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Department of Civil Engineering

A Practical Carbonation Model for Service Life Design of Reinforced Concrete Structures

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Declaration

This dissertation is being submitted in partial fulfillment for the Degree of Master of Science in Civil Engineering at the University of Cape Town. It has not been submitted before for any degree or examination in any other university. I know the meaning of plagiarism and declare that all the work in this document, save for that which is properly acknowledged, is my own.

Signature: Signed by candidate

Date: 07th September 2020

Dedication

Dedicated to my family for the love and support.

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Abstract

The increase in atmospheric carbon dioxide concentration due to global warming has a direct impact on the amount of carbonating concrete structures. For the past years, numerous studies have been done in South Africa on the subject and models developed to predict carbonation in concrete structures. Despite the large amount of resources and research effort put into developing these models, the translation from theory to practice represents a great challenge for design engineers in the field of durability design. This study presents a design tool based on existing models for use in practical applications. The proposed design tool assists in computing the service life of carbonating concrete structures and provides reliability values associated with the service life. It accounts for different binder compositions and binder types, as well as different locations and environmental land uses in South Africa. The validation of the design tool was done by comparing the service life prediction results to existing models, which generally showed good agreement. The developed design tool can be applied for predicting the long-term performance of new RC structures as well as improving the basis for quality assessment of existing, newly built RC structures. For the design of new structures, the designer is required to make certain assumptions concerning the information to be used for the simulation. These include values for the binder type, binder content, OPI, cover depth, land use and exposure parameters. For the quality control of new structures, the way in which the model parameters are obtained differs from that of new structures. As the structure already exists, both the concrete quality, cover depth and environmental loading can be measured directly on the structure with appropriate testing procedures. The outcome of applying the design tool for the analysis of concrete produced for the Gauteng Freeway Improvement project (GFIP) is also presented, with a case study of precast and in-situ structures chosen for the analysis.

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Abbreviations and symbols

CCI - Chloride Conductivity Index

CH - Calcium hydroxide (Portlandite)

CO_2 - Carbon dioxide

CSF - Condensed silica fume

DI - Durability Index

FA - Fly ash

GFIP - Gauteng Freeway Improvement project

GGBS - Ground granulated blast furnace slag

ILS - Initiation limit state

LSF - Limit-state function

MCS - Monte Carlo simulation

OPI - Oxygen Permeability Index

PC - Portland cement

P_f - Probability of failure

RC - Reinforced concrete

RH - Relative humidity

SANRAL - South African National Road Agency Limited

SCM - Supplementary cementitious material

SL - Service life

w/b – Water/binder ratio

WSI - Water Sorptivity Index

CHAPTER 1

1. Introduction

1.1 Background of study

The increase in atmospheric carbon dioxide (CO_2) concentration due to global warming has a direct impact on the degree of carbonation of reinforced concrete (RC) structures. The carbonation phenomenon results from the interaction between cement hydration products in concrete and the atmospheric CO_2. This leaves the steel reinforcement vulnerable to corrosion (depassivation) (Dyer, 2014; Bohni, 2005; Bertolini, et al., 2013; Li, 2016). After depassivation, corrosion may occur and propagate depending on the availability of corrosion agents (moisture and oxygen). Corrosion of steel reinforcement in RC structures has been reported to be the major cause of premature deterioration around the world (Venkat, et al., 2014).

Premature deterioration of RC structures is becoming a major issue all over the world, with governments spending large amounts of money on repairs. According to the Corrosion Institute of Southern Africa (CISA, 2004), reinforcement corrosion costs are estimated at 5.2% of GDP in South Africa, 1.02% in Japan (Shibata, 2002) and 3.1 to 4.2% in the USA (Volcan , 2014; NACE, 2002). Deterioration of RC structures due to corrosion of steel reinforcement is marked by two stages; the initiation stage and propagation stages as shown in Figure 1.1. The initiation period corresponds to the ingress of CO_2 up to the level of the reinforcement. This results in the depassivation of the concrete layer around the reinforcement. The service life of a structure can be defined with respect to the relevant limit state which for this study is the initiation limit state (ILS) marked by corrosion initiation and relates to the time during which the structure is able to meet its specified durability requirements with an acceptable level of safety (Richardson, 2002). (fib, 2006; EN 1991-1, 2002) provides a target level of safety value (reliability value) for ILS equals 1.5 for structures with 50-years reference period.

