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ORIGINAL ARTICLE

Durability design of reinforced concrete structures: a comparison of the use of durability indexes in the deemed-to-satisfy approach and the full-probabilistic approach

R. Muigai · P. Moyo · M. Alexander

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Abstract To show the application of the chloride conductivity index test in service life prediction (SLP) using both the deemed-to-satisfy and probabilistic approaches to performance-based durability design. It is desirable to adopt a performance-based approach with respect to durability design of reinforced concrete (RC) structures. This is based on the perception that the durability of RC is achieved when the limiting value from an established test method is met. In South Africa, the durability index (DI) approach has been developed, which permits performance-based specifications for durability of RC. This approach involves the application of a test method together with a SLP model. This integrated approach links material properties directly with the expected service life of RC structures and environmental conditions. Two DIs are relevant to degradation processes in RC: the chloride conductivity index which is related to chloride ingress, and the oxygen permeability index related to carbonation. The study presented here focuses on the application of the chloride conductivity index as the main input parameter of a SLP model concerned with chloride-induced reinforcement corrosion. The methodology and output of the SLP model as applied in the deemed-to-satisfy approach are compared with those of the probabilistic approach. Both approaches are exemplified using a concrete pier cast in situ in a marine environment. The performance-based durability specifications from the deemed-to-satisfy approach are found to be more conservative compared to those of the probabilistic approach.

Keywords Durability design · Durability indexes · Reinforced concrete

Mathematics Subject Classification (2000) 65C05

Abbreviations

P_f	Probability of failure
$\beta_{target,ILS}$	Target reliability at initiation limit state
$P_{target,ILS}$	Limiting probability of failure at initiation limit state
D_i	Diffusion coefficient (2-years)
CCT	Chloride conductivity test
CDF	Cumulative density function
DI	Durability index
FORM	First order reliability methods
ILS	Initiation limit state
LSF	Limit state function
LSM	Limit states method
MCS	Monte-Carlo simulation
RC	Reinforced concrete
SLP	Service life prediction
SLS	Serviceability limit state
ULS	Ultimate limit state

R. Muigai (✉) · P. Moyo · M. Alexander
Department of Civil Engineering, University of Cape Town, Private Bag X3, Rondebosch 7701, South Africa
e-mail: rachel.muigai@uct.ac.za

1 Introduction

Durability design of reinforced concrete (RC) structures in adverse environments is most commonly concerned with ensuring the ability of the concrete to resist the penetration of aggressive agents during its intended service life. This largely involves controlling the quality and thickness of the cover layer protecting the reinforcement [28]. The cover layer is most susceptible to poor construction practices (such as poor curing and inadequate compaction) which in turn increases the penetration of aggressive agents from the environment. In South Africa (SA), durability indexes (DIs) have been adopted as engineering measures of the potential resistance of concrete cover to the transport of fluids and ions through concrete. The transport mechanisms considered are gas permeability, water sorptivity and chloride ion conductivity [2]. The DIs are derived from three test methods: the chloride conductivity test (CCT), where chloride ion resistance is important; the oxygen permeability test to establish carbonation resistance; and a water sorptivity test to examine concrete for surface water absorption. DIs characterize both the macrostructure and microstructure of the cover concrete and have been shown to be sensitive to material parameters such as binder type, processing influences such as type and degree of curing, and environmental influences such as temperature and relative humidity [4]. The DI tests also serve as valuable inputs to SLP models. SLP models allow the expected service life of a structure to be predicted based on considerations of environmental conditions, cover thickness and concrete quality. The environmental classes used are related to the EN 206-1 classes, modified for South African conditions (Table 1), while concrete quality is represented by the appropriate DI parameter. The oxygen permeability index is used in the carbonation SLP model, while the chloride model utilizes chloride conductivity to characterize material quality. The SLP models are used for both design and performance specification of new structures by allowing appropriate combinations of concrete quality (DI value) and cover for selected binders, that will provide sufficient resistance of the concrete to aggressive agents [1]. This integrated approach that links material properties with the expected service life of RC structures and environmental conditions is at present essentially a deemed-to-satisfy approach as put forth by the FIB model code (service life design) (fib Bulletin [9]. The performance-based deemed-to-satisfy approach is preferred in design in that it is based on calibrated results from SLP models and is not based solely on experience and simple rules-of-thumb as is the case in prescriptive approaches such as most current specifications, e.g., SANS 10100 [31]. The deemed-to-satisfy approach is deterministic in nature.

Table 1 Marine environmental classes (natural environments only) (after EN206-1 as cited in [3])

Corrosion induced by chlorides in water	
Designation	Description
XS1	Exposed to airborne salt but not in direct contact with seawater
XS2a[a]	Permanently submerged
XS2b[a]	XS2a + exposed to abrasion
XS3a[a]	Tidal, splash and spray zones
XS3b[a]	XS3a + exposed to abrasion

[a] These sub clauses have been added for South African coastal conditions

However, the high degree of variability that exists in the model parameters makes it difficult to predict the degree of deterioration of RC structures with certainty. This essentially calls for a probabilistic tool to account for uncertainties and variability in the physical and material parameters in the model. The study presented in this paper proposed the development of a probabilistic SLP model to take account of the range of possible values for each input parameter at the initiation limit state (ILS) of a RC structure. The probabilistic SLP model would be able to predict a range of expected times to corrosion initiation rather than a single value, so as to allow owners to make a more rational and accurate selection of durability parameters and economical decisions for a RC structure. It would thus assist in obtaining a balance of economy as well as safety of the concrete structure.

