with the standard deviation of $I(\ )$ and inversely with $N^{1/2}$. These observations are important in determining the number of simulations required for a particular level of confidence. To actually calculate confidence levels, an estimate of $\sigma_I\ ()$ is required. The variance is given by

$$\mathrm{var}\big[I(\ )\big]=\int\ldots\int\big[I\big(G(x)\le 0\big)\big]^2\,dx-J^2 \tag{A.41}$$

so that the sample variance is given by:

$$S^2_{I(G\le 0)}=\frac{1}{N-1}\left(\left\{\sum_{j=1}^{N}I^2\big[G(\hat{x}_j)\le 0\big]\right\}-N\left\{\frac{1}{N}\sum_{j=1}^{N}I\big[G(\hat{x}_j)\le 0\big]\right\}^2\right) \tag{A.42}$$

where the last { } term is simply the mean (A.39) or, by (A.40) the estimate J_1 for p_f.
On the basis that the central limit theorem applies, the following confidence statement can be given for the number (J_1) of trails in which 'failure' occurs:

$$P\left(-k\sigma<J_1-\mu<+k\sigma\right)=C \tag{A.43}$$

where μ is the expected value of J_l given by (A.39) and σ is given by (A.91). For confidence interval C = 95%, k = 1.96, as can be verified from standard normal tables. As σ is not known, it may be estimated using (A.42). However, this is not very helpful at the beginning of a Monte Carlo study.

It has been suggested (Broding 1964) that σ and μ in (A.43) can be approximated by the binomial parameters $\sigma=(Nqp)^{1/2}$ and $\mu=Np$, with $q = 1-p$, provided that $Np > 5$ when $p < 0.5$. If these are substituted in (A.43) there is obtained

$$P\left[-k(Nqp)^{1/2}<J_1-np<+k(Nqp)^{1/2}\right]=C \tag{A.44}$$

If the error between the actual value of J_1, and the observed value is denoted by $\varepsilon = (J_1-Np)/Np$ and this is substituted into (A.44), it follows easily that $\varepsilon = k[(1-p)/Np]^{1/2}$. Thus, for N = 100 000 samples, and $p = p_f = 10^{-3}$ (expected), the error in (J_1) and hence p_f will be less than 20% with 95% confidence (as then k = 1.96). A first estimate of the number N of simulations for a given confidence level C in the failure probability p_f can be obtained from (Broding 1964)

$$N > \frac{-\ln\left(1-C\right)}{p_f} \qquad \text{(A.45)}$$

Thus, for a 95% confidence level and p_f = 10 ^{3}, the required number of simulations is more than 3000. The actual number of variates to be calculated is, of course, N times the number of independent basic variables. Others have suggested that the number of simulations may need to be of the order of 10000 - 20000 for approximately 95% confidence limit, depending on the function being evaluated (Mann 1974).

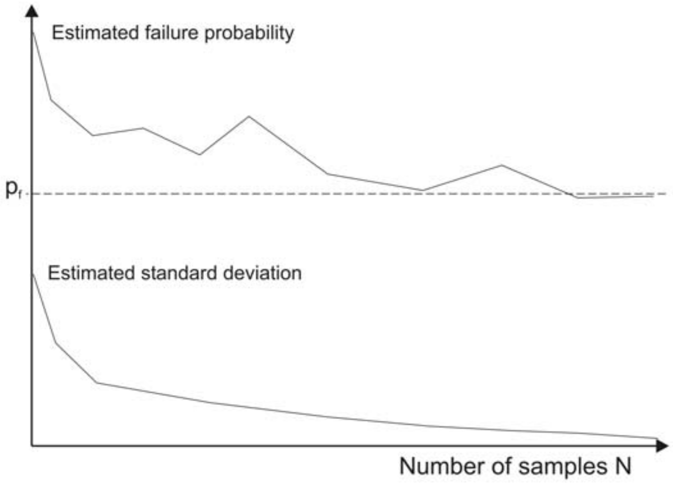


Figure A.6 - Typical convergence of probability estimate with increasing sample size (Melchers 1999).

The above *rules* while often useful, do not tell the analysist much about the accuracy being achieved in any particular Monte Carlo analysis. A useful tool for this purpose is to plot (or print-out) progressive results of the estimate of p_f and its estimated variance as obtained from (A.38) and (A.42) respectively (Melchers 1999) (see Figure A.6).

A.4.6 Variance Reduction Techniques

The concept behind simulation appears to be simple; however, its application in engineering reliability analysis and its acceptance as an alternative reliability evaluation method depend mainly on the efficiency of the simulation. To achieve efficiency, the number of simulation cycles needs to be greatly reduced. It is the simulator's task to increase the efficiency of the simulation by expediting the execution and minimizing computer storage requirements. Alternatively, efficiency can be increased by reducing the variance or the error of the estimated output variable without disturbing the expected or mean value and without increasing the sample size. This need led to the development of *variance-reduction techniques* (VRTs). The type of VRT that can be used depends on the particular model under consideration. It is usually impossible to know beforehand how much variance reduction might be achieved using a given technique (Haldar 2000).

The VRTs can also be grouped according to description or purpose (i.e.. sampling methods, correlation methods, and special methods). These groupings are somewhat arbitrary: however, they produce a better understanding of the concept involved. The sampling methods either constrain the sample to be representative or distort the sample to emphasize the important aspects of the function being estimated. Some of the commonly used sampling methods are systematic sampling, importance sampling, stratified sampling, Latin hypercube sampling, adaptive sampling, randomization sampling, and conditional expectation. The correlation methods employ strategies to achieve correlation (both positive and negative) between functions, random observations, or different simulations to improve the accuracy of the estimators. Some of the commonly used correlation methods are common random numbers, antithetic variates, and control variates. Other special VRTs available are partition of the region, random quadratic method, biased estimator, and indirect estimator. The VRTs can also be combined to further increase the efficiency of the simulation (Haldar 2000).

VRTs increase the efficiency and accuracy of the risk or reliability estimation using a relatively small number of simulation cycles; however, they increase the computational difficulty for each simulation, and a considerable amount of expertise may be necessary to implement them. The most desirable feature of simulation, its basic simplicity, is therefore lost. Also, in the age of high-speed computers, the number of simulation cycles or time required to analyze a problem may be less important than in the past. In any case, it is important to understand the logic and concepts behind some of the VRTs commonly used in engineering. The commonly used VRTs include sampling methods and correlation methods. The two types of methods can also be combined (Haldar 2000).

A.4.6.1 Importance Sampling

The basic idea of importance sampling is to concentrate the distribution of sampling points in the region of most importance, that is, the area that mainly contributes to the failure probability, instead of spreading them out evenly among the whole range of possible values of the basic variables. One method to achieve this is illustrated in figure A.7 (Harbitz 1986). Efficient variance reduction is obtained by simulating only outside the p-sphere, because no failure occurs within the β-sphere.

The basic mathematical definition of failure probability is given by (A.18), in which the joint probability density function for basic random variables, X, is integrated over the failure region, that is, $g(x) < 0$. A simple indicator function with respect to the performance function *g(x)* can be defined as

$$I_g(x) = \begin{Bmatrix} 0, & if & g(x) > 0 \\ 1, & if & g(x) < 0 \end{Bmatrix} \tag{A.46}$$

With this, (A.18) can be rewritten as

$$p_f = \int_{g(x)<0} I_g(x) f_X(x) dx \tag{A.47}$$

This is just the expected value of $I_g(x)$. Therefore, in basic Monte Carlo simulation, the probability of failure is simply

$$p_f = \frac{1}{N} \sum_{i=1}^{N} I_g(x_i) = \frac{n_f}{N} \tag{A.48}$$

where n_f is the number of failures.

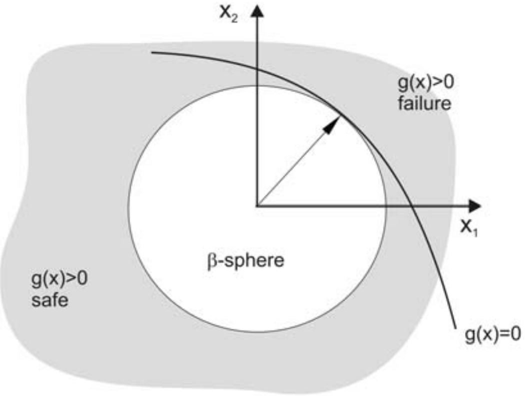


Figure A.7 - Harbitz's Importance Sampling Method (Harbitz 1986).

For importance sampling, a new sampling PDF $f_1(x)$ is defined so as to obtain samples in the desired region $f_1(x)$ is known as the sampling density function. The probability of failure is given by

$$p_f = \int_{g<0} \left[I_g(x) \frac{f(x)}{f_1(x)} \right] f_1(x).dx \tag{A.49}$$

Equation (A.49) is obtained by multiplying and dividing the integrand in (A.98) by $f_1(x)$. Similar to (A.48), the simulated estimate of p_f is

$$p_f = \frac{1}{N} \sum_{i=1}^{N} I_g(x_i) \frac{f_X(x_i)}{f_1(x_i)} \tag{A.50}$$

where *N* is the number of simulations and x_i, that is $(x_1, x_2, \ldots, x_n)_i$ is the set of values of the basic random variables at the *i*th simulation.

The accuracy of the importance sampling estimate depends on the choice of the sampling density $f_1(x)$. Several methods have been developed for this purpose (Harbitz 1986),(Karamchandani 1989) and (Melchers 1989). The standard error of the estimate p_f is

$$s=\sqrt{\frac{1}{N(N-1)}\left\{\sum_{i=1}^{N}\left(I_g(x_i)\frac{f_X(x_i)}{f_1(x_i)}\right)^2-\frac{1}{N}\left[\sum_{i=1}^{N}\left(I_g(x_i)\frac{f_X(x_i)}{f_1(x_i)}\right)^2\right]\right\}} \tag{A.51}$$

A.4.6.2 Importance Sampling based on the β-point

If the β-point has been determined before the simulation techniques are used importance sampling can be very effective with the β-point as the point around which the samplings are concentrated, see figure A.1A. The sampling density function $f_{\hat{s}}$ in (A.50) is the normal density of uncorrelated variables with expected values u^*_i, $i=1,2,\ldots n$ and common standard deviations σ. p_f is estimated by

$$p_f=\frac{1}{N}\sum_{j=1}^{N} I\left[g\left(\sigma\hat{\bar{u}}_j+\bar{u}^*\right)\right]\frac{f_{\bar{U}}\left(\sigma\hat{\bar{u}}_j+\bar{u}^*\right)}{f_{\bar{U}}\left(\hat{\bar{u}}_j\right)}\sigma_n \tag{A.52}$$

Where $f_U\left(\bar{u}\right)$ is the standardized normal density function and u_j is a sample generated from standard normal variables.

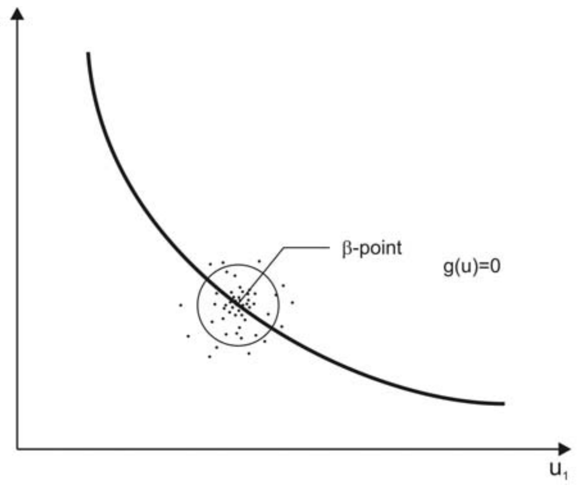


Figure A.8 - Importance sampling around the β-point (Naess 1995).

The standard error is estimated by (A.51). The efficiency of the importance sampling can be expected to be dependant on the choice of the standard deviation of the sampling density, seen in figure A.9. It should be noted that if a failure mode has multiple β-points importance sampling based on only one β-point is not efficient. In this case more general methods have to be used.

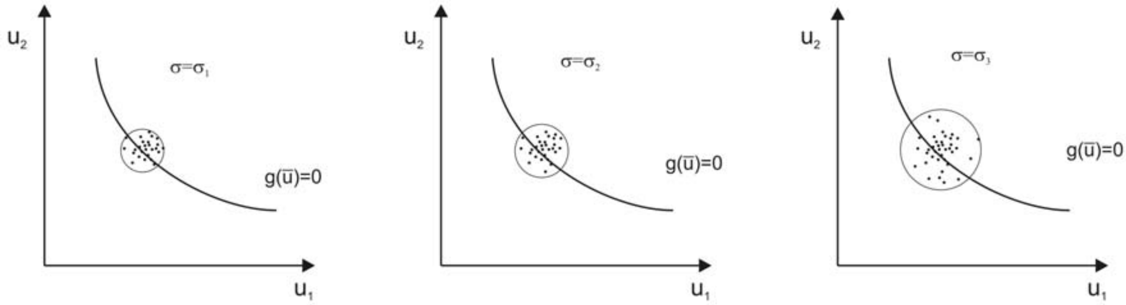


Figure A.9 - Different standard deviations of the sampling density, $\sigma_1 < \sigma_2 < \sigma_3$ (A.3).

A.4.6.3 Stratified Sampling

In the stratified sampling method, the domain of integration is divided into several regions, so that emphasis can be placed by simulating more from the regions that contribute to the failure event. The total domain of integration is divided into m mutually exclusive regions, that is, $R_1, R_2, \ldots, R_m$. Using the theorem of total probability, we can estimate the probability of failure as

$$p_f = \sum_{j=1}^{m} \left[P(R_j) \frac{1}{N_j} \sum_{i=1}^{N_j} I_g(x_i) \right] \tag{A.53}$$

where $P(R_j)$ is the probability of region R_j, N_j is the number of simulation cycles performed in region R_j, and $I_g(\,)$ is the indicator function defined in (A.46). This strategy ensures that no region is missed (Naess 1995).

A.4.6.4 Adaptive Sampling

In the importance sampling method, the region of importance is usually not known in advance. However, the efficiency of the simulation can be improved if it can be updated using the information obtained from the first few simulation cycles. This observation led to the development of the adaptive sampling methods. Two methods of adaptive sampling are multimodal sampling (Karamchandani 1989) and curvature-based sampling (Wu 1987). Both methods start with a sampling density centered on the design point identified by FORM. As the simulation progresses, the sampling density is updated. In the multimodal method, the sampling density is a weighted sum of density functions centered in different regions. Some authors used this concept for system reliability analysis (Mahadevan 1997). In the curvature-based method, the curvature of the limit state is updated every few simulations, and this information is used in selecting the future samples. It has been reported in the literature (Ayyub 1997, Mahadevan 1997). that this technique can reasonably estimate the probability of failure with only 100 to 400 cycles, whereas similar results can only be obtained by using several million direct simulation cycles.