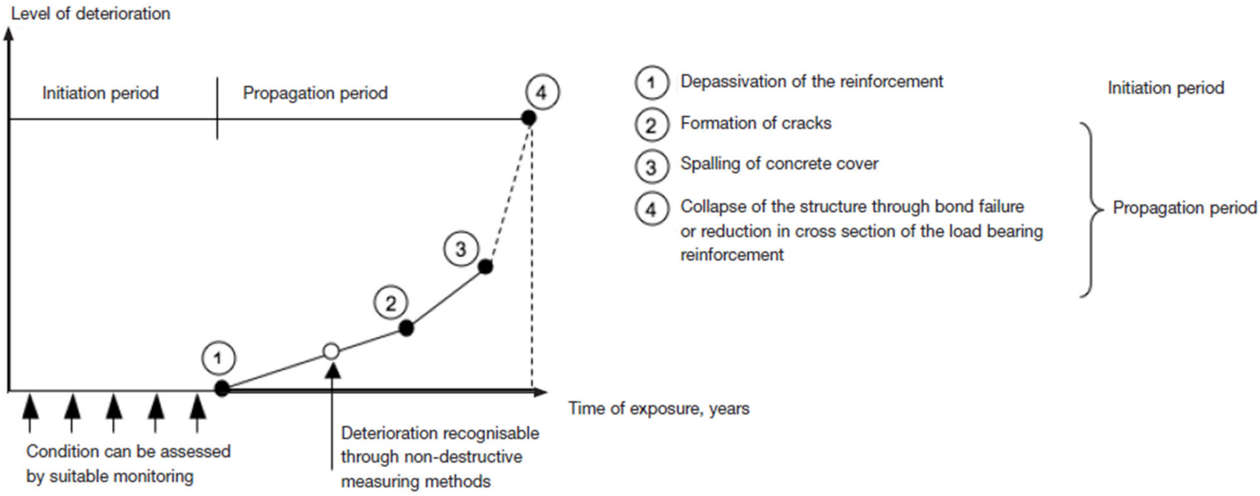

Figure 1. 1 : Deterioration process of reinforcement corrosion: two-phase model for service life (fib, 2006).

The durability of a RC structure is associated with the resistance against deterioration of concrete over the intended service life. This deterioration largely depends on the interaction between the concrete and its environment, with the concrete influencing the resistance to the ingress of aggressive agents (e.g. CO_2) and the environment influencing the degree of aggressiveness (Ballim, et al., 2009). Durability design of carbonating structures is therefore involved with specifying concrete with appropriate resistance to the progression of the carbonation front up to the level of the steel reinforcement during its service life.

Predicting the service life of a RC structure is important in today's construction industry, as increasingly owners are demanding that the designers and contractors provide assurance of a durable structure over the predefined service life. The deterioration of RC structures may be expected and even accounted for using reliable deterioration models (service life models), or it may be unexpected due to inadequate understanding of deterioration mechanisms or changes in external environments or the use of the structure (Alexander, et al., 2017). Deterioration models are generally used during the design and construction phases of new structures, in order to develop performance specifications; by combining both cover quality and cover thickness.

The international concrete industry currently largely relies on prescriptive specifications for durability design (Alexander, et al., 2017). Progress in both research and technology has led to innovative and economic concrete being used nowadays, making prescriptive approaches to durability design no longer efficient in certain instances. The prescriptive approach fails to provide a means to verify or control the presumed concrete durability (Mehta, 2006). This has led to a shift to performance specifications to address durability requirements (Alexander, et al., 2017).

As a consequence of the inefficiency of the prescriptive approach, the durability index (DI) approach was developed in South Africa and subsequently adopted in the local industry. The DI approach makes use of prediction models to estimate the service life of RC structures. DI's characterize the microstructure of the concrete cover and have been shown to be sensitive to material parameters – e.g., binder type, processing influences, type and degree of curing and environmental influences, e.g. temperature and relative humidity (Stanish, et al., 2007).