2 Service life prediction model

Much of South African infrastructure is situated in coastal areas and is thereby exposed to marine environments. The relatively harsh environment, heavy seas and prevailing onshore winds, coupled with inadequate attention to durability with regard to both design and construction, has resulted in premature deterioration of many RC structures in SA [19]. The main cause of deterioration is due to reinforcement corrosion induced by chlorides from seawater penetrating into the RC structure.

The life of a structure under reinforcement corrosion is marked by three stages, schematically presented in Fig. 1. The initiation stage corresponds to chloride penetration and accumulation of chloride ions in the vicinity of the rebar. Its duration depends on the quality and depth of the cover concrete and on the chloride concentration required to initiate the corrosion process [28]. The second stage (propagation) is that of active reinforcement corrosion and is marked by crack formation. The final stage involves extensive spalling and loss of steel cross-section, leading possibly even to structural collapse.

The service life of a structure can be defined with respect to the relevant limit state which for this study is the ILS marked by corrosion initiation and is defined as the time during which the structure is able to meet its specified durability requirements with an acceptable level of safety [28].

Owing to the problems associated with chloride-induced corrosion in RC structures, reliable prediction of chloride ingress into concrete and time to corrosion initiation, $T_{\text{initiation}}$, are the key elements for consideration in RC durability design [28]. So far, mathematical models developed to predict $T_{\text{initiation}}$ are mostly based on Fick's second law of diffusion, since the predominant transport process of chloride ions into saturated concrete is diffusion. The Fickian-type model applicable to SA marine environment and which was used in this study is represented by Eq. 1 [34].

$$C_{(x,t)} = C_s\left(1 - \text{erf}\left(\frac{x}{2\sqrt{D_i t^{(1-m)}}}\right)\right) \tag{1}$$

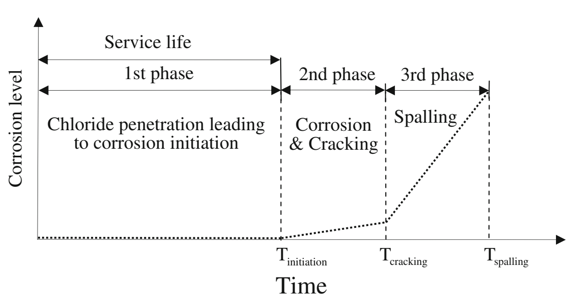

Fig. 1 The corrosion process (adopted from [16]

where $C_{x,t}$ is the chloride concentration at distance x(mm) from the exposed surface at a certain time t (*years*); C_s is the surface chloride concentration (%Cl^{-1} by mass of cement); D_i is diffusion coefficient at 2-years (mm^2/year), erf is the error function, and m is a coefficient representing the reduction in the diffusion coefficient with time due to chloride binding.

D_i may be determined by fitting Eq. 1 to chloride profiles obtained from existing structures or from test samples stored under desired conditions. However, the procedure is relatively slow and time consuming hence accelerated penetration or migration tests are used to obtain diffusion coefficients [21]. Accelerated tests have been developed worldwide. In SA, the CCT, shown in Fig. 2, is used to obtain a chloride conductivity value.

The CCT measures the resistance of a 30 ± 2 mm thick concrete sample (Ø 68 ± 2 mm) to chloride conduction under the action of an applied voltage (10 V) across the specimen. Prior to the test, the specimens are oven-dried at 50°C for 7 days following which they are vacuum saturated in a 5 M NaCl solution for 1 h ± 15 min, then soaked in a 5 M NaCl for a further 18 ± 1 h to ensure a saturated equilibrium. From the CCT a chloride conductivity value (σ) is determined from Eq. 2

$$\sigma = \frac{it}{VA} \tag{2}$$

where σ is the chloride conductivity (mS/cm), i is the measured current (mA), V is the measured voltage (V), t is the specimen thickness (cm) and A is the cross-sectional area (cm^2).

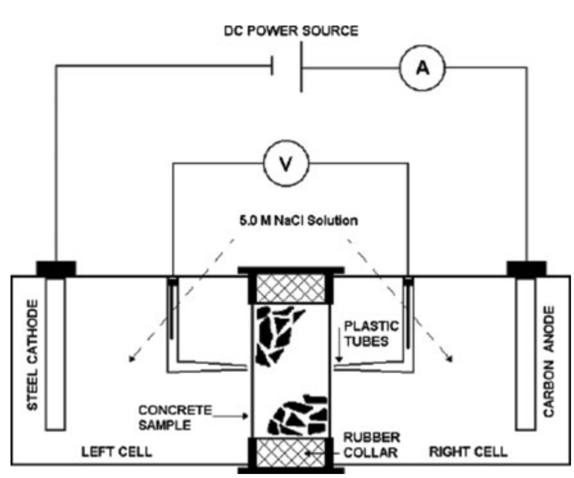

Fig. 2 Chloride conductivity test [2]