A.4.6.5 Conditional Expectation

In the conditional expectation method, all the basic random variables are simulated except one, known as the control variable. The control variable is generally selected to be the random variable with the largest variability, and its CDF provides a known function in terms of other simulated variables. The conditional expectation can be evaluated by this known expression. The only limitation is that the control random variable must be statistically independent of all other random variables in the performance function of interest. The variance is reduced by removing the random fluctuations of the control variable on which conditioning was not performed by not generating them. The steps involved in the conditional expectation method can be summarized as follows:

- Step 1. Select the control variable (i.e., the random variable of the largest variability). Let it be X_j.
- Step 2. Rewrite the quantity to be estimated, in this case p_f as

$$p_f = P[\, X_j < g'\,(\, X_l, X_2, \ldots, X_{j-l}, X_{j+l}, \ldots, X_n\,)] \tag{A.54}$$

- Step A. Randomly generate values of all these variables except X_j using Monte Carlo simulation and calculate a sample of p_f as follows:

$$p_f = P[\, X_j < g'\,(\, X_{li}, X_{2i}, \ldots, X_{(j-l)i}, X_{(j+l)i}, \ldots, X_{ni})] \tag{A.55}$$

or

$$p_f = F_{Xj}\,[\, g'\,(\, X_{li}, X_{2i}, \ldots, X_{(j-l)i}, X_{(j+l)i}, \ldots, X_{ni})] \tag{A.56}$$

where $F_{Xj}[\ ]$ is the CDF of X_j.

- Step 4. Repeat Step 3 N times then calculate the mean of p_{fi} denoted as p_f as

$$p_f = \frac{\sum_{i=1}^{N} p_{f_i}}{N} \tag{A.57}$$

The variance of p_f is

$$Var\left(p_f\right) = \frac{Var\left(p_{f_i}\right)}{N} = \frac{\frac{1}{N-1}\sum_{i=1}^{N}\left(p_{f_i} - p_f\right)^2}{N} \tag{A.58}$$

APPENDIX 2
DURACON – USERS MANUAL

DURACON

Probability-Based Durability Analysis of Concrete Structures in Severe Environment

USER MANUAL

Version 1.0 **September 2004**

DISCLAIMER

The DURACON Software has primarily been developed in order to provide a basis for probability-based durability analysis of concrete structures in severe environment. The program is intended for use by professionals who are competent to evaluate its significance and limitations, and who will accept responsibility for the application of the material it contains. The author responsible for the development of the software shall not be liable for any loss or damage arising thereof. Users of this software should note that the specifications contained may be the subject for change without notice.

TABLE OF CONTENT

1. INTRODUCTION

In recent years, much research work has been carried out in order to obtain a more controlled durability and long-term performance of concrete structures in chloride containing environment. In particular, the development of new procedures for probability-based durability design has proved to give a more realistic and powerful basis for the durability analysis [1-3]. In order to facilitate the probability-based approach to a durability analysis, a simple software named DURACON has been developed.

The software is based on a Monte Carlo simulation of Fick's Second Law of Diffusion adjusted to allow for the time dependency of the diffusion coefficient and the effect of temperature. This approach is primarily being applied for obtaining a more controlled durability and long-term performance of new concrete structures, but it may also be used as a basis for condition assessment of existing concrete structures in chloride containing environment [4-6].

After a short description of the model for the DURACON Software, the Manual gives a simple step-by step explanation for the application of the software. For selection of appropriate input parameters to the software, reference is made to "Practical Guidelines for Probability-Based Durability Design of Concrete Structures in Marine Environment" [7]. As part of the Manual, some typical results of a probability-based durability design are given in order to demonstrate the practical application of the DURACON Software.

2. MODEL DESCRIPTION

2.1 General

For the DURACON Program, the modelling of chloride penetration and time to depassivation are based on Fick's Second Law of Diffusion in combination with a time dependent diffusion coefficient [8], and the software also incorporates the stochastic nature of the individual durability parameters which are needed as input to the program. These durability parameters include the diffusion coefficient, which may either be obtained from accelerated laboratory testing or curve fitting of chloride profiles from existing concrete structures, the time dependence of the diffusion coefficient and the critical chloride content for depassivation of embedded steel, both of which may be obtained from existing literature or other experience for the given type of cement and concrete. The concrete cover and the environmental exposure expressed in the form of surface chloride concentration, which may either be obtained from measurements or previous experience, are also important durability and input parameters for the software program.

As time to depassivation and onset of steel corrosion is used as a basis for the Serviceability Limit State (SLS), the program expresses the probability of failure or risk for the SLS to be reached after a certain period of time. For new concrete structures, this provides an appropriate basis for establishing overall durability criteria for the structure in question [7], while for existing concrete structures where the chloride front has still not reached the embedded steel, the program can be used for estimating the probability of the onset of corrosion after a further period of exposure.

2.2 Rate of chloride penetration

Fick's Second Law of Diffusion is given by the following expression:

$$\frac{dC(x,t)}{dt} = D_C \cdot \frac{d^2C(x,t)}{dx^2} \tag{1}$$

where $C(x,t)$ is the chloride ion concentration at a distance x from the concrete surface after being exposed for a period of time t, and D_C is the chloride diffusion coefficient. By solving this equation for predefined boundary conditions, the following equation is obtained:

$$C(x,t) = C_S\left[1 - erf\left(\frac{x}{2\sqrt{D.t}}\right)\right] \tag{2}$$

where C_S is the chloride ion concentration on the concrete surface, and *erf* is the error function.

The time dependence of the diffusion coefficient is normally expressed as:

$$D(t) = D_0 \cdot \left(\frac{t}{t_0} \right)^{\alpha} \tag{3}$$

where D_0 is the diffusion coefficient at a given time t_0, and the exponent α represents the time dependence of the diffusion coefficient or the increased ability of the concrete to resist chloride penetration over time.

By substituting Eq. 3 into Eq. 2, an expression is obtained that permits the prediction of chloride penetration based on the time dependent diffusion coefficient, given by:

$$c_X = c_S \left[1 - erf\left(x / 2.\sqrt{D_0 \cdot t \cdot (t/t_0)^{\alpha}} \right) \right] \tag{4}$$

2.3 Probabilistic approach

The probabilistic approach is based on the Monte Carlo Method (MCM), which can be briefly described as a statistical simulation method, where sequences of random numbers are applied to perform the simulation. In the present application of simulation, the physical process is simulated directly by use of the modified Fick's Second Law of Diffusion for describing the transport process. The only requirement is that all the input parameters to the equation be described by a probability density function (PDF). Once the PDF's of the various durability parameters of the system are known, the probability of failure is based on the evaluation of the limit state function for a large number of trials.

When a simulation method is used for calculating the probability of failure, the failure function is calculated for each outcome. If the outcome is in the failure region, then the contribution to the probability of failure is obtained. The probability of failure is estimated by the following expression:

$$p_f = \frac{1}{N} \cdot \sum_{j=1}^{N} I\left[g\left(r_j, s_j \right) \right] \tag{5}$$

where N is the number of simulations, $I\ [g(r_j,s_j)]$ is the indicator function and $g(r_j,s_j)$ is the limit state equation, where s represents the environmental load and r is the resistance of the concrete against chloride penetration.

The standard error of the probability of failure is estimated by:

$$s = \sqrt{\frac{p_f (1 - p_f)}{N}} \tag{6}$$

Since the accuracy of the Monte Carlo Method (MCM) mainly depends on the number of trials and the method is easy to implement, a simulation based on MCM appears to be both simple and intuitive.

2.4 Assumptions and limitations

It should be noted that the rate of chloride penetration may also be controlled by other mechanisms such as capillary absorption. However, based on current knowledge, it appears that diffusion is a dominating transport process for the chloride penetration through the concrete cover. For an existing concrete structure, the mechanism for chloride penetration may not be so important for the durability analysis, since the input parameter for the diffusion coefficient D_o also will reflect the prevailing moisture and temperature conditions for the given structure. Since there is still a lack of relevant data and information on the various input parameters, a critical interpretation of obtained results and sound engineering judgement are important for a proper utilization of the software.

3 INSTALLATION

The program can be installed on any PC that runs a version of Windows® 98, 2000, NT, Me, and XP. Figure 1 illustrates the generic structure of the program seen from the user's point of view.

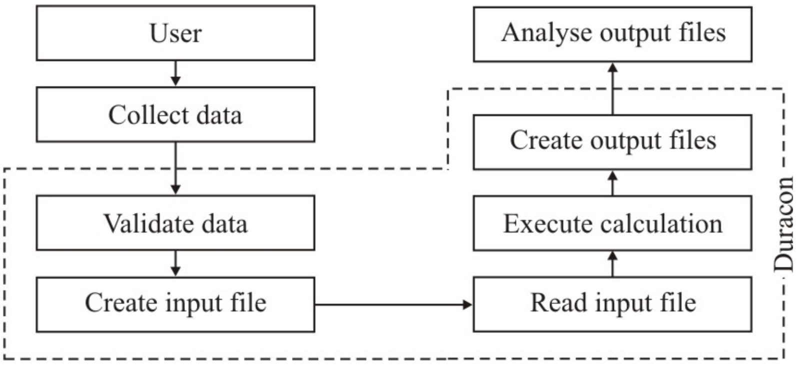


Figure 1 - Generic structure for the use of the program.

All necessary data must be introduced into the input file. Once the input data file is created, the DURACON Program is executed specifying which input data file is to be used. Once the program has executed the calculation, it will create two result files. The user can then consult the result files and analyse them accordingly.

DURACON requires very little disk space for installation. However, once DURACON is run, CPU usage increases dramatically. Therefore, it is recommended that the computer has a fast processor and sufficient RAM memory (greater than 64MB).

For installation, run *Setup.exe* from the installation CD into that directory. Then, the analysis can be executed by clicking on the *Duracon* icon (Figure 2).

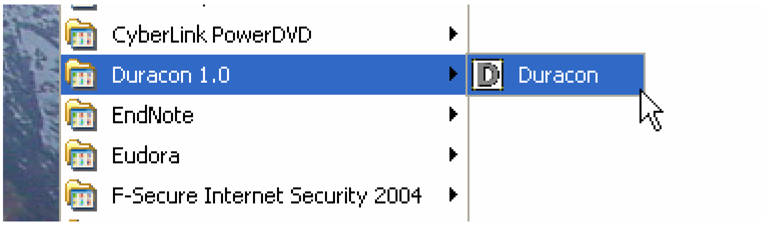


Figure 2 – The Duracon.exe file that initiates the program.

4. OPERATION

4.1 Opening

The description in this chapter follows step by step the process you go through when performing an analysis. There are three options that can be chosen every time DURACON is run:

1) Create an input data file
2) Edit an existing data file
3) Perform a simulation

Figure 3 illustrates the software's opening window. After opening DURACON, you can start by creating an input data file. Figure 4 shows the introduction window and the main window of the program.

Version 1.0 (Beta Version) - Freeware Mode (#0) - Serial # 1107.464.301.0181
Registered to Rui Miguel Ferreira (University of Minho)

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Figure 3 – DURACON's initial window.

DURACON
Probabilistic based Durability Design of Concrete Structures in Marine Environment

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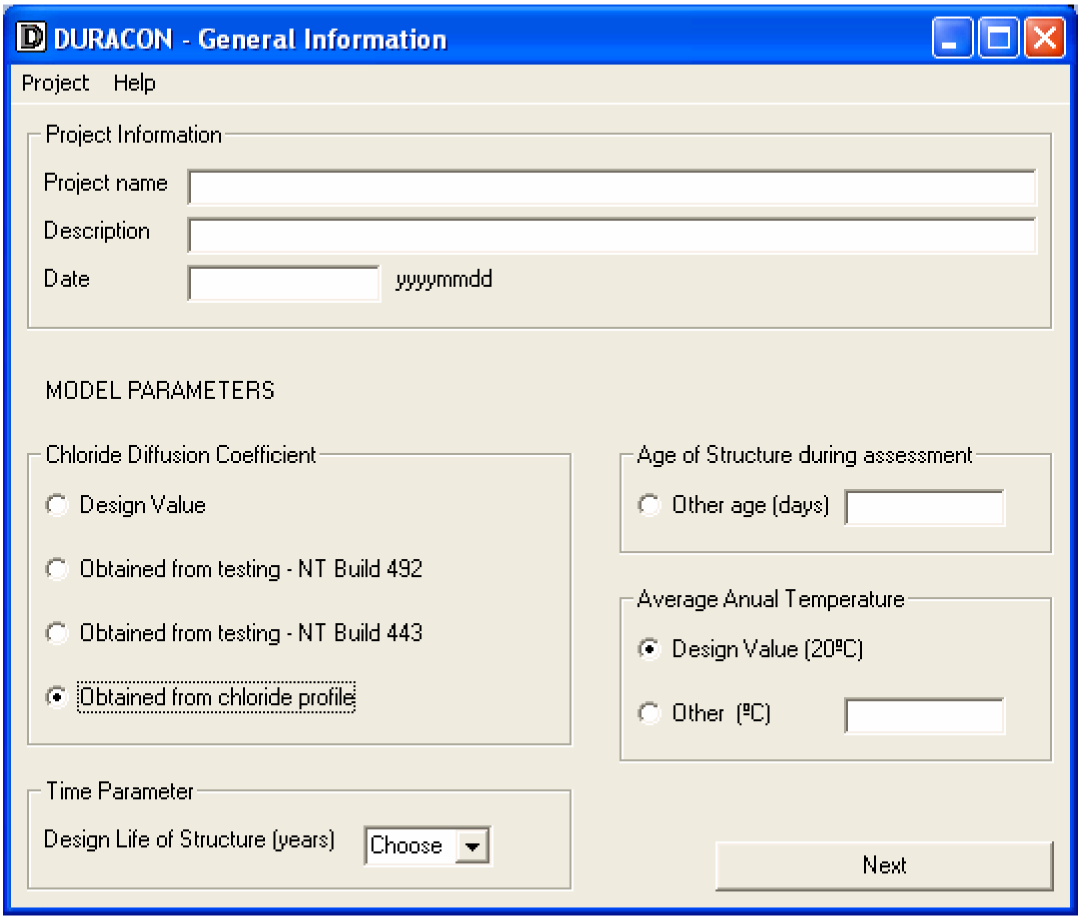
Figure 4 – Main window for DURACON.