The DIs are derived from test methods which provide a measure of the penetrability of the cover concrete. The tests yield transport-related parameters and include the Oxygen Permeability Index (OPI) (the negative log of the coefficient of permeability k). The OPI test has shown a strong correlation with carbonation depth and has further been developed for performance-based design and specification. Using suitable service life models and depending on the exposure conditions and given service life, limiting values for OPI and cover depths for use as performance specifications have been developed in South Africa (Alexander, et al., 2006; Alexander & Mackechnie, 2001).

1.2 Problem statement

Carbonation cannot be ignored with regards to durability and service life of RC structures. A correct understanding and quantification of carbonation is essential for planning the maintenance and repair of new and existing structures. Over past years, numerous studies have been done by researchers in South Africa on concrete carbonation, and models developed to predict carbonation in RC structures. These include amongst others attempts by (Ballim, 1994; Mackechnie, 1996; Ballim & Lampacher, 1996; Lampacher, 2000; Mackechnie & Alexander, 2002; Yam, 2004; Alexander, et al., 2007; Salvoldi, 2010; Yunusa, 2014). Despite the resources and research efforts put into developing these carbonation models, the translation of this existing knowledge into good and appropriate engineering practice represents a great challenge for engineers and professionals as the models are generally too detailed and require input of data that is often not available to the engineer. There is therefore a need to develop design tools based on the existing carbonation models for use in practical applications. This will involve refining the model parameters that cannot easily be tested or specified to parameters that can easily be specified or tested.

The introduction of probabilistic-based service life assessment of RC structures has shown to be valuable and has been widely adopted in design codes replacing deterministic approaches

(McGee, 1999; fib, 2006; DuraCrete, 2000; DuraCrete, 1998). South African carbonation models are deterministic in nature with each model parameter characterized by a mean value. However, because of the random nature of the physical and the chemical properties of the concrete cover, as well as the uncertainties in the geometry of the structure and the environmental conditions, it is necessary to resort to a probabilistic approach in which the parameters are represented by a statistical distribution, a mean value and a standard deviation. The outcome of such a probabilistic approach is the probability that the limit state function, which corresponds to the margin between the desired and the undesired state, is exceeded within the design service life.

From the above discussions, it is evident that comprehensive and fundamental research is needed, aiming at developing an appropriate engineering design tool for the service life design of new and existing RC structures; one that translates the existing knowledge into practical application. Also, there is a need to resort to a probabilistic approach in which the parameters are represented by a statistical distribution, a mean value and a standard deviation. It is hoped that the resulting design tool may contribute towards enhancing the use of the carbonation model for durability design of RC structures in South Africa.

1.3 Research aim and objectives

The aim of this research is to propose a computer-based model for service life design of RC structures subject to carbonation-induced reinforcement corrosion, which can be used by engineers in practice. This model must be physically and chemically correct, simple and user-friendly, and include some probabilistic considerations in estimating the service life of RC structures. The proposed probabilistic model is based on an existing empirical model, with some alterations made in order to make it more user-friendly and functional. In order to achieve this, the following secondary objectives needed to be achieved.

i. Select an appropriate model relevant to the South African Durability Index approach and determine the relevant limit state function parameters for the model.
ii. Defining statistical information (distribution function, mean and standard deviation values) for the limit-state function parameters.
iii. Modelling carbonation depth in terms of probability of exceeding the limit state function, then expressing it in terms of reliability parameters.

1.4 Scope and limitations of study

The focus of this study was to develop a computer-based model for carbonation prediction. A carbonation model relevant to the South African durability index approach was used as base model for the research.

The statistical quantification of the model parameters was based on information obtained from literature. Two categories of uncertainty exist when using empirical models for service life prediction; aleatory (random) uncertainties and epistemic (subjective) uncertainties (Chio et al., 2006). The former is irreducible as it is inherent to the model while the latter is reducible and stems from measurements and lack of knowledge on the data. To develop this model, only uncertainty that stems from measurements and lack of knowledge of the data was considered. This involved the variability in the concrete cover and permeability parameters.