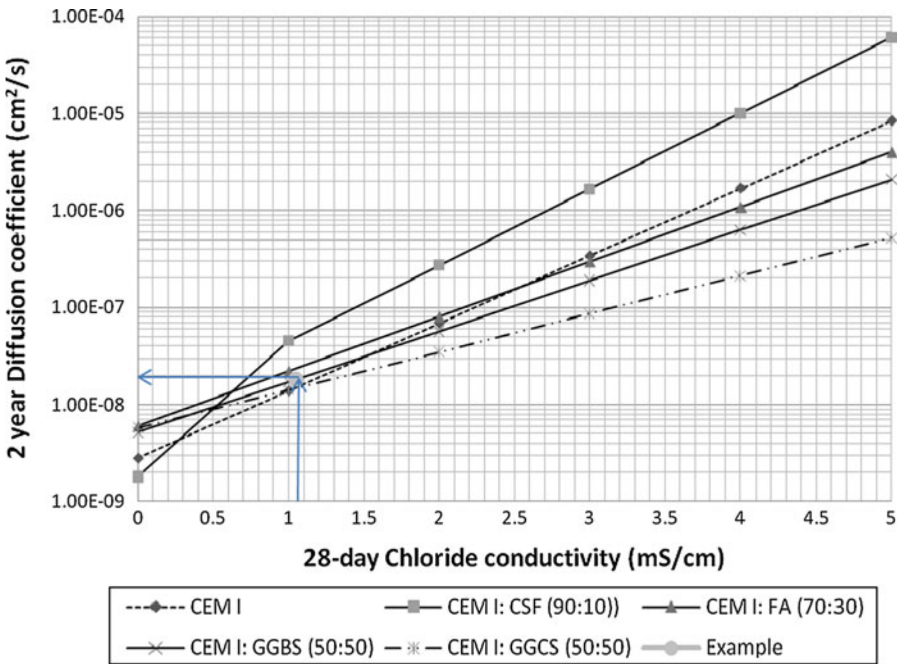

Fig. 3 Relationship between 2-year diffusion coefficient and 28-day chloride conductivity value [18]

The CCT has a number of advantages: first, the electric field application has minimal impact on the concrete due to the relatively short test time (s). Secondly, the test itself is the most rapid of all chloride tests developed to date; this includes the rapid migration test developed by Tang and Nilsson [35] and the bulk diffusion test [25]. Lastly, results from a round-robin test carried out in SA, to determine the precision of the CCT, indicated the CCT to be both repeatable and reproducible provided that the basic test parameters are met [33].

Mackechnie and Alexander [20] established a relationship between the 2 year diffusion coefficients($D_{2\ \mathrm{years}}$) and the 28-day result from the CCT. These were established primarily from laboratory-based experimental correlations between 28-day conductivity index values and chloride diffusion coefficients but modified by examining chloride ingress in marine structures of different ages in SA. Thus, there was a measure of actual in-service calibration in the model. In addition, the specimens covered a range of binder types (100% Portland cement and combinations of PC with supplementary cementitious materials that included fly ash and blast furnace slag); water/binder ratios and curing regimes.

From the correlations, Mackechnie and Alexander [20] set out a chart, shown in Fig. 3, in which 2 year diffusion coefficient($D_{2\ \mathrm{years}}$) is determined from the 28-day chloride conductivity value, and the SLP model (Eq. 1) is used to determine the service life of a structure.

From the graph, FA = Fly Ash; GGBS = Ground granulated blast furnace slag; GGCS = Ground granulated corex slag; CSF = Condensed silica fume.

From Fig. 3, it is shown that a chloride conductivity value of 1.05 mS/cm corresponds to a 2-year diffusion coefficient value of 1.83E−08 cm^2/s (57.8 mm^2/year).

In this study, Eq. 1 is applied in durability design to give material specifications in terms of a 2-year diffusion coefficient (D_i), which accounts for long-term effects such as chloride binding and continued cementing reactions. Figure 3 is then applied to give the 28-day chloride conductivity value that should be specified for design.

3 Deemed-to-satisfy approach

The deemed-to-satisfy approach is a set of "rules" for dimensioning, materials and product selection, and execution procedures [4]. The approach used in the South African context utilizes the SLP model for chlorides (Eq. 1) in selecting concrete cover to the

Table 2 Maximum chloride conductivity values (ms/cm) for different environmental classes and binder types: deemed to satisfy approach—monumental structures (cover = 50 mm) [4]

EN 206 Class[b]	70:30 CEM I: FA[a]	50:50 CEM I: GGBS[a]	50:50 CEM I: GGCS[a]	90: 10 CEM I: CSF[a]
XS1	2.50	2.80	3.50	0.80
XS2a	2.15	2.30	2.90	0.50
XS2b, XS3a	1.10	1.35	1.60	0.35
XS3b	0.90	1.05	1.30	0.25

[a] *FA* fly Ash; *GGBS* ground granulated blast furnace slag; *GGCS* ground granulated corex slag; *CSF* condensed silica fume

[b] Refer to Table 1

reinforcement by finding an optimum between quality and quantity of concrete covers. The output of the design is in terms of a limiting value of the chloride conductivity index for a given cover value. This allows the designer to select an optimal material composition that will resist the environmental actions threatening the structure, and hopefully prolong the period to corrosion initiation. Thus, durability design of RC structures using this deemed-to-satisfy approach offers the designer and material supplier flexibility in selecting an appropriate choice of materials and mix proportions of concrete that will satisfy the limiting DI value.