4.2 Input data file

From the *Create/Edit Data files* menu, chose *New*. The *Model Parameters* window will then be opened, as shown in Figure 5. In the *General Information* window various choices must be made and information supplied.



DURACON - General Information

Project Help

Project Information

Project name

Description

Date yyyymmdd

MODEL PARAMETERS

Chloride Diffusion Coefficient

○ Design Value

○ Obtained from testing - NT Build 492

○ Obtained from testing - NT Build 443

● Obtained from chloride profile

Age of Structure during assessment

○ Other age (days)

Average Anual Temperature

● Design Value (20ºC)

○ Other (ºC)

Time Parameter

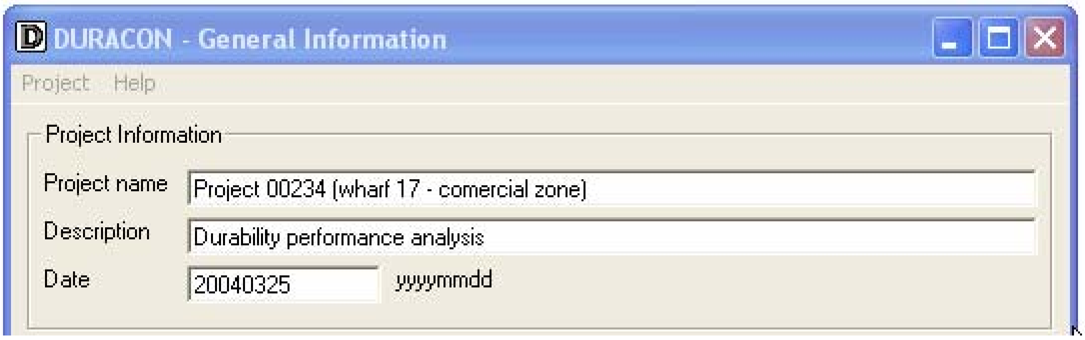
Design Life of Structure (years) Choose

Next

Figure 5 – General Information window.

In *Project Information*, the *Project Name* and project *Description* are introduced. These fields can not be left empty, and can not exceed 200 characters. Only alphanumeric characters are recognised as well as **_ – /** **( )** and **** .

The *Date* is a numeric field where only numeric characters are recognised. No other characters should be used to separate the year, the month and the day. An example is given in Figure 6. The *yyyymmdd* format must be adopted.



Project Information

Project name: Project 00234 (wharf 17 - comercial zone)

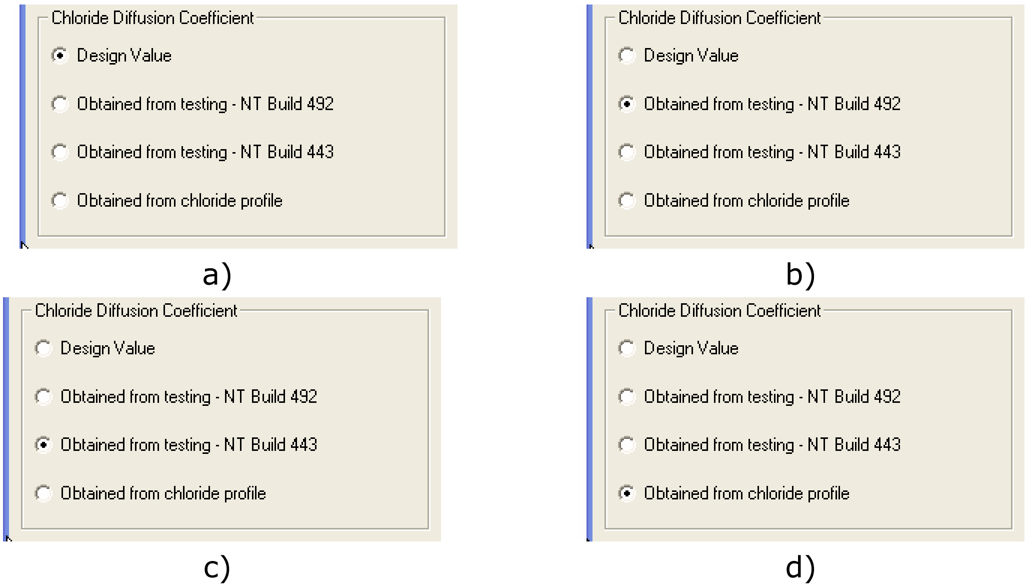
Description: Durability performance analysis

Date: 20040325 yyyymmdd

Figure 6 – Model Parameters window.

The first option of the *Model Parameters* is the *Definition of the Diffusion Coefficient*. (Figure 7). Four options are available depending on the procedure used to determine its value.

If the structure is in the design phase, a design value must be given (Figure 7a). If the diffusion coefficient is determined according to NT Build 492 [9], then the second option is selected (Figure 7b). If the diffusion coefficient is determined by the chloride profile fitting of Fick's Second Law of Diffusion, then the third option (Figure 7c) is selected. The final option is similar to the previous one, but in this case the actual chloride profile is given and the diffusion coefficient as well as the chloride surface content is calculated (Figure 7d). If the fourth option is selected, then the *Profile Parameters* window is shown to collect information on the chloride profile.


Chloride Diffusion Coefficient
● Design Value
○ Obtained from testing - NT Build 492
○ Obtained from testing - NT Build 443
○ Obtained from chloride profile

a)

Chloride Diffusion Coefficient
○ Design Value
● Obtained from testing - NT Build 492
○ Obtained from testing - NT Build 443
○ Obtained from chloride profile

b)

Chloride Diffusion Coefficient
○ Design Value
○ Obtained from testing - NT Build 492
● Obtained from testing - NT Build 443
○ Obtained from chloride profile

c)

Chloride Diffusion Coefficient
○ Design Value
○ Obtained from testing - NT Build 492
○ Obtained from testing - NT Build 443
● Obtained from chloride profile

d)

Figure 7 – Various options for the Diffusion Coefficient.

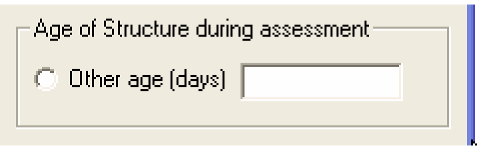
The following data needed is the *Definition of the Time Parameter*. Here, the design life of the structure is chosen from a list of predefined values, as shown in Figure 8.

Time Parameter

Design Life of Structure (years) Choose

Figure 8 – Definition of the time parameters of the Model Parameters window.

Next comes the *Definition of the Age Parameter*. Either the design value of 28 days is chosen or another value is given by the user, as shown in Figure 9.

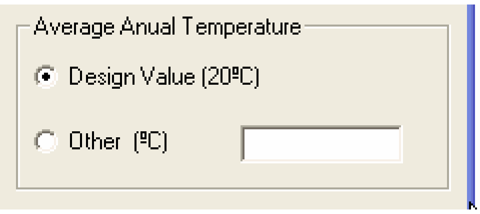


Age of Structure during assessment

○ Other age (days)

Figure 9 – Definition of the Age Parameter of the Model Parameters window.

The final information necessary to proceed to the next page is the *Definition of the Average Annual Temperature*, as shown in Figure 10.



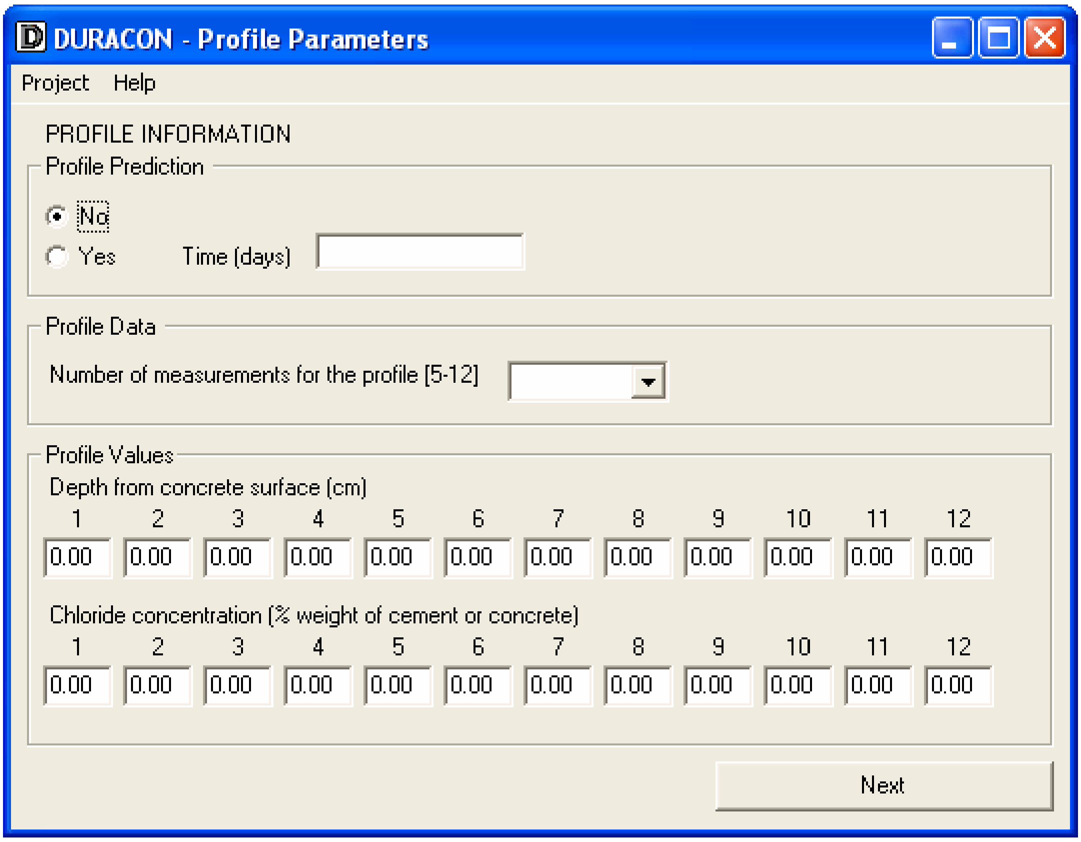
Average Anual Temperature

● Design Value (20ºC)

○ Other (ºC)

Figure 10 – Definition of the Average Annual Temperature.

Once you press the *Next* button, either the *Profile Parameters* window (Figure 11) or the *Distribution Data* window appear. This depends on the procedure chosen to determine the diffusion coefficient. If option (Figure 7d) is not activated then the *Profile Parameters* window is not needed and it is automatically replaced by the *Distribution Data* window.



PROFILE INFORMATION

Profile Prediction

● No

○ Yes Time (days)

Profile Data

Number of measurements for the profile [5-12]

Profile Values

Depth from concrete surface (cm)

1	2	3	4	5	6	7	8	9	10	11	12
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Chloride concentration (% weight of cement or concrete)

1	2	3	4	5	6	7	8	9	10	11	12
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Figure 11 – Number of simulation to perform.

Another feature that DURACON presents is the *Profile Prediction* procedure. If you wish to predict the shape of the chloride profile curve for a future date, this option must be activated and the time in days measured starting from the age of the profile. The *Profile Prediction* option is shown in Figure 12.

PROFILE INFORMATION

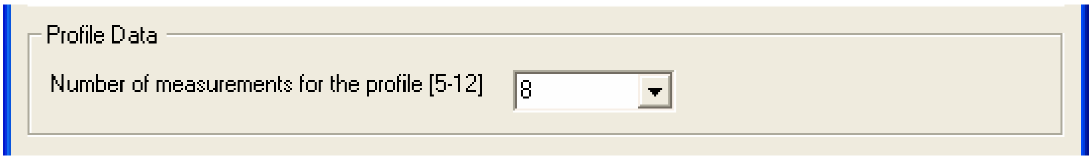
Profile Prediction

○ No

● Yes Time (days) 1800

Figure 12 – Date cell for profile prediction.

In *Profile Data*, the number of measurements per profile is defined (Figure 13). The number of measurements the profile contains should be in between 5 and 12.

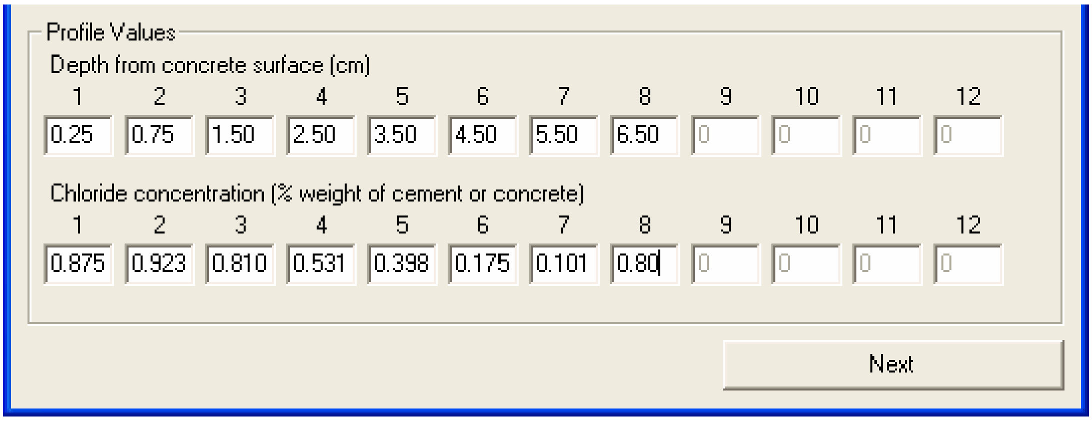


Profile Data

Number of measurements for the profile [5-12]: 8

Figure 13 – Definition of profile size.

Finally, in *Profile Values*, the successive depths of the measurements and the values of the chloride concentration are introduced, as shown in Figure 14.