The failure probability is computed using the Monte Carlo simulation method. For different random values, the limit state function is verified. A simple count of the number of failure results helps to estimate the probability of failure. Increasing accuracy with this approach is obtained as the number of simulations turn to infinity. The Monte Carlo method has as a principal advantage the fact that it is easy to implement. However, the main inconvenience with this method is that better accuracy is obtained at higher simulations values, which is usually associated with high computational effort.

Though several carbonation models exist worldwide, this research is restricted to the model relevant to the South African Durability Index approach. Hence, the parameters considered are limited to the level of details of the model used. However, the proposed framework could be extended to any other model.

Dealing with the Monte Carlo method in evaluating failure probability often requires a very high number of simulations. Unfortunately, the computational power was limited, such that the number of simulations was limited to 10,000 simulations.

1.5 Outline

This thesis comprises of six chapters. The content of each chapter are as follows:

Chapter one:

This chapter presents an introduction to the study, covering the theoretical background of the research. The problem statement and justification of research is presented as the need to develop a design tool based on existing carbonation models for use in practical applications. The research objectives, scope and limitations are also presented in this section.

Chapter two:

This chapter presents a detailed literature review of the research topic and is further split into the following sub-sections; The South African durability index approach with the OPI test method, the Gauteng Freeway Improvement Project (GFIP) initiated by the South African National Road Agency Limited (SANRAL), common binder types used in South Africa, carbonation phenomena and its associated mechanism, factors influencing carbonation-induced corrosion, carbonation prediction models relevant to the South African durability index approach, probabilistic approach to durability design, methods of computing failure probability.

Chapter three:

This chapter presents the proposed probabilistic model. This consist of phase one, which describes the parameters relevant for carbonation and the relevant statistical information and phase two describes the modelling of service life and the probability of exceeding the limit state in an MS Excel spreadsheet.

Chapter four:

Chapter four presents a brief explanation of the developed software program, with a detailed user guide for the developed Excel program presented in Appendix 1. This chapter also presents the results of the comparison of the developed model with the existing OPI - carbonation model as well as a sensitivity analysis of the input parameters to check the validity and sensitivity of the developed model.

Chapter five:

Chapter five presents the application of the probabilistic model for the durability investigation. A case study of the Gauteng Freeway Improvement Project (GFIP) is presented.

Chapter six:

This chapter presents the conclusions drawn from the research work, literature and the discussion presented in Chapter 4 and chapter 5. Also included in this Chapter are recommendations for future work in this area.

Also included in this dissertation is a detailed user guide for the developed Excel program. This is presented within the Appendices.

CHAPTER 2

2. Literature Review

2.1 Introduction

The review of literature in this chapter provides a detailed background of the research topic and is divided into nine sections. Section 2.1 general introduction, Section 2.2 explains the South African Durability Index approach with the OPI test method. Section 2.3 gives an overview of the common binder types used in South Africa. Section 2.4 describes the carbonation phenomena and its associated mechanism. Section 2.5 explains the factors influencing carbonation-induced corrosion. This is followed by Section 2.6 which presents a review of carbonation prediction models relevant to the South African durability index approach, while Section 2.7 presents a brief review on the probabilistic approach to durability design. Section 2.8 describes the methods of computing failure probability. Section 2.9 provides a summary of the literature review.

2.2 South African DI approach and OPI method

Durability design of reinforced concrete (RC) structures in adverse environments is mostly concerned with ensuring the presence of a quality concrete cover that can resist the penetration of aggressive agents during the intended service life (Richardson, 2002). In South Africa, the approach adopted to address durability concerns is the Durability Index (DI) approach. The approach makes use of DIs as engineering measures of the potential resistance of the concrete cover to the transport of fluids and ions through concrete (Alexander, et al., 1999). The DI approach is derived from three test methods: the Chloride Conductivity Index test (CCI), where chloride ion resistance is measured; the Oxygen Permeability Index (OPI) test to establish carbonation resistance; and a Water Sorptivity Index (WSI) test to examine concrete for surface water absorption (Muigai, et al., 2012). The DI test values serve as a valuable input to service life prediction models. The DI test relevant to carbonation is the OPI test method.