Table 2 gives a summary of limiting 28-day chloride conductivity values for various environmental classes and binder types. These values relate to concrete structures that have an expected service life of at least 100 years. The shaded value in Table 2 is used in a later example that compares the specifications of deemed-to-satisfy approach with those from a probabilistic approach.

4 Accounting for uncertainties in design

The input parameters of the SLP model (Eq. 1) exhibit variability arising from material variability, construction tolerances, experimental errors and modelling errors. The uncertainties are termed physical, statistical, and model uncertainties. The cover depth, x exhibits physical uncertainty due to variations in construction tolerances. Statistical uncertainties result from: measurement errors of, e.g., cover depth, number of samples tested, data handling and transcription errors depending on the quality assurance observed during the test. Model uncertainties are associated with the use of simplified mathematical models or relationships between the basic variables to represent the actual complex physical and chemical phenomena of chloride ingress and corrosion initiation [36]. This may be made due to lack of knowledge of the behaviour of materials in their service environment. In addition, diffusion models may limit the number of variables, leaving out a set of parameters which during model idealization are judged as secondary or of negligible importance to corrosion initiation. The aforementioned sources of uncertainty affect the predictive ability of the SLP model (Eq. 1). The uncertainties make it difficult if not impossible to say with any conviction that there is a uniquely defined value for the model output. Rather, there is a certain probability range of output values which the designer can apply to the problem at hand, based on practical or economic considerations. Thus, a probabilistic approach to durability design is both rational and offers promise of further refinements in design.

5 Probabilistic approach to durability design

In reality non-uniform deterioration of RC structures reflects the uncertainties discussed earlier. The variable nature of deterioration can be predicted by using probabilistic tools. The work presented here purposes to contribute to a better understanding of the application of probabilistic approaches to durability design.

Modelling of the service life of RC structures using probabilistic approach is made possible through the application of reliability theory as used in structural load design [23]. Reliability theory incorporates statistical databases and probability theory to give a relative measure of the confidence in the ability of the RC structure to perform its function in a satisfactory manner (ISO 13823, [13, 23]. Probabilistic methods can be either full-probabilistic or semi-probabilistic (partial factor method) and involve the use of reliability-based design and the limit state methodology. The limit-state method (LSM) for design of RC structures has been adopted by various standards and codes for design: first by ISO 2394 [14] and later by ISO 13823 [13] and fib Model Code (service life design) (fib Bulletin [9]. Although [14] includes durability in its principles, the LSM has not been developed for failures due to material deterioration to the extent that it has for failures due to gravity, wind, snow and earthquakes (ISO 13823 [13]).

The LSM incorporates the use of mathematical models which describe the deterioration mechanisms in concrete up till a specified limit state. The limit state is the border that separates desired states from the undesired or adverse states which a structure may come to exhibit during its lifetime (ISO 13823 [13]). Three types of limit-states are defined in ISO 13823 [13], namely the ultimate limit state (ULS), the serviceability limit state (SLS) and the ILS. ULS depicts the situation in which the safety of the RC structure is considered (for example the risk of collapse), while SLS defines the situation in which the functionality of the structure is considered (for example the limitation of crack widths). ILS precedes both the occurrence of ULS and SLS, and is used to describe the onset of quantifiable deterioration such as corrosion initiation. The decision on specifying the type of limit state is based on technical reasons, however, other aspects, such as economical, functional, and social reasons also need to be considered [22]. The study presented in this article focuses on the durability design of RC structures in saline environments and hence the appropriate limit-state is the ILS.

The mathematical model at the ILS is a function of a set of variables $f(x, C_s, D_i, t, m, C_{crit})$ that represent the loss of performance with time (ISO 13823 [13]). The variables include parameters defining material properties, geometric properties and environmental properties. These variables are stochastic in nature and require the use of a reliability-based design methodology to perform an analysis of the model.

To carry out a reliability analysis at the ILS, the variables are characterized further as either "action" effect, $S_{(t)}$ or as an initiation limit, $S_{\lim}$(ISO 13823 [13]). The basic requirement for the ILS defined at any time, t, during the design life of the RC structure/component can be generally characterized by an inequality that follows a limit state format given by Eq. 3 (ISO 13823 [13]).

$$S_{\lim} > S_{(t)} \tag{3}$$

For chloride induced corrosion, $S_{(t)}$ is the action effect represented by the SLP, (value of $C_{(x,t)}$ in Eq. 1). The other parameter in Eq. 3, $S_{\lim}$, is the resistance effect represented by C_{crit}, which is the critical (threshold) chloride concentration at the rebar level and which when exceeded results in corrosion initiation.