Profile Values

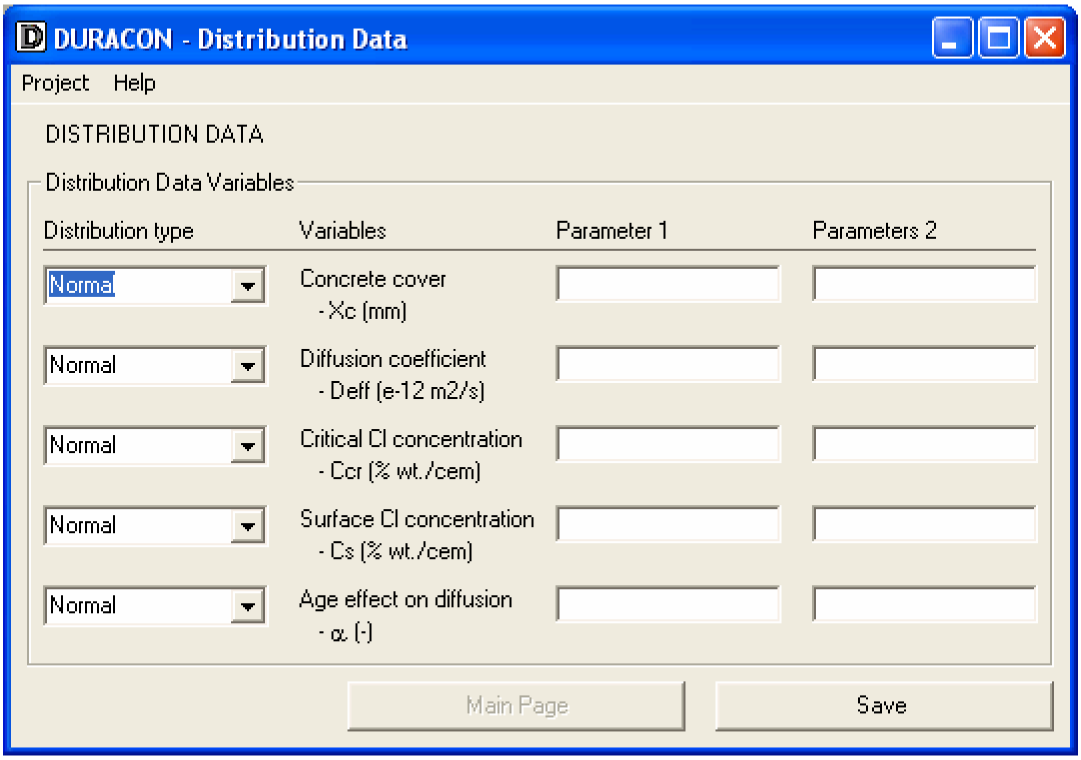
Depth from concrete surface (cm)

1	2	3	4	5	6	7	8	9	10	11	12
0.25	0.75	1.50	2.50	3.50	4.50	5.50	6.50	0	0	0	0

Chloride concentration (% weight of cement or concrete)

1	2	3	4	5	6	7	8	9	10	11	12
0.875	0.923	0.810	0.531	0.398	0.175	0.101	0.80	0	0	0	0

Figure 14 – Profile values.



DISTRIBUTION DATA

Distribution Data Variables

Distribution type	Variables	Parameter 1	Parameters 2
Normal	Concrete cover - Xc (mm)		
Normal	Diffusion coefficient - Deff (e-12 m2/s)		
Normal	Critical Cl concentration - Ccr (% wt./cem)		
Normal	Surface Cl concentration - Cs (% wt./cem)		
Normal	Age effect on diffusion - α (-)		

Figure 15 – Distribution Data window.

The values of the concrete depth must increase in size. Care should be taken to correctly assess the wash-out effect on the surface of the concrete.

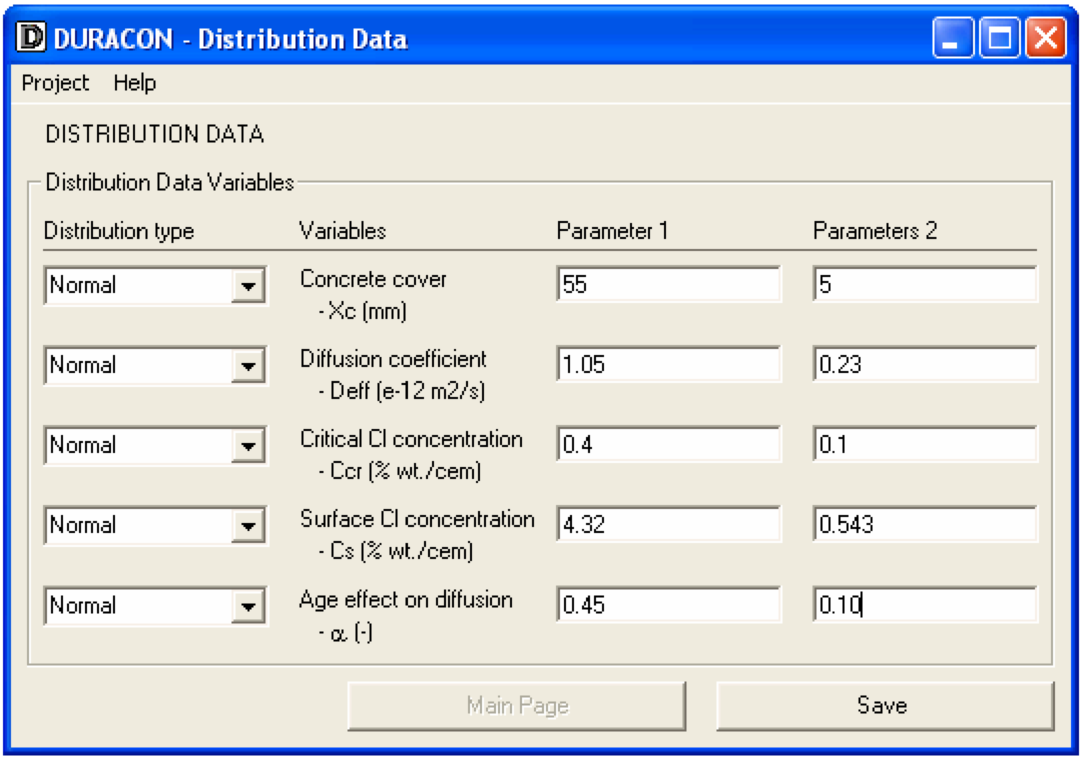
This is also visible in Figure 15.

Once the values are introduced and the *Next* button has been press, we come to the last window, the *Distribution Data* window (Figure 15).

The column on the left hand side of the window is where the distribution characteristics of the data are defined. The distribution options are Normal, Lognormal and Beta. In addition to these distributions, deterministic variables can be chosen. To characterise each of these distributions, only two parameters are needed. These are presented on the two columns to the right of the window.
Table 1 resumes the meaning of each parameter function of the distribution type.

Table 1 – Description of the distribution parameters.

Distribution Type	Parameter 1	Parameter 2
Normal	average	standard deviation
Lognormal	log(Average)	log(Standard deviation)
Beta	parameter Q	parameter R
Deterministic	value	zero (always)



DISTRIBUTION DATA

Distribution Data Variables

Distribution type	Variables	Parameter 1	Parameters 2
Normal	Concrete cover - Xc (mm)	55	5
Normal	Diffusion coefficient - Deff (e-12 m2/s)	1.05	0.23
Normal	Critical Cl concentration - Ccr (% wt./cem)	0.4	0.1
Normal	Surface Cl concentration - Cs (% wt./cem)	4.32	0.543
Normal	Age effect on diffusion - α (-)	0.45	0.10

Figure 16 – Distribution Data window.

Once all the information has been introduced, it must be saved. Once saved, printing is permitted. The program creates two files on the disk when the save button is pressed. The first (***filename.dat***) is used by DURACON to perform the simulation and can be edited manually if necessary. The second (***filename.drc***) is used to load the information of the data file into the file creating/editing option.

These files can be edited manually but advance knowledge for this option is required. This information can be obtained by consulting the software's technical support.

Finally, to edit an input data file, choose Open from the Create/Edit Data files menu. The cells will automatically be filled in. Any changes can then be performed.

4.3 Simulation

To perform a simulation (Figure 17), select an input data file (Figure 18) and the simulation is initiated immediately.

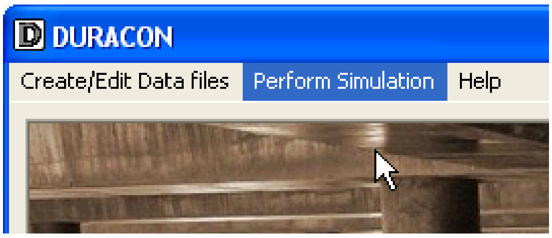
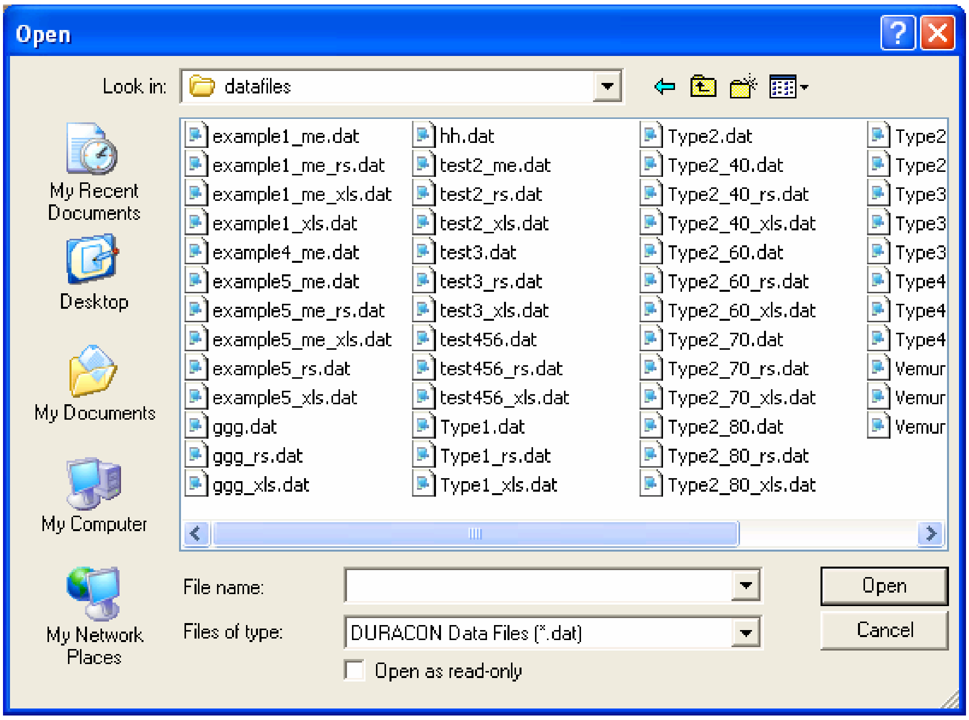


Figure 17 – Option to perform a simulation.



Look in: datafiles

File name:

Files of type: DURACON Data Files (*.dat)

☐ Open as read-only

Figure 18 – Choosing the input data file for calculation.

The duration of the simulation may vary considerable depending on various factors such as number of simulations, distribution type of model parameters, number of time intervals, among others.

Once the simulation has ended, two result files are created. The first (***filename_rs.dat***) contains all the input information, the results and the statistical data for the last time frame simulated. The second (***filename_xls.dat***) contains a small resume of the results so that the file can easily be imported into any graphic software (An example is given in the Appendix for MS Excel).

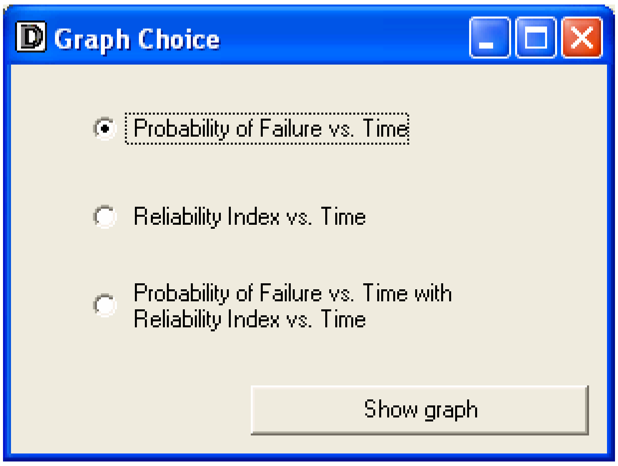
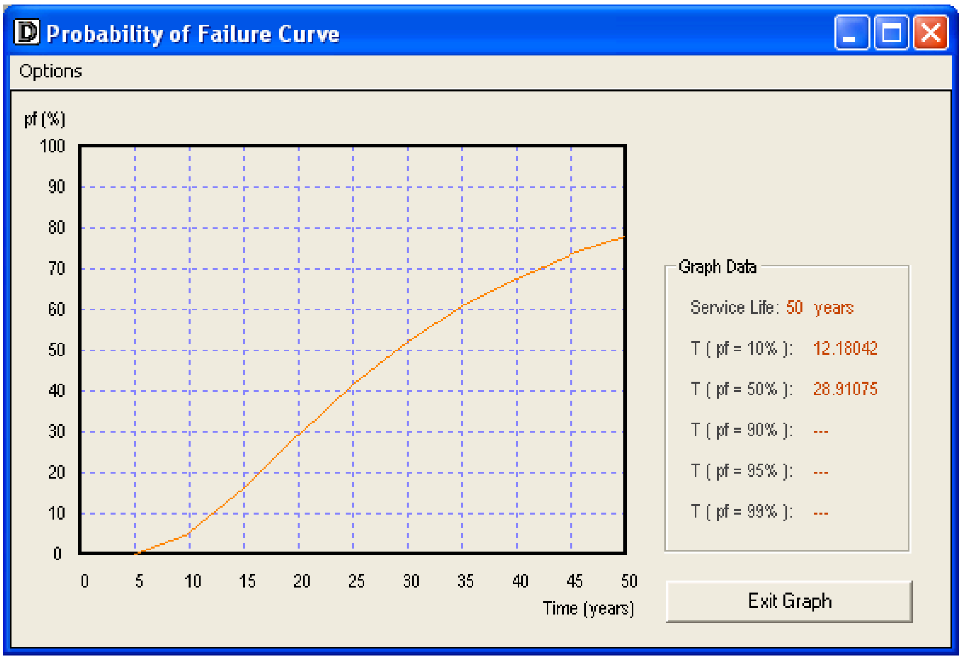


Figure 19 – Choosing type of graph to display.