2.2.1 The OPI test method

The OPI test method developed by Ballim (1994) is one of the DI test methods. The OPI test measures the overall micro- and macrostructure of the cover layer of concrete in terms of quantifiable engineering parameters (durability indices). It consists of measuring the pressure decay of oxygen passed through a concrete core sample, 30 mm thick and 68 to 70 mm in diameter. The concrete core is placed in a falling head permeameter (Figure 2.1). The OPI is defined as the negative log of the coefficient of permeability. Common OPI values for concrete generally ranges from 8,5 to 10,5 with values measured on a logarithmic scale. High OPI values indicates a concrete of potentially higher quality (lower permeability).

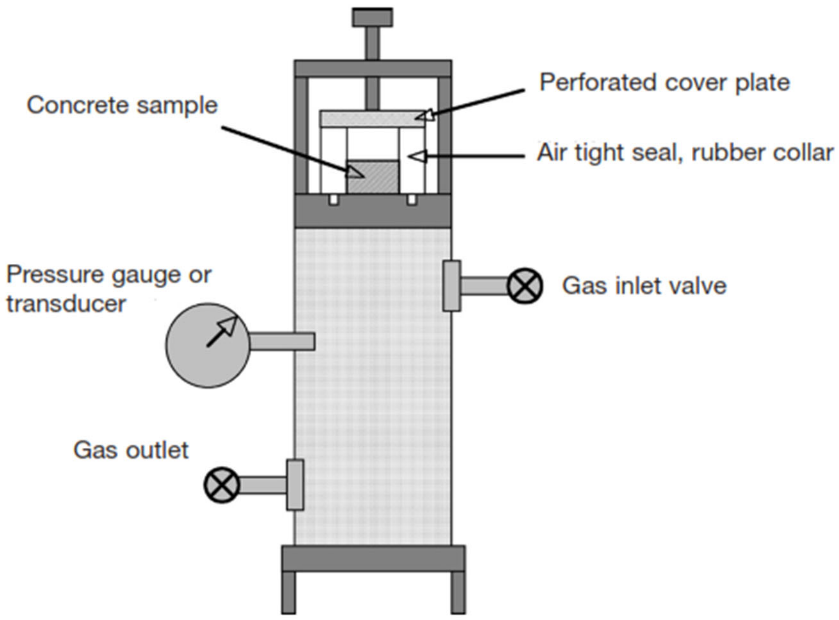

Figure 2. 1 : Schematic of an Oxygen Permeameter (Ballim, et al., 2009).

The OPI test is used with both in-situ and laboratory made concrete to measure permeability. The test is particularly sensitive to micro-voids and cracks as these acts as short circuits for the permeating gas. It can also be used to assess the degree of interconnectedness of the pore structure (Ballim, et al., 2009).

Correlations between OPI values and carbonation depth, after natural exposure conditions, have been found to be good as can be seen in Figure 2.2. The correlations are based on empirical relationships, which has served as the basis in developing a service life prediction model (Mackechnie & Alexander, 2002). The OPI test values serve as input values for durability predictions. This has been implemented by the South African Road Agency Limited (SANRAL) in a large-scale project, the Gauteng Freeway Improvement Project (GFIP) (SANRAL, 2010).