The relationship between $C_{(x,t)}$ and C_{crit} is modelled mathematically as a function of basic variables, $f(x, C_s, D_i, t, m, C_{crit})$, representing concrete dimensions, material properties and the environment of the RC structure as shown by Eqs. 4 and 5.

$$Z = C_{crit} - C_{(x,t)} = f(x, C_s, D_i, t, m, C_{crit}) \tag{4}$$

$$\Rightarrow C_{crit} - C_s\left[1 - \mathrm{erf}\left(\frac{x}{2\sqrt{(D_i t^{1-m})}}\right)\right] = 0 \tag{5}$$

The parameters in the limit state function (LSF) Eq. 5 are statistically quantified using data obtained from in situ and laboratory tests to give their respective distribution types, mean values and coefficients of variation (COV). An example of the statistical quantification of the surface chloride concentration parameter for structures constructed using Portland cement in very severe spray marine exposure (XS1) is given in Fig. 4. The figure shows two possible distributions that would be fitted to the surface chloride

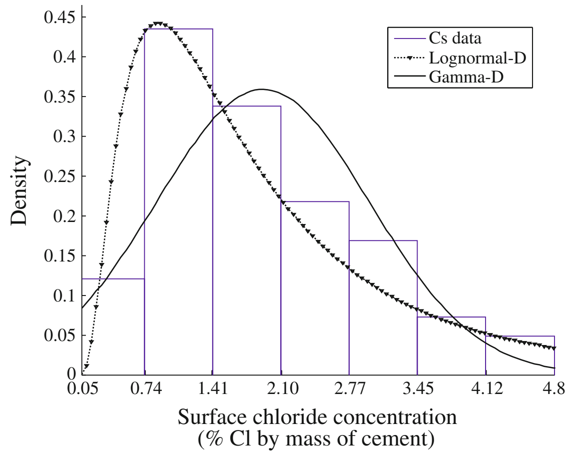

Fig. 4 Histogram and distribution fit of surface chloride concentration of OPC concrete in very severe spray marine environment [24]

concentration data which were obtained from chloride profiling of marine structures in SA.

The statistical information of the LSF parameters is then exploited to provide improved uncertainty estimates in the output, which is usually stated in terms of the probability that the condition represented by the LSF occurs, that is, the probability that $C_{(x,t)}$ is equal or greater than C_{crit}. The probability of this occurring during the design service life of the structure is termed as the probability of failure (P_{f}) and the condition is represented by Eq. 6 (ISO 13823 [13]):

$$P_{\mathrm{f}} = P\left(C_{(x,t)} \geq C_{\mathrm{crit}}\right) \tag{6}$$

The probability of failure (P_{f}) Eq. 6 can be cumbersome to use when its value becomes very small, and for this reason, the reliability index is usually preferred [26]. The reliability index, β is defined by the expression given by Eq. 7 [8]

$$P_{\mathrm{f}} = \varphi(-\beta) \tag{7}$$

where φ is the standard normal Gaussian cumulative distribution function.

The condition represented by Eq. 6 can be represented schematically through the use of probability density functions (PDFs) of $C_{(x,t)}$ and C_{crit} as shown in Fig. 5. The interaction between the two density functions represents the probability of failure (P_{f}).

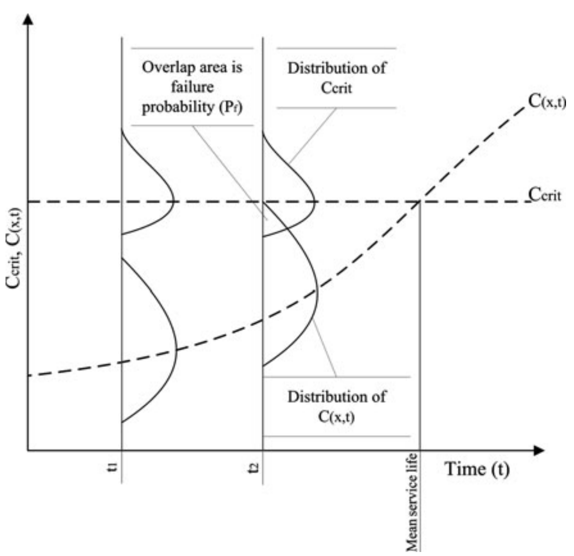

Fig. 5 Illustration of the decrease of "resistance" with time and increase of "load" with time (adapted from [30]

P_{f} is minimum at the beginning of the structure's service life but continuously increases with time due to deterioration of the structure. The dotted lines represent the mean values of $C_{(x,t)}$ and C_{crit} at any particular time.

The probability of failure (P_{f}) can be assessed using a number of well-established methods. The first-order reliability method (FORM) [11, 15, 27] and Monte Carlo simulation (MCS) are two such methods. The study presented here applied the MCS method in which a MATLAB® based subroutine was developed to aid in the analysis of Eq. 5.

The probability of failure obtained from the MCS, for a given choice of model parameters, should be verified for acceptance by comparing it to an acceptable (target) probability of failure, $P_{\mathrm{target,ILS}}$ such that (ISO 13823 [13]; [32]:

$$P_{\mathrm{f}} = P\left\{C_{(\mathrm{crit})} - C_{(x,t)} < 0\right\} < P_{\mathrm{target,ILS}} \tag{8}$$

$P_{\mathrm{target,ILS}}$, in Eq. 8, is given in durability standards and may be determined using two ways: (a) implicitly by conducting a cost-benefit analysis: this involves studying the variation of the initial cost, maintenance costs and the expected failure costs to arrive at the most economical target probability of failure for design $P_{\mathrm{target,ILS}}$ [26] or (b) explicitly by setting a value at the ILS that is comparable with the cost of repair estimated from actual case histories. The explicit values of the target levels for ILS design have been reported in [14], fib Model Code for service life design (fib Bulletin [9] and [7], and are summarized in Table 3.