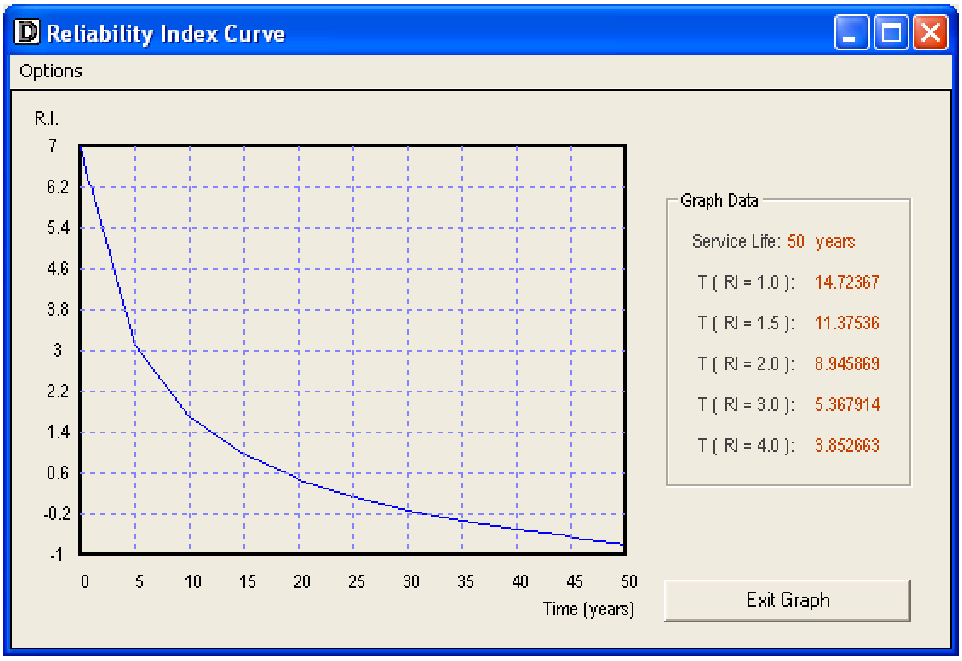
The user then has the option of viewing the outcome of the simulation graphically. The options displayed are shown in Figure 19.



Service Life: 50 years
T (pf = 10%): 12.18042
T (pf = 50%): 28.91075
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

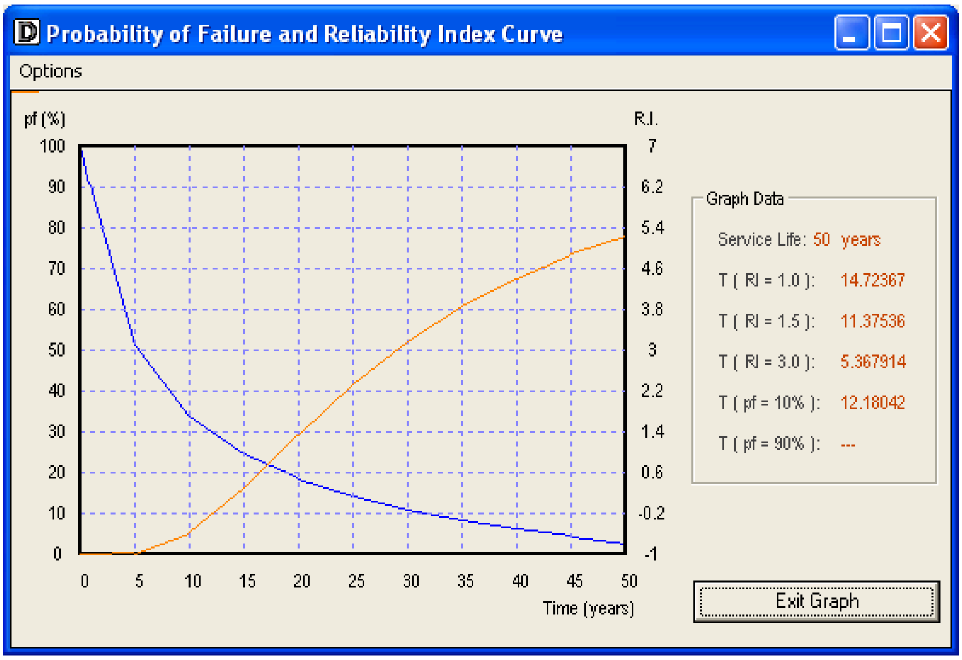
Figure 20 – Probability of Failure versus time curve.

The first two options show the Probability of Failure vs. Time curve and the Reliability Index vs. Time curve individually, as shown in Figure 20 and 21 respectively. If the user wishes, he can view both of these overlapped. This is the third option. Figure 22 shows an example of an overlapping graph.



Service Life: 50 years
T (RI = 1.0): 14.72367
T (RI = 1.5): 11.37536
T (RI = 2.0): 8.945869
T (RI = 3.0): 5.367914
T (RI = 4.0): 3.852663

Figure 21 – Reliability Index versus time curve.



Service Life: 50 years
T (RI = 1.0): 14.72367
T (RI = 1.5): 11.37536
T (RI = 3.0): 5.367914
T (pf = 10%): 12.18042
T (pf = 90%): ---

Figure 22 – Probability of Failure and Reliability Index versus time curve.

5. PRACTICAL APPLICATIONS

5.1 General

In order to demonstrate how the DURACON Software can be applied, some typical results of a probability-based durability design are given in the following. For a new concrete harbour structure, the overall requirement for safety and durability was based on a maximum risk level of 10 % for steel corrosion to occur during a service period of at least 100 years [7]. At the same time, the minimum durability requirements according to the current concrete code with respect to w/s ratio, cement content and concrete cover also had to be fulfilled [10].

As part of the durability design, two different cases of durability analysis are given:

- In the first case, the objective was to select a suitable type of cement or binder system in order to produce a concrete with adequate resistance against chloride penetration (chloride diffusivity).
- In the second case, the objective was to find out how an increased concrete cover in the beams of the concrete deck would improve the risk level for corrosion to occur.

5.2 Effect of cement type

Based on three possible types of cement with and without addition of silica fume, some concrete test mixtures were produced in order to test the chloride diffusivity [9]. The three types of cement included a high-performance portland cement (CEM I 52.5 LA), a blended fly-ash cement (CEM IV/A) and a blast-furnace slag cement with approximately 70 % slag (CEM III/B 42.5 LH HS). The test mixtures without silica fume had a cement content of 420 kg/m^3 and a w/c ratio of 0.45. The test mixtures with silica fume had a cement content and silica fume content of 390 kg/m^3 and 39 kg/m^3, respectively, while the w/c ratio was 0.38 and 0.35 for k-factors of 1 and 2, respectively.

For all the concrete mixtures, the durability analysis was based on the same concrete cover and surface chloride concentration (exposure conditions). The average value of concrete cover was 50 mm assuming a normal distribution and a standard deviation of 5 mm, while the average value of surface chloride concentration was 5.4 % by weight of cement, also assuming a normal distribution with a standard deviation of 0.54 %.

Based on the obtained chloride diffusivities and input parameters as shown in Table 2, the durability analysis was carried out with the results shown in Figure 23. For the pure portland cement (Type 1), a service period of less than 5 years would be obtained, while a combination with 10 % silica fume (Type 2) would give an increased service period of approximately 10 years before the risk level of 10 % for corrosion to occur would be reached. Based on the slag cement without (Type 3) and with silica fume (Type 4), service periods of approximately 30 years and more than 100 years, respectively, would be reached. For the fly-ash cement (Type 5), a service period similar to that of Type 2 would be reached, while a combination with 10 % silica fume would give a service period of approximately 25 years. These results are roughly in accordance with practical experience [11].

Table 2 - Input parameters for analyzing the effect of cement type [7].

Concrete Quality	Durability Parameters		
	D_{28d} (e-12.m^2/s)	α	c_{CR} (% wt. cement)
Type 1 (CEM I 52.5 LA)	N(10.5;0.66) 1)	N(0.37;0.07)	N(0.48;0.15)
Type 2 (CEM I 52.5 LA + 10% CSF)	N(4.74;0.51)	N(0.39;0.07)	
Type 3 (CEM III/B 42.5 LH HS)	N(5.3;0.59)	N(0.60;0.15)	
Type 4 (CEM III/B 42.5 LH HS + 10% CSF)	N(1.42;0.09)		
Type 5 (CEM IV/A)	N(10.1;0.81)	N(0.51;0.07)	
Type 6 (CEM IV/A + 10% CSF)	N(5.5;0.86)		

1) Normal distribution

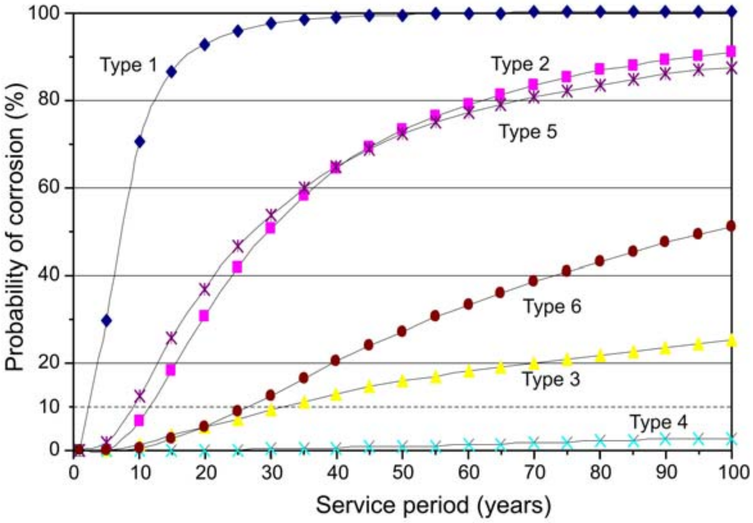
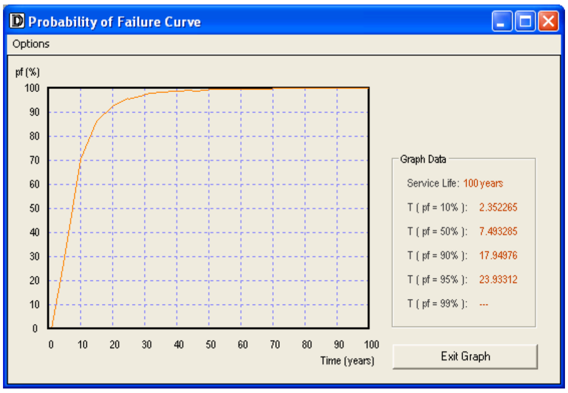


Figure 23 - Effect of cement type on probability of corrosion in a concrete harbour structure [7].

The above example demonstrates how important a proper selection of cement type is in order to obtain a concrete with adequate resistance against chloride penetration (chloride diffusivity) and thus adequate safety against steel corrosion.

For the above example, the slag cement in combination with silica fume would fulfill the overall requirement to safety and durability without any additional protective measures. If it would not be possible to produce such a concrete quality, however, other strategies and protective measures for ensuring proper durability should be considered.

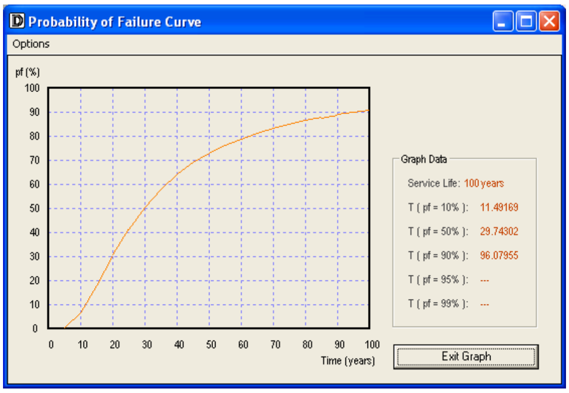
The results as obtained from the DURACON Software for the six concrete mixtures are displayed in Figure 24. Since it was convenient to compare all these graphs in the same figure (Figure 23), however, all the data files were converted into new graphs by use of MS Excel as described in Appendix.



Graph Data

Service Life: 100 years
T (pf = 10%): 2.352265
T (pf = 50%): 7.493285
T (pf = 90%): 17.94976
T (pf = 95%): 23.93312
T (pf = 99%): ---

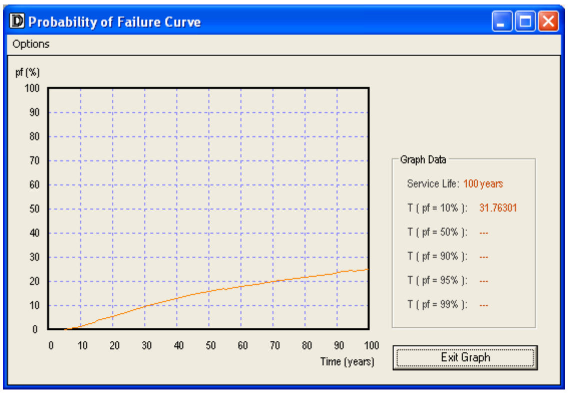
a) Type 1



Graph Data

Service Life: 100 years
T (pf = 10%): 11.49169
T (pf = 50%): 29.74302
T (pf = 90%): 96.07955
T (pf = 95%): ---
T (pf = 99%): ---

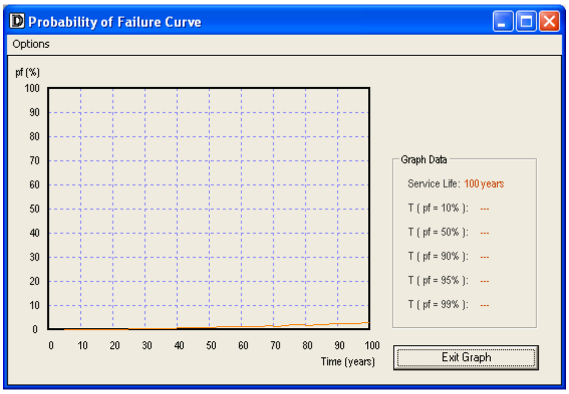
b) Type 2



Graph Data

Service Life: 100 years
T (pf = 10%): 31.76301
T (pf = 50%): ---
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

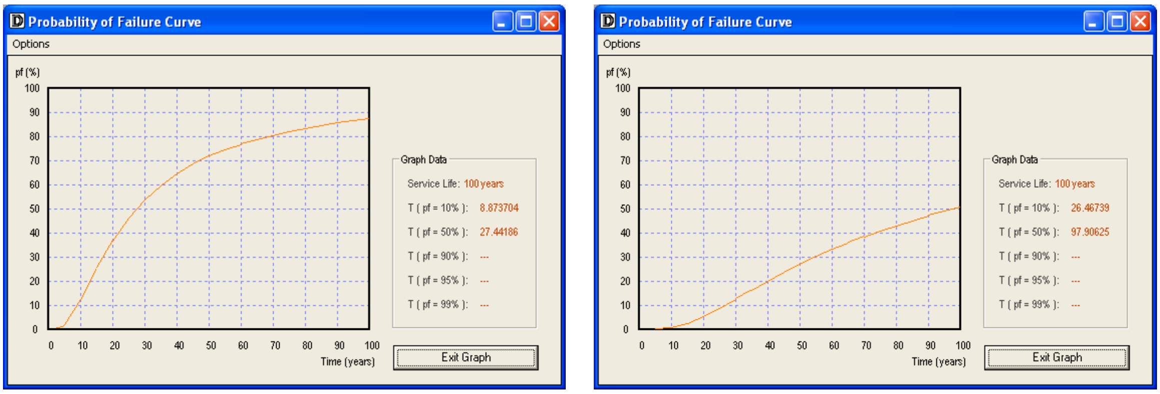
c) Type 3



Graph Data

Service Life: 100 years
T (pf = 10%): ---
T (pf = 50%): ---
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

d) Type 4



Service Life: 100 years
T (pf = 10%): 8.873704
T (pf = 50%): 27.44186
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

e) Type 5

Service Life: 100 years
T (pf = 10%): 26.46739
T (pf = 50%): 97.90625
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

f) Type 6

Figure 24 - Graph results from the DURACON Software for the six concrete mixtures.

5.3 Effect of concrete cover

Based on the same data as for Concrete Mixture Type 2 in the above example (portland cement with 10 % silica fume) (Table 3), a new durability analysis in order to find out the effect of increased concrete cover above the minimum requirement of 50 mm was carried out (Figure 25)

Table 3 - Input parameters for analyzing the effect of concrete cover [7].

Parameter	Average	Standard deviation	Comments
D_0	4.74	0.51	Chloride diffusivity (10E-12 m^2/sec)
α	0.39	0.07	Time dependence
C_{CR}	0.48	0.15	Critical chloride content (% by wt. cement)
C_S	5.4	0.54	Surface chloride content (% by wt. cement)
x_C	50	5	Concrete cover (mm)
	60	5	
	70	5	
	90	5	
	120	5	

As can be seen from Figure 25, the concrete cover is also of great importance for the probability for corrosion. While a nominal concrete cover of 50 mm only would give a service period of approximately 10 years, an effective, nominal concrete cover of 120 mm would give a service period of more than 100 years.

Since it may be difficult to significantly increase the concrete cover in the lower part of the deck beams beyond 70 mm, only a partial replacement of the conventional reinforcement with stainless steel reinforcement could be used in order to obtain a substantial increase of the effective concrete cover beyond 70 mm.

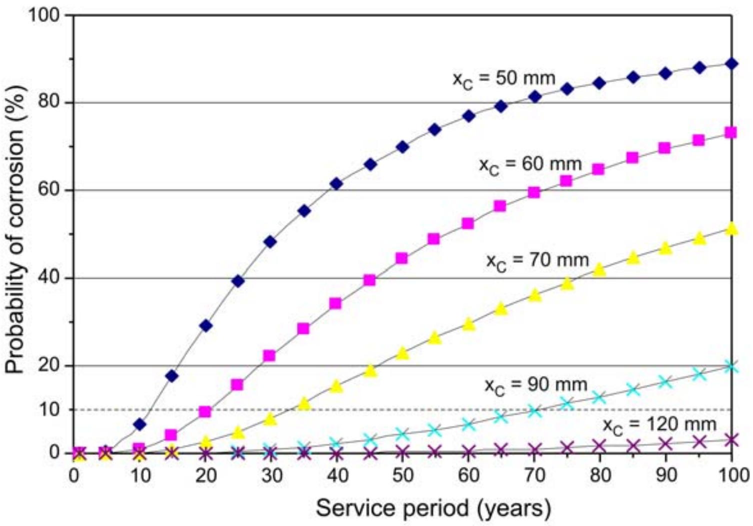
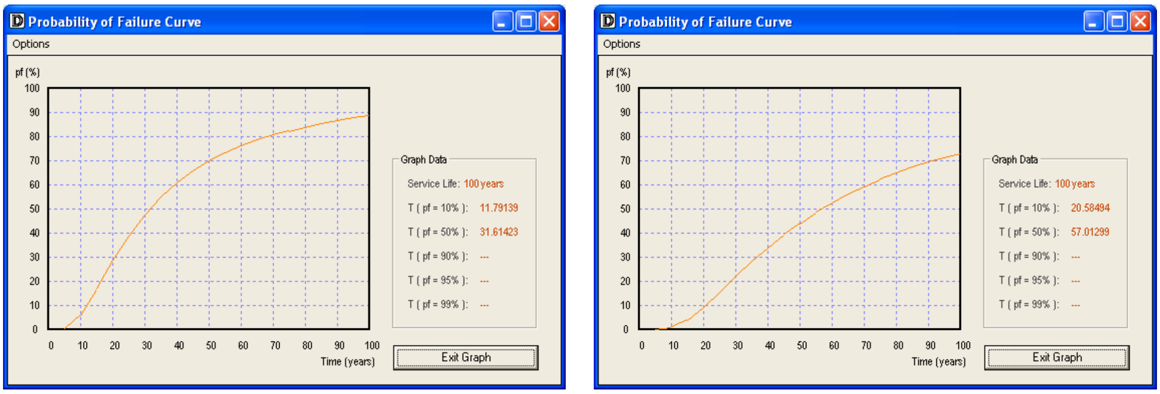


Figure 25 - Effect of concrete cover on the probability of corrosion in a concrete harbour structure [7].

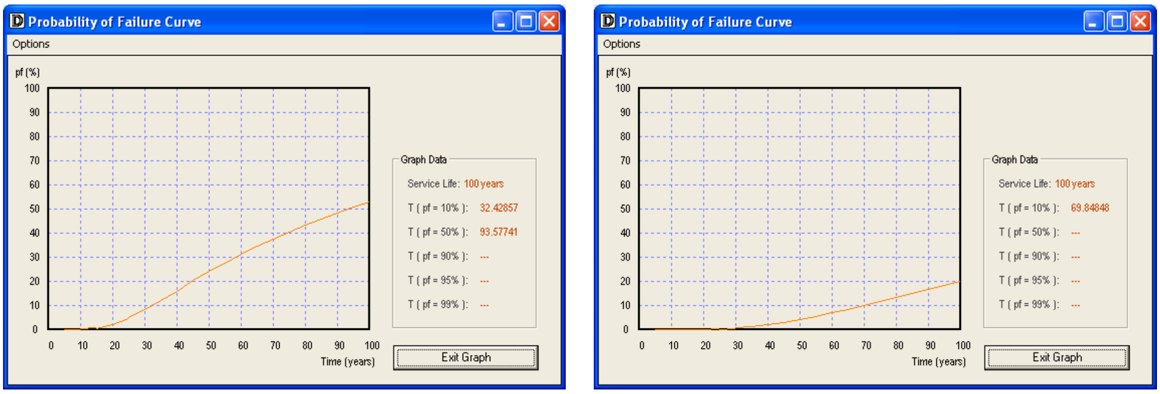
The results as obtained from the DURACON Software for the five depths of concrete cover are displayed in Figure 26. In order to obtain Figure 25, also here all the data files were converted into new graphs by use of MS Excel as described in Appendix.



Graph Data
Service Life: 100 years
T (pf = 10%): 11.79139
T (pf = 50%): 31.61423
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

Graph Data
Service Life: 100 years
T (pf = 10%): 20.58494
T (pf = 50%): 57.01299
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

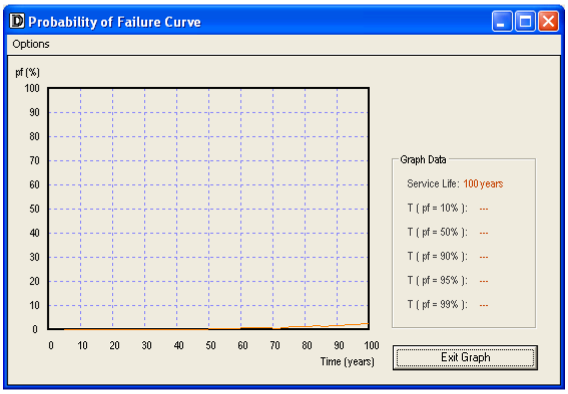
a) 50 mm *b) 60 mm*



Service Life: 100 years
T (pf = 10%): 32.42857
T (pf = 50%): 93.57741
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

Service Life: 100 years
T (pf = 10%): 69.84848
T (pf = 50%): ---
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

c) 70 mm *d) 90 mm*



Service Life: 100 years
T (pf = 10%): ---
T (pf = 50%): ---
T (pf = 90%): ---
T (pf = 95%): ---
T (pf = 99%): ---

e) 120 mm

Figure 26 - Graph results from DURACON for the four concrete types.

6. REFERENCES

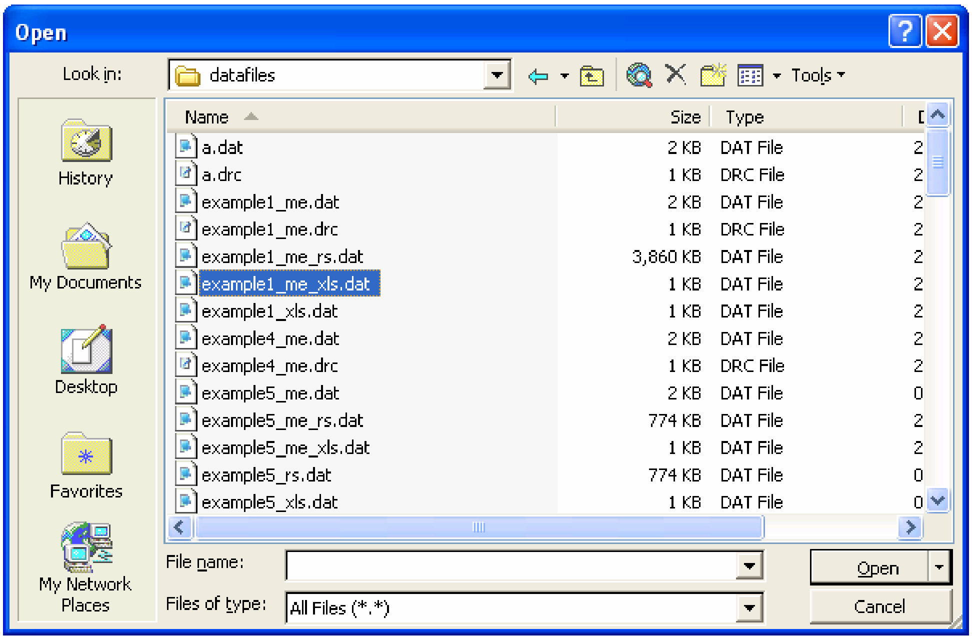
1. "DuraCrete: General Guidelines for Durability Design and Redesign", The European Union – Brite EuRam III, Project No. BE95-1347: "Probabilistic Performance Based Durability Design of Concrete Structures", Document R 15, February 2000, 109 p.
2. Gehlen, C. and Schiessl, P., "Probability-Based Durability Design for the Western Scheldt Tunnel", Structural Concrete, No. 2, 1999, pp. 1-7.
3. McGee, R., "Modelling of Durability Performance of Tasmanian Bridges", Proceedings, The 8th International Conference on the Application of Statistics and Probability, Sydney, Australia, 1999.
4. Ferreira, M., Årskog, V. and Gjørv, O.E., "Probability-Based Durability Analysis of Concrete harbor Structures", Proceedings, Fourth International Conference on Concrete Under Severe Conditions, Environment and Loading", Seoul National University, Korea Concrete Institute, Seoul, Vol. 2, 2004, pp. 999-1006.
5. Årskog, V., Ferreira, M. and Gjørv, O.E., "Durability and Performance of Norwegian Concrete Harbor Structures", Proceedings, Fourth International Conference on Concrete Under Severe Conditions, Environment and Loading", Seoul National University, Korea Concrete Institute, Seoul, Vol. 1, 2004, pp. 133-140.
6. Årskog, V., Ferreira, M. and Gjørv, O.E., "Durability Design and Performance of Concrete Barges", Proceedings, Fourth International Conference on Concrete Under Severe Conditions, Environment and Loading", Seoul National University, Korea Concrete Institute, Seoul, Vol. 2, 2004, pp. 1604-1611.
7. Årskog, V., Ferreira, M. and Gjørv, O.E, "Practical Guidelines for Probability-Based Durability Design of Concrete Structures in Severe Environment", Report BML 200403, Department of Structural Engineering, Norwegian University of Science and Technology, NTNU, Trondheim, Norway, 2004.
8. Takewaka, K. and Mastumoto, S., "Quality and Cover Thickness of Concrete based on the Estimation of Chloride Penetration in Marine Environments", ACI SP 109-17, American Concrete Institute, 1988, pp. 381-400.
9. Nordtest method NT Build 492, "Concrete, Moartar and Cement Based Repair Materials: Chloride Migration Coefficient from Non-Steady State Migration Experiments", 1999, 8 p.
10. EN 206-1 "Concrete – Part 1: Specification, properties, production and conformity", 2000.
11. Gjørv O.E., "Durability and Service Life of Concrete Structures", Proceedings, The First fib Congress 2002, Session 8, V. 6, Japan Prestressed Concrete Engineering Association, Tokyo, 2002, pp. 1-16.

APPENDIX

CONVERSION OF DATA FILES INTO NEW GRAPHS BY USE OF MS EXCEL

As shown in the Software Manual, the final results of the durability analysis are always displayed in the form of a single graph. In order to obtain a better basis for comparison of the graphs from several analyses, it may be convenient to show all the graphs together in the same figure. This can be done by converting all the data files from DURACON into new graphs by use of MS Excel as described in the following.