If the condition expressed by Eq. 8 is satisfied, the designer can then proceed to give the design specifications that include concrete proportioning, bar arrangement, and placement conditions that are expected to meet requirements for the service life of the structure and members under the envisaged conditions of the deteriorating external forces.

Table 3 Indicative values of P_{target} and β_{target} for 100 years' service life design

Reference	$P_{\mathrm{target,ILS}}$	$\beta_{\mathrm{target,ILS}}$
[12]	0.05–0.20	0.8–1.6
[7]	0.067	1.5
[9]	0.04	1.8

5.1 Application of the full probabilistic approach

The probabilistic design method has been applied in several degradation models, such as in the Brite-EuRam project BE95-1347 [6], called the DuraCrete method (1996–1999) where the approach was subsequently applied in the design of high relevance structures such as the Western Scheldt Tunnel in 2000 [10]. Within the Duracrete model the Rapid Chloride Migration test was adopted as the standard laboratory test method to quantify the potential chloride transport properties of a concrete mix [6].

The study presented in this article is similar in principle to the DuraCrete approach but differs in two ways. First the study applies a prediction model applicable for South African environmental conditions to specify material parameters that will ensure that the RC structure remains durable within its service life. Secondly, the model used applies the chloride conductivity (DI) test to determine values for the material resistance parameter. The CCT used here was developed in SA and has been widely adopted by the local construction industry.

5.2 Drawbacks of the full probabilistic approach

Notwithstanding the usefulness of the full probabilistic approach, the method has inherent problems. The first problem is the interpretation of the acceptable (target) probability of failure, P_{target}. P_{target} can assume different values depending on various situations. Table 3 gives the indicative intervals of P_{target} and β_{target} values as reported in literature. The reason for the variability in P_{target} arises from the fact that different researchers have different interpretations of the ILS. There is no clarification in literature, on what the practical meaning is of a certain probability of corrosion initiation in terms of either the amount of depassivated steel, damaged concrete surface area or maintenance costs. Consequently, the accepted probability for corrosion initiation ($P_{\text{target,ILS}}$) varies between different design standards depending on the interpretation of the 'probability of corrosion initiation'.

The second drawback of the probabilistic approach arises from the fact that the method relies on characterizing each parameter in terms of mean, standard deviation and statistical distribution. This is a problem due to the fact that the amount of data available for most of the parameters in the model tends to be limited.

Thirdly, the method relies heavily on numerical methods for analysis. It requires the use of statistical software packages that incorporate numerical methods such as MCS.

6 Durability design example

6.1 Concrete pier

This section is concerned with exemplifying the full-probabilistic approach and comparing the results with those given by the deemed-to-satisfy approach. The section aims at creating an understanding of the different solutions the designer has to performance-based durability design.

The probabilistic approach is exemplified for a concrete pier to be cast in situ in an extreme splash and tidal marine environment (XS3b) using a blend of Portland cement and ground granulated blast furnace slag—OPC: GGBS (50:50). GGBS has been selected as it has been found to be beneficial in the marine environment in that it is able to bind chlorides with time and resist the ingress of chlorides [2, 5].

The designers' task is to specify the required material resistance against chloride penetration in terms of a chloride conductivity value and/or corresponding diffusion coefficient given the service life design model input parameters in Table 4. The statistical values for the model parameters data, relating to OPC: GGBS in XS3b environmental conditions are obtained from a statistical quantification of data collected from both laboratory work and existing structures for the respective parameter. Local data on the critical chloride concentration parameter are not yet available, hence the statistical data for this parameter were obtained from Duracrete [6]. It should also be noted that the reduction factor 'm' for diffusion coefficient is assumed in this case to be deterministic due to insufficient data for use in its statistical quantification.

6.2 Analysis

For probabilistic analysis the SLP model (Eq. 1) is represented as a LSF (Eq. 8) and a solution of the LSF is obtained using MCS techniques. The design task is to calculate a set of values for material resistance against chloride penetration (D_i or chloride conductivity values) corresponding to the probability of the event 'corrosion initiation' using the set of parameters in Table 4.

The material resistance value selected for design specification will be that corresponding to a probability of failure of 6.7% for the 100 year service life of the structure—which is the target probability of failure, $P_{\text{target,ILS}}$, specified by [7]. This value corresponds to a target reliability index ($\beta_{\text{target,ILS}}$) of 1.5 (Table 3).

In addition, the D_i (leading to the chloride conductivity) value specified for design will ensure that the structure meets its performance requirement (100 years' service life) for the design cover depth of 50 ± 10 mm. If during the analysis the choice of the geometry parameter (cover depth) results in a chloride conductivity value that cannot be economically attained for the specified performance, then a tradeoff is made by adjusting either parameter appropriately until the condition specified by Eq. 8 is satisfied.

6.3 Results

The design verification for Eq. 8 was carried out using MCS. A MATLAB® based subroutine programme developed for this study was applied to carry out MCS on the respective stochastic quantities of the parameters in Table 4.

The simulated values were then substituted into Eq. 9 and the diffusion coefficient calculated.

$$D_i = \left[\frac{2}{x}\text{erf}^{-1}\left(1 - \frac{C_{\text{crit}}}{C_{\text{s}}}\right)\right]^{-2} \cdot \frac{1}{t^{(1-m)}} \cdot \theta_{D_i} \tag{9}$$

Parameters in Eq. 9 have been previously defined in Eq. 1. The model uncertainty parameter $\theta_{\text{D}_\text{i}}$ is included in Eq. 9 to account for error in the diffusion coefficient value. Due to lack of a suitable value for this parameter, this study assumes the statistical values for $\theta_{\text{D}_\text{i}}$ as: Normal distribution with a mean value of 1.0 and a standard deviation of 0.01.

Figure 6 shows the cumulative density function (CDF) obtained from the analysis.

For a failure probability of 0.067 (line 1–1), a diffusion coefficient value of 52 mm^2/year was obtained. The deemed-to-satisfy approach specifies a chloride conductivity value of 1.05 (Table 2) which corresponds to a diffusion coefficient value of 1.83E−08 cm^2/s (57.8 mm^2/year) and a probability of failure of 0.1 (line 2–2).

At a probability of failure of 0.5 the corresponding diffusion coefficient is 115 mm^2/year (3.65×10^{-12} m^2/s). The 50% probability of failure (line 3–3) corresponds to the value attained using deterministic values for each of the parameters in Eq 9.

Table 5 gives a summary of the diffusion coefficient values together with their corresponding chloride conductivity values at the 6.7 and 50% failure probabilities. Also included in Table 5 are 2-year diffusion coefficient results for an assumed service life of 50 years.

For the 100 year service life, the diffusion coefficient value of 52 mm^2/year (1.65×10^{-12} m^2/s) corresponds to a chloride conductivity value of 1 mS/cm (the chloride conductivity value is determined from a correlation chart of 2 year diffusion coefficients and 28-day chloride conductivity values that was established by Mackechnie [17]. This means that the chloride conductivity value measured on 28-day old cores taken from the concrete pier should not be greater than 1.0 mS/cm. It can be observed that the design chloride conductivity value obtained from the probabilistic approach is lower than that specified by

Table 4 Input parameters for reliability analysis of a RC pier

Reference	Parameter	Units	Mean	COV	Shape parameter (α)	Scale parameter (β)	Distribution
Ronné [29]	Concrete cover	mm	50	0.2			Truncated normal
Duracrete [6]	Critical chloride content	%	0.48	0.31			Truncated normal
Muigai [24]	Surface chloride concentration	%	4.13	0.21	22.68	0.18	Gamma
Mackechnie [17]	Reduction factor	–	0.68	–			Deterministic
	Design service life	years	100	–			Deterministic

COV coefficient of variation

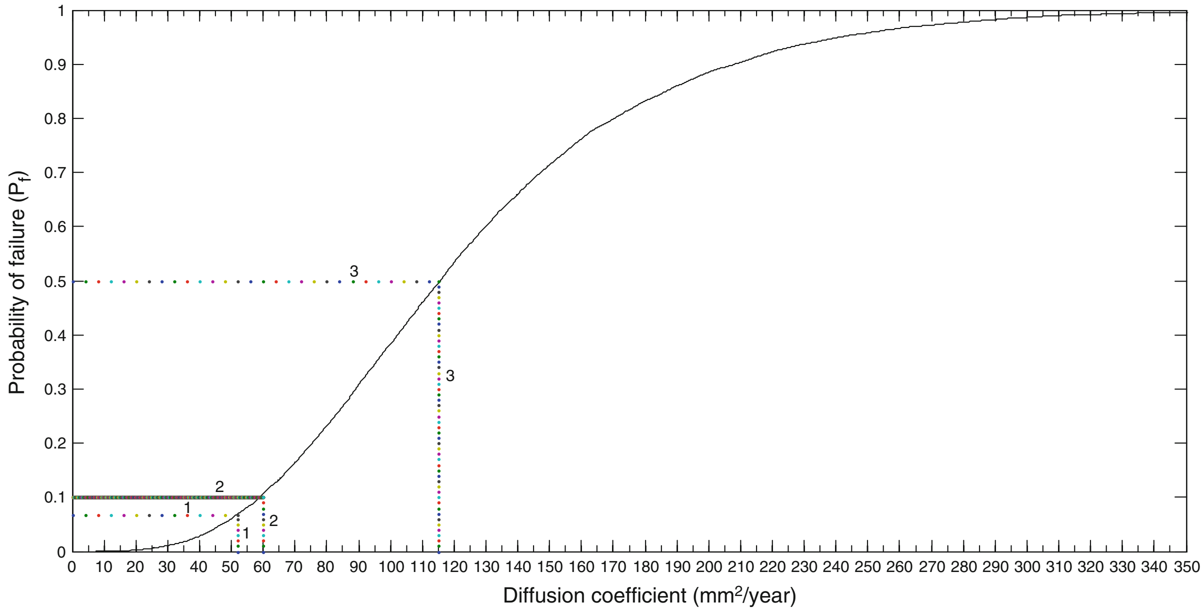

Fig. 6 CDF of probability of failure for diffusion coefficient values

Table 5 Results of the reliability analysis

Label from Fig. 6	Probability of corrosion initiation (P_f)	Design values ($T = 100$ years)		Design values ($T = 50$ years)
		Diffusion coefficient (2-year) (mm^2/year)	Chloride conductivity (28-days) (mS/cm)	Diffusion coefficient (2-year) (mm^2/year)
1–1	0.067	52.0	1.00	63.5
2–2	0.1	57.8	1.05	73.5
3–3	0.5	115.0	1.64	144.2

the deemed-to-satisfy approach of 1.05 mS/cm for the same parameter values (Fig. 3).