1. In MS Excel, open a file and set the file type to *All Files (*.*)*



Look in: datafiles

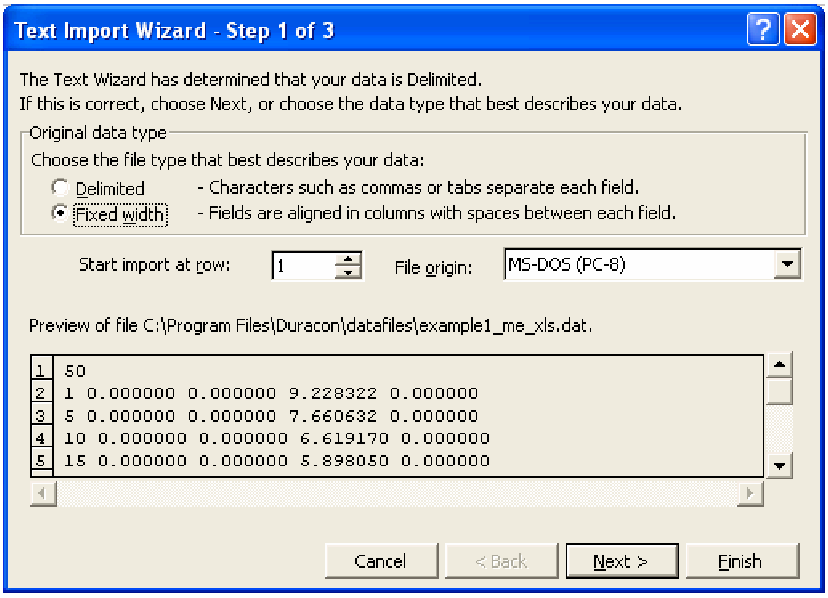
Name	Size	Type
a.dat	2 KB	DAT File
a.drc	1 KB	DRC File
example1_me.dat	2 KB	DAT File
example1_me.drc	1 KB	DRC File
example1_me_rs.dat	3,860 KB	DAT File
example1_me_xls.dat	1 KB	DAT File
example1_xls.dat	1 KB	DAT File
example4_me.dat	2 KB	DAT File
example4_me.drc	1 KB	DRC File
example5_me.dat	2 KB	DAT File
example5_me_rs.dat	774 KB	DAT File
example5_me_xls.dat	1 KB	DAT File
example5_rs.dat	774 KB	DAT File
example5_xls.dat	1 KB	DAT File

File name:

Files of type: All Files (*.*)

2. Select the output data file with the extension **_xls.dat*, and open it.

3. The *Text Import Wizard* opens up because the file is not a generic MS Excel file. Chooses *Delimited*, and press *Next*.



The Text Wizard has determined that your data is Delimited.
If this is correct, choose Next, or choose the data type that best describes your data.

Choose the file type that best describes your data:

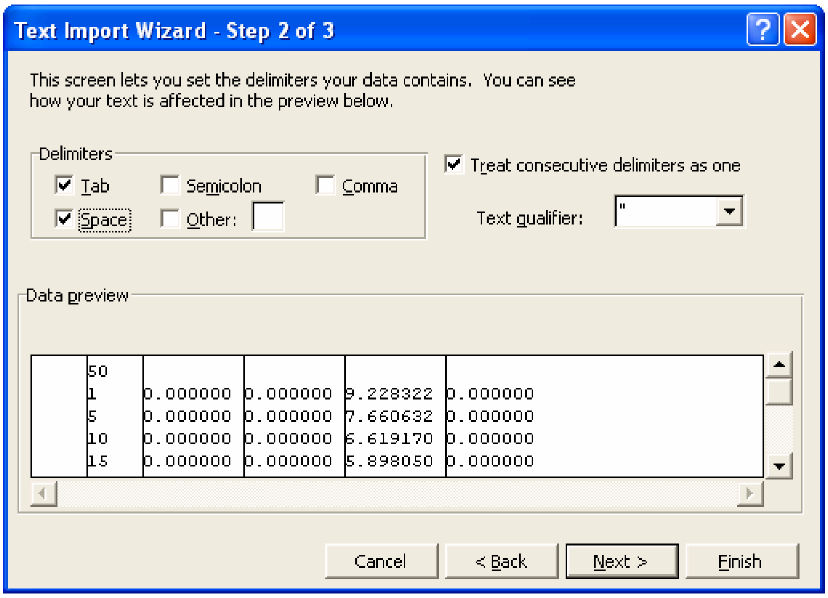
○ Delimited - Characters such as commas or tabs separate each field.
● Fixed width - Fields are aligned in columns with spaces between each field.

Start import at row: 1 File origin: MS-DOS (PC-8)

Preview of file C:\Program Files\Duracon\datafiles\example1_me_xls.dat.

```
1 50
2 1 0.000000 0.000000 9.228322 0.000000
3 5 0.000000 0.000000 7.660632 0.000000
4 10 0.000000 0.000000 6.619170 0.000000
5 15 0.000000 0.000000 5.898050 0.000000
```

4. Activate the *Space* option in *Delimiters* and press *Finished*.



This screen lets you set the delimiters your data contains. You can see how your text is affected in the preview below.

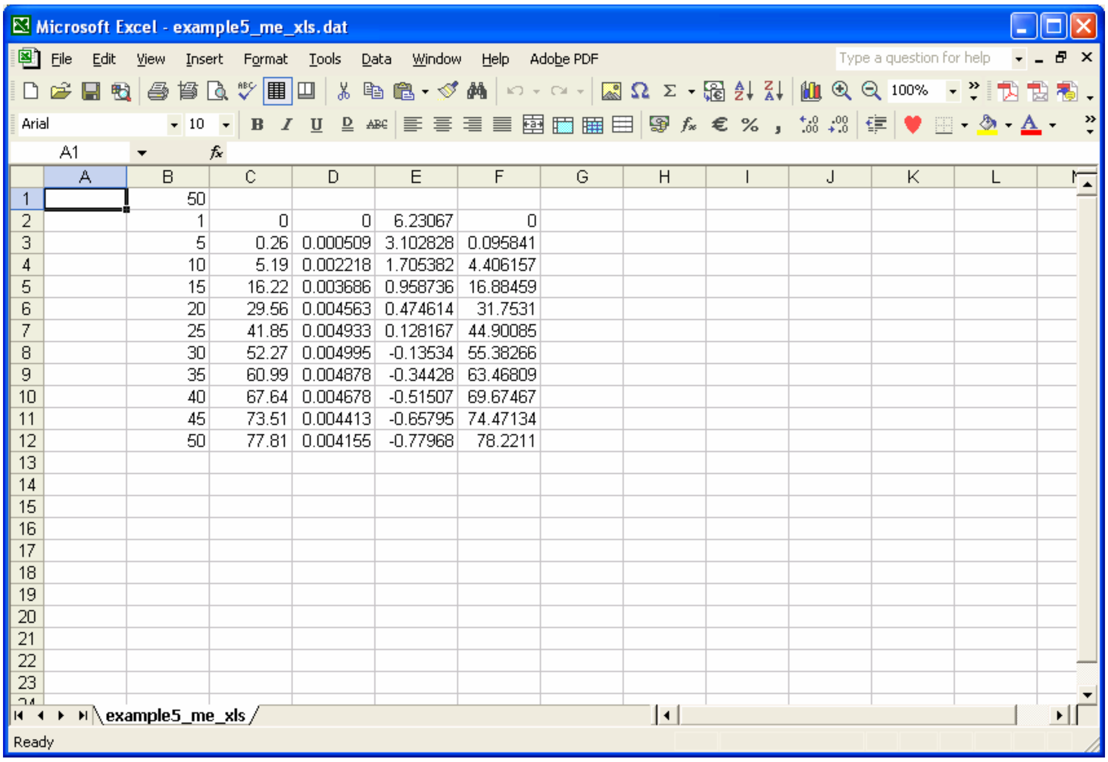
☑ Tab ☐ Semicolon ☐ Comma
☑ Space ☐ Other:

☑ Treat consecutive delimiters as one

Text qualifier: "

```
50
1    0.000000  0.000000  9.228322  0.000000
5    0.000000  0.000000  7.660632  0.000000
10   0.000000  0.000000  6.619170  0.000000
15   0.000000  0.000000  5.898050  0.000000
```

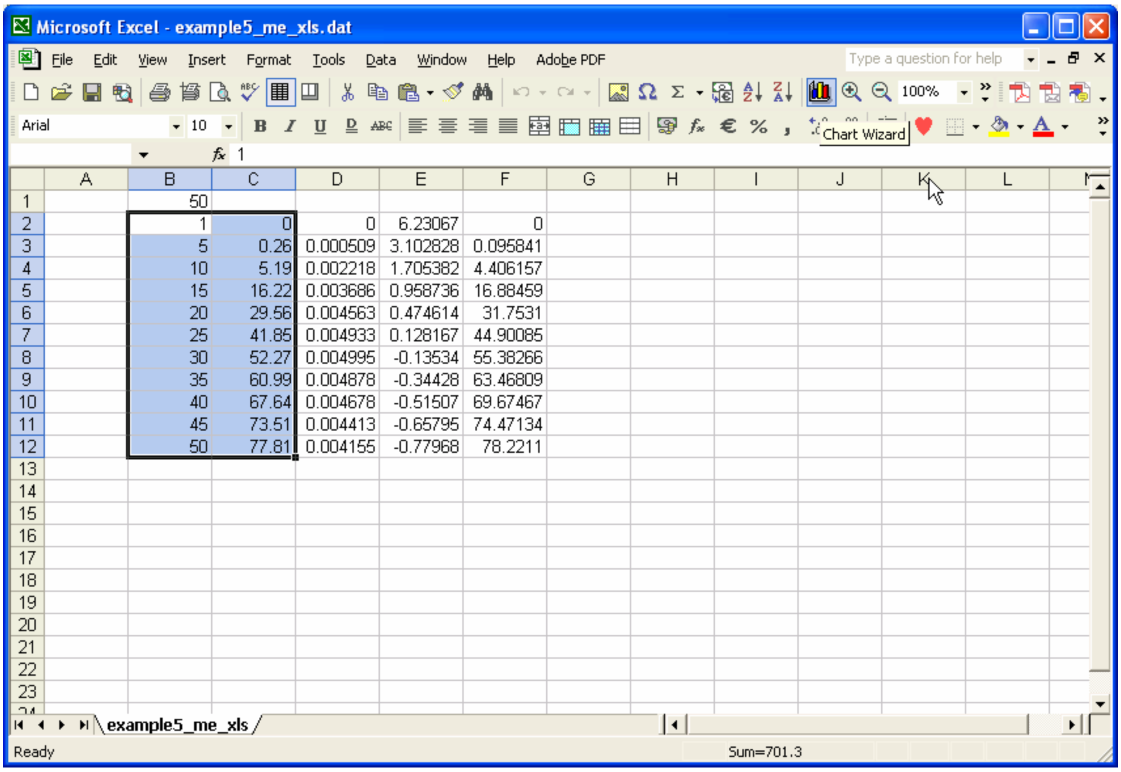
5. The output data file is then converted into MS Excel.



	A	B	C	D	E	F
1		50				
2		1	0	0	6.23067	0
3		5	0.26	0.000509	3.102828	0.095841
4		10	5.19	0.002218	1.705382	4.406157
5		15	16.22	0.003686	0.958736	16.88459
6		20	29.56	0.004563	0.474614	31.7531
7		25	41.85	0.004933	0.128167	44.90085
8		30	52.27	0.004995	-0.13534	55.38266
9		35	60.99	0.004878	-0.34428	63.46809
10		40	67.64	0.004678	-0.51507	69.67467
11		45	73.51	0.004413	-0.65795	74.47134
12		50	77.81	0.004155	-0.77968	78.2211

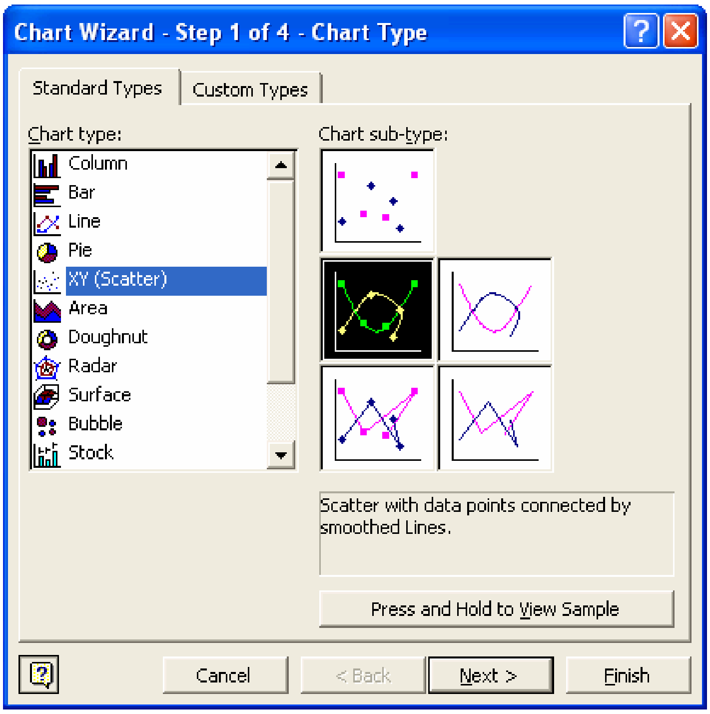
6. In the first line, the time frame for the simulation is shown. The columns of data are from left to right: Time (years), Probability of Failure (%), Standard Error, Reliability Index, Probability of Failure calculated with Reliability Index (%).

7. Chose the columns of data to place into a graph and activate the *Chart Wizard*.



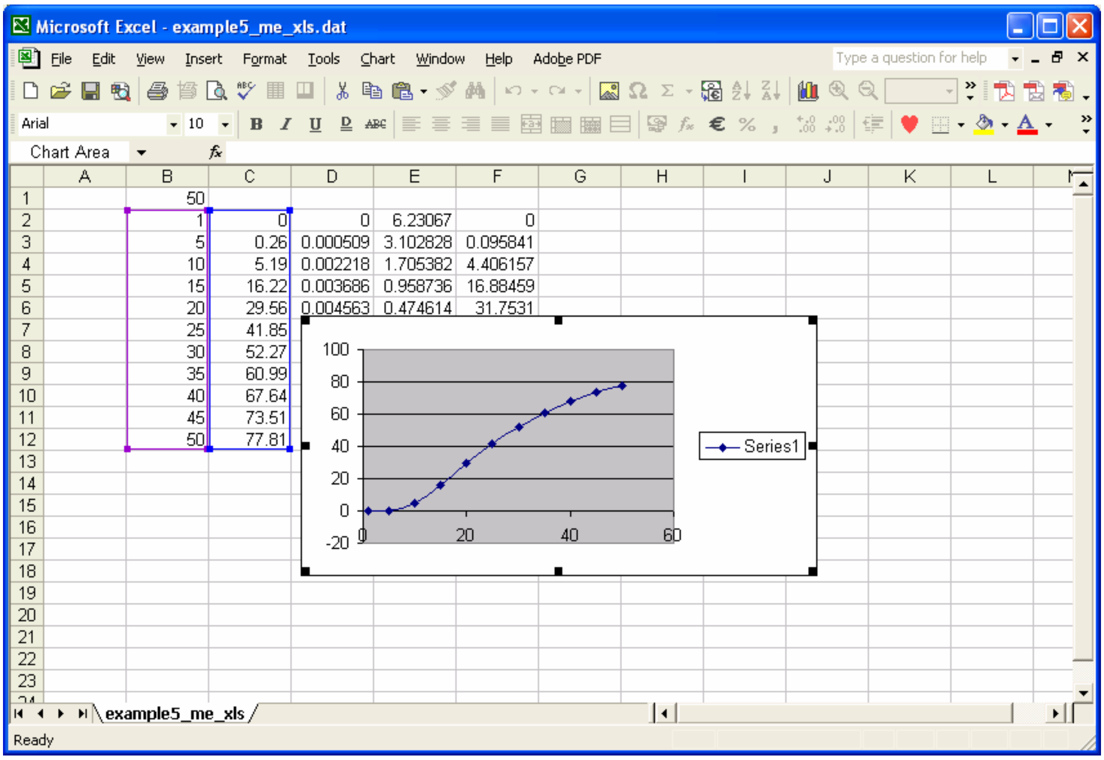
	A	B	C	D	E	F
1		50				
2		1	0	0	6.23067	0
3		5	0.26	0.000509	3.102828	0.095841
4		10	5.19	0.002218	1.705382	4.406157
5		15	16.22	0.003686	0.958736	16.88459
6		20	29.56	0.004563	0.474614	31.7531
7		25	41.85	0.004933	0.128167	44.90085
8		30	52.27	0.004995	-0.13534	55.38266
9		35	60.99	0.004878	-0.34428	63.46809
10		40	67.64	0.004678	-0.51507	69.67467
11		45	73.51	0.004413	-0.65795	74.47134
12		50	77.81	0.004155	-0.77968	78.2211

8. Select *XY (Scatter)* and the *Scatter with data points connected by smoothed lines option*. Then press *Finish*.



Scatter with data points connected by smoothed Lines.