In conclusion, for the RC pier to have a 6.7% probability of corrosion initiation during its 100 year life time, the designer should specify a chloride conductivity value of not greater than 1.0 mS/cmand cover depth of 50 ± 10 mm.

The quality of concrete specified for the 50 year service life is less compared to that of the 100 year life-time.

7 Sensitivity analysis of the variables

The results shown in Fig. 6 only relate to the example given in Table 4. A sensitivity analysis is required to show the change in probability of failure due to variation of each of the parameters given in Table 4. A sensitivity analysis was carried out by varying the parameter of interest over the valid range (i.e. $\approx \bar{x} \pm 3s$), and maintaining the other variables at their respective base case settings. Table 6 gives a summary of the information required for the sensitivity analysis.

The diffusion coefficients are presented in units of mm^2/year (1 mm^2/year = 3.171×10^{-14} m^2/s). These units were chosen to simplify the presentation of the results, and can be directly used in the service life calculations because the unit measure for concrete cover is mm and the unit for service life is years.

As an example, the sensitivity of probability of failure to changes in cover depth is shown in Fig. 7. From the Fig. 7 it can be shown that increasing the cover depth leads to a decrease in probability of failure.

Table 6 Range of parameters for sensitivity analysis

Parameter	Base case (mean value)	Range of values
x (mm)	50.0	0 → 100
D_i (mm^2/year)	52	0 → 110.2
C_{cr} (% mass of cement)	0.48	0.03 → 0.93
C_s (% mass of cement)	4.13	1.53 → 6.73
t (years)	100	–
m	0.68	–

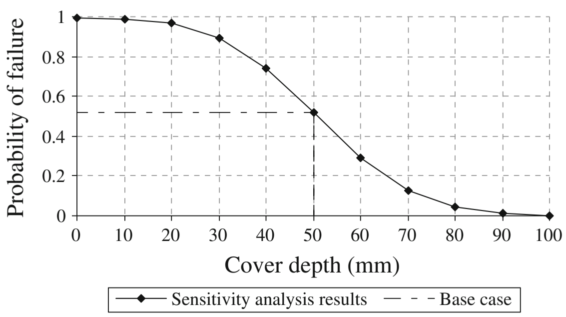

Fig. 7 Sensitivity of cover depth to probability of corrosion initiation

A summary of the results of the sensitivity of the estimated service life (100 years) to changes in the parameters represented in Table 6 are given in Table 7.

The sensitivity study results shown in Table 7 show the most influential parameter to failure probability, to be concrete cover, followed by the diffusion coefficient, critical chloride content and lastly surface chloride concentration.

8 Further studies to improve the probabilistic model

The design example given in this paper for chloride-induced corrosion using chloride conductivity value, as the design DI value, is based on established relationships between 28-day chloride conductivity value and 2 year diffusion coefficients'. This relationship is based on correlation studies that applied diffusion data collected from concrete structures in the Western Cape of SA and hence the relationship does not take into account the range of marine environmental conditions in SA. Chloride ingress is dependent on environmental conditions such as sea water and air temperature and relative humidity. These

Table 7 Sensitivity of parameters to P_f

Number	Parameter	Sensitivity (%)	
1	x (mm)	0.47–77.05	↓Decreasing sensitivity
2	D_i (mm^2/year)	3.85–21.47	
3	C_{cr} (% mass of cement)	6.87–15.47	
4	C_s (% mass of cement)	4.3–13.15	

conditions vary widely along the South African coastline, for example sea temperatures along the Atlantic side of the Cape Peninsula are on average 5°C lower than the adjacent False Bay while sea temperatures at East London (bordering the Indian ocean) are on average several degrees warmer than those in False Bay [17].

Further work is necessary to test 28-day chloride conductivity index values against chloride ingress in different marine environments in SA. A study is currently in place at the University of Cape Town, to investigate the ingress of chlorides into various different types of concrete based on site exposure in the Cape Town and Durban harbours. In this study, measurements taken on site-exposed samples are correlated to laboratory-based measurements of chloride conductivity index values and diffusion coefficients. These correlations are expected to refine the existing service life models and make it applicable to various regions in SA.

Only when the refined model is in place can the actual verification of model predictions using the probabilistic approach proceed.

9 Conclusions

The probabilistic service life design methodology presented in this study follows the same principles of reliability and performance given in structural design standards such as [14]. The methodology takes into consideration the scatter of the influencing parameters of durability and represents these parameters in the form of a limit-state function (LSF).

Based on this article the following main conclusions can be drawn:

(i) This work has demonstrated the importance of accounting for the high variability that exists in basic variables for durability design. Uncertainties in concrete material properties, structural geometry and micro-environmental load were successfully accounted for and represented using stochastic quantities (mean, coefficient-of-variation and probability density function).

(ii) The application of a probabilistic model to design for durability not only involves selecting appropriately the materials and the mix proportion of concrete and deciding structural details of the structure but also ensuring the reliability of performance of the designed structure throughout the service life.

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