9. The graph can now be edited and more data added to it.



	A	B	C	D	E	F
1		50				
2		1	0	0	6.23067	0
3		5	0.26	0.000509	3.102828	0.095841
4		10	5.19	0.002218	1.705382	4.406157
5		15	16.22	0.003686	0.958736	16.88459
6		20	29.56	0.004563	0.474614	31.7531
7		25	41.85			
8		30	52.27			
9		35	60.99			
10		40	67.64			
11		45	73.51			
12		50	77.81			

APPENDIX 3
RECOMMENDATIONS FOR PROBABILITY-BASED DURABILITY DESIGN OF CONCRETE STRUCTURES

Recommendations for probability-based durability design of concrete structures

Miguel Ferreira

Contents

1. Introduction

Durability is one of the most important issues in construction, as it influences sustainability, service life, cost of repair and refurbishment and environmental impact of the building materials, components and structures.

As a result of extensive research, much progress has been made on modelling of degradation processes and the performance of materials. However, it is still difficult to consider more than one degradation process at a time. In reality, deterioration is generally the result of a complex combination of processes. As a result of this and the inherent variability of concrete, modelling should not be expected to produce exact prediction of service life. Nevertheless, for concrete structures in marine environment, the predominant degradation mechanism is considered to be chloride corrosion.

Most of the degradation factors affecting concrete and its reinforcement are time dependent in their effect. It is somewhat surprising, therefore, that few standards or codes of practice for design of concrete structures currently contain any indication of the service life that can be expected from those structures.

This document provides basic recommendations for durability design of concrete structures in marine environment. The probabilistic durability design is based on the DURACON software.

2. Designing for durability – General considerations

The design considerations relevant to corrosion protection depend on the type of structure and, to a significant degree, its environment and intended use. Certain minimum measures, for example, adequate concrete cover and concrete quality, should always be specified. Depending on the type of structure and its expected exposure, however, additional design considerations should be required to ensure satisfactory performance over the intended service life of the structure.

The current approach in most codes of practice and standards for design of concrete structures is to provide a set of prescriptive requirements for various parameters controlling the quality of the concrete related to the exposure conditions and the cover to reinforcement. No indication of the service life that might be expected is given and no advice is provided on how to vary the prescriptions to meet a particular required service life. The designer should make an assumption of the expected service life when using any particular code. This is an undesirable situation which is conducive to the needs of neither the designer nor the client.

Durability design helps to improve the control of durability over the required service life. A recommended durability design procedure is shown in Figure 1.

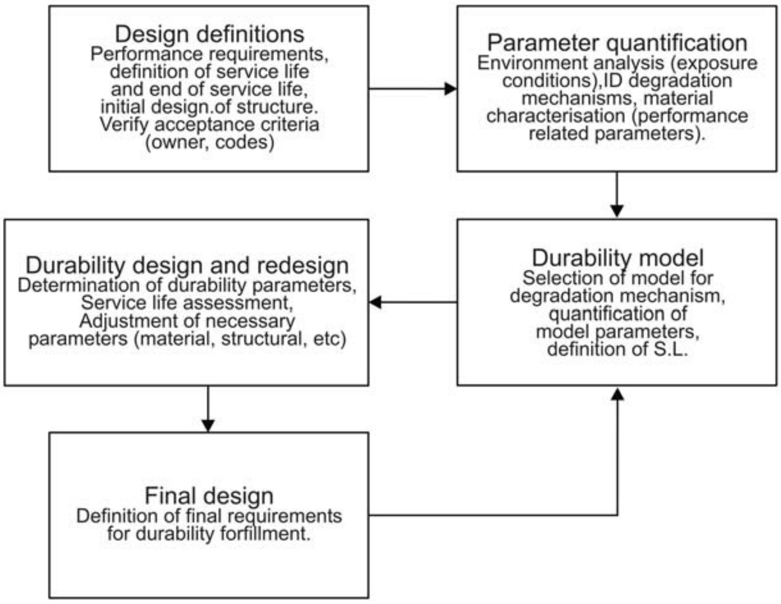


Figure 1 - Systematic methodology for service life prediction of building components.

Concrete structures, such as docks, piers, and storage tanks, located in a marine environment are vulnerable to chloride-induced corrosion. Chloride ions and other ions in seawater can penetrate the concrete. Because both water and oxygen must be available for electrochemical corrosion to occur, that portion of a marine concrete structure located in the tidal and splash zones is generally the most susceptible to corrosion. All segments of a marine structure, however, are at risk for chloride-induced corrosion, but low oxygen concentrations significantly reduce corrosion rates in submerged portions.

The owner, by defining his present and possible future demands and wishes, should define the project brief including the required service life and acceptable level of maintenance within agreed budget limits, and the level of maintenance and monitoring of the structure.

The designer, by preparing design specifications (including proposed quality control schemes) and conditions, must identify exposure conditions, develop a practical strategy for resisting these exposure conditions and quantify concrete quality and cover needed to resist these conditions (information obtained from standards or from the design models), and the type of monitoring of structure, if delegated by the client.

Rostam and Schiessl (1996) sum up the responsibility of clients in that they must ask for quality and long service life (i.e. specify required service life), check the quality and performance of the structure received, and, importantly, be willing to pay for that quality.

3. Recommendation for durability design

These recommendations are for the durability analysis of concrete structures in marine environment. Therefore, it is assumed that the main degradation phenomenon is the corrosion of the reinforcement.

3.1 Basic background information

The starting point for a durability design is to collect the background information. This information is the basis for the durability analysis. Once this information has been collect, the designer can proceed with an initial durability analysis.

The information can be divided into three main groups:

- the environment;
- the structure;
- the material (concrete).

3.3.1 Analysing the environment

The analysis of the environment is crucial as it will provide important information for the durability analysis. A few pertinent observations are made with regards to the environment:

- Define the position of the structure in relation to the sea (distance from the sea; parts of structure exposed directly to the sea);
- Identify main wind directions (to identify surfaces exposed to rain);
- Study tidal variations (height of structure in relation to the sea);
- Quantify expected chloride concentration (for example on neighbouring structures).

3.3.2 Analysing the structural design

The analysis of the structural design provides information regarding the physical protection available. Basic information that must be collect is:

- Identify the required service life, and limit state (event) that defines it’s conclusion;
- Identify structural elements according to the environmental exposure classes;
- Identify structural surfaces according the shelter from the predominant wind/rain directions;
- Identify the reinforcement concrete cover;
- Identify reinforcement type (steel, stainless steel, etc.);

3.3.3 Analysing the concrete properties

The analysis of the concretes properties provides information regarding the quality of the concrete to be used. The information required is:

- Identify concrete grade;
- Identify water/binder ratio;
- Identify binder type and content;

4. Recommendations for definition of probabilistic parameters

Once the basic project information is defined, and based on the DURACON software for a probabilistic-based durability design of concrete structures in a marine environment, several other parameters also require definition.

The parameters that need defining can be divided into three groups, similar to those present in the previous chapter:

- Environmental loading parameters;
- Structural parameters, and
- Concrete quality parameters.

4.1 Environmental loading parameters

The environmental loading parameters are the chloride concentration on the surface of the concrete structure as a result of the contact with the marine environment, and the average annual temperature.

4.1.1 Surface chloride concentration

The surface chloride concentration depends, among other things, mainly on the location of the structure and its orientation in relation to the predominant winds and rain. It is well known that in a structure, the variation of the surface chloride concentration varies from element to element, and even on different surfaces of the same element.

If there are structures in the vicinity in an identical situation, the measurement of the surface chloride concentrations on these structures will help to quantify this parameter. Care should be taken to choose those elements that are likely to have higher surface chloride concentrations due to their location (closer to the tidal zone) and due to their sheltering (no exposure to rain). These measurements should also be accompanied by information on the type of cement and concrete mix, if possible.

Not always will it be possible to measure neighbouring structures to obtain the value of surface chloride concentration. In these cases, some empirical expressions are available to estimate an initial value. A basis for the estimation of C_S can be found in (HETEK 83 1997, Breit 1997, Fluge 2000).

If measuring directly from neighbouring structures, sufficient measurements should be made to determine the scatter of the parameter. If the value is determined empirically, a CoV (coefficient of variation) of 10 % is considered to be appropriate.

4.1.2 Temperature

The temperature is introduced as a yearly average. If it is possible to determine the average value and the scatter associated with it, temperatures below zero should not be considered as being zero, since the ionic activity once the temperature is reduced to zero is practically halted.

4.2 Structural parameters – Concrete cover

The only structural parameter is the concrete cover. The concrete cover is the distance between the surface of the concrete and the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant).

The concrete cover should be chosen keeping in mind the required service life and the environmental aggressivity. The EC2 (2001) classifies the concrete cover according to the environmental exposure. In this standard, the concrete cover is the sum of a minimum value and a deviation due to several factors such as poor workmanship, etc.

The nominal cover should be specified in the design project. It is defined as a minimum cover, c_{MIN}, plus an allowance in design for deviation, Δc_{DEV} (Eurocode 2 - 2002). Due to the quality of the workmanship, the placement of the concrete and other factors, a standard deviation of between 5 and 15 mm can be adopted. The value of x_{C} is assumed to be normally distributed.

4.3 Concrete quality parameters

Cement type (and cement content) influence the durability properties such as the diffusion coefficient, absorption, permeability, etc. The concrete quality parameters are the chloride diffusion coefficient, the age dependency of the diffusion coefficient and the critical chloride concentration.

4.3.1 Chloride diffusion coefficient

The chloride diffusion coefficient is the basic parameter that quantifies the concrete quality.

In some cases, it is possible to test the concrete mix to be used in the structure previously. Once the cement type has been chosen and the concrete mix defined, specimens should be tested to determine the value of the diffusion coefficient. In these situations, not only can the average diffusion coefficient be determined, but also the scatter. If possible the specimens should be cured in conditions that simulate the actual environment.

When the above situation is not possible, then the use of empirical equations to estimate the diffusion coefficient should be used (HETEK 53, 1996).

The diffusion coefficient is usually considered to be normally distributed. However, in some situations, to avoid the possibility of negative values, the lognormal distribution is also used. When the diffusion coefficient is determined empirically, a CoV of 10 % is admissible.

4.3.2 Age dependency of the diffusion coefficient

The age dependency of the diffusion coefficient is the most difficult parameter to determine. The main factors that affect the age dependency of the diffusion coefficient are the same as those that affect the diffusion coefficient.

Values for the age dependency of the diffusion coefficient are presented in the Duracrete Project report (1998).

4.3.3 Critical chloride concentration

The critical chloride concentration is the minimum total chloride content, or the chloride threshold level, required for corrosion to initiate. It is dependent on a number of factors such as the water / cement ratio, the cement type, the pH of the pore solution, the capacity of the cement paste to bind the chloride, etc. It is therefore clear that it cannot be represented by a single value.

Literature threshold levels suggest a realistic range for initiation conditions of pitting corrosion, between total chloride contents of 0.20 and 0.80 % by weight of binder (Browne 1982, Breit 1997, Hansson 1990, Frederiksen 2002). A value suggested in literature (Duracrete 2000) for a probabilistic evaluation is a normally distributed concentration, with average of 0.48 and a standard deviation of 0.15 % chloride by weight of binder, for an OPC concrete with about 350 kg/m^3.

5. Durability design update

The knowledge about the service life of a structure is uncertain due to the random variation of the geometry, material characteristics, execution and environment. This random variation can be assessed and in part controlled by testing and quality control at several stages during the construction of a structure.

However, during the initial durability design, most parameters are based on assumptions, tabled values or empirically determined. Once the structure is actually under construction, realistic information concerning the parameters can be obtained. The aim of the tests and the quality control is to define and to update the distribution parameters of these variables.

During the period of use, where the relevant parameters can be tested during inspection and maintenance activities, and the time dependency can be determined. The information obtained during the period forms the basis of an assessment of the performance of the structure, and can, if necessary, be used to determine a repair strategy for the structure.

From a durability design point of view this means that design input data with different levels of accuracy must be used at the different stages:

- At the design stage, where the specific materials, the quality of execution, and the structure-environment interaction are all unknown. Preliminary assumptions of environmental aggressivity must be made. Codes and design guides can provide a first set of data. These assumptions are some of the most important ones to be made by the designer.
- At the construction stage, where the specific materials can be tested and the quality of execution can be tested. All the assumptions made can be measured and realistic values are obtained for the durability analysis.
- At the handing over stage, where the "as-built" conditions of the structure can be tested in situ.

APPENDIX 4
INSTALLATION CD

A4.1 Contents of installation CD

The installation CD has the following contents (directories):

- install (directory with the setup files for the DURACON software);
- structure 1 (with the data and result files from the durability analysis of structure 1);
- structure 2 (with the data and result files from the durability analysis of structure 2);
- structure 3 (with the data and result files from the durability analysis of structure 3);
- structure 4 (with the data and result files from the durability analysis of structure 4).