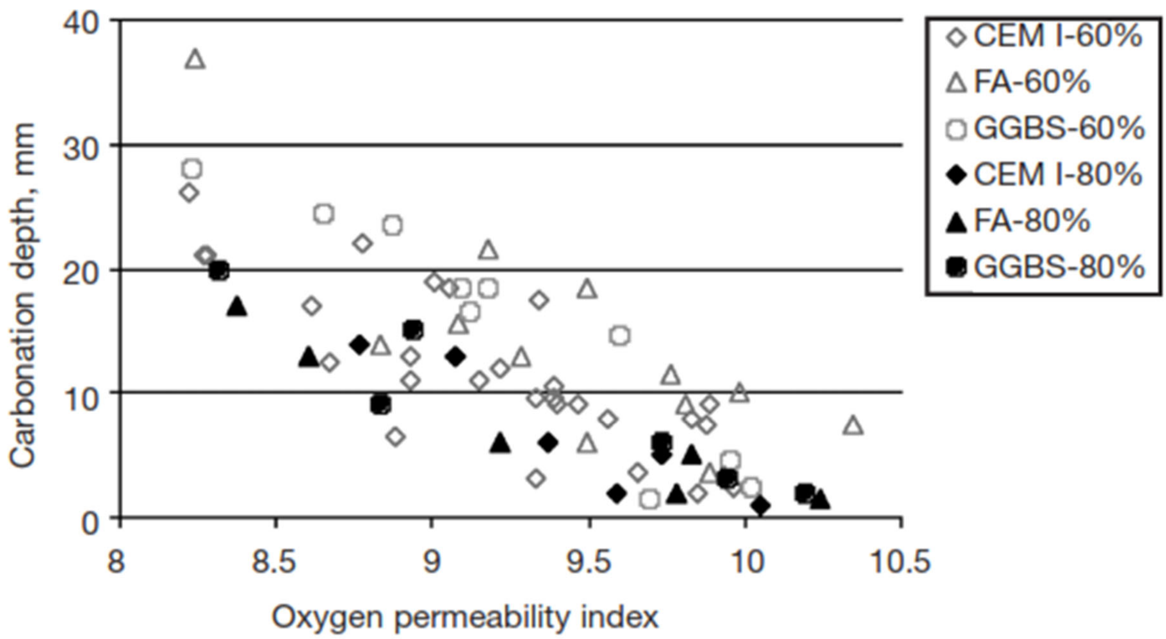

Figure 2. 2 : Carbonation depth in various concrete versus oxygen permeability index (measured at 28 days) for four years exposure at an average relative humidity of 60% or 80% (Mackechnie & Alexander, 2002).

2.3 Common binder types used in South Africa

Binders in concrete generally refer to the cementitious materials which serve as a binding material, glue or adhesive in hardened concrete and mortar. They consist, with very few exceptions, of Portland cement or a blend of Portland cement and cement extenders (Grieve, 2009). In South Africa, cementitious materials are available as:

- Common cement which includes Portland cement (PC) on its own, and factory-made blends as PC with cement extenders.
- Masonry cement which is formulated for use in mortar and plaster
- Cement extenders sold separately. These include:
  - Ground granulated blast furnace slag (GGBS)
  - Fly ash (FA)
  - Condensed silica fume (CSF)

2.3.1 Portland cement and supplementary cementitious materials: composition types and properties

2.3.1.1 *Portland cement (common cements)*

The South African standard SANS 50197-1 distinguishes five types of Portland cements. These include; Portland cement (CEM I), Portland-composite cement (CEM II), blastfurnace cement (CEM III), pozzolanic cement (CEM IV) and composite cement (CEM V). Portland cements are made from oxides of lime (CaO), silica (SiO_2), alumina (Al_2O_3) and iron (Fe_2O_3). For South African cement clinker, the composition of the oxides is shown in Table 2.1 (Grieve, 2009; Alexander & Beushausen, 2010). Four main compounds are present in cement clinker: tricalcium silicate, dicalcium silicate, tricalcium aluminate and tetracalcium aluminoferrite. Gypsum is also added to cement clinker in order to retard the rate of hydration. The typical compound composition of South Africa CEM I cement is shown in Table 2.2 (Grieve, 2009).

Table 2. 1: Composition of Portland cement clinker (Grieve, 2009).

Oxide	**% by mass in cement**
CaO	63 – 69
SiO_2	19 – 24
Al_2O_3	4 – 7
Fe_2O_3	1 – 6
MgO	0.5 – 3.6
Na_2O + 0.658 K_2O	0.2 – 0.8

Table 2. 2: Compound composition of South African CEM I cements (Grieve, 2009).

Compound	**Formula**	**Abbreviation**	**% by mass in cement**
Tricalcium silicate	$3CaO.SiO_2$	C3S	60 - 70
Dicalcium silicate	$2CaO.SiO_2$	C2S	8 - 30
Tricalcium aluminate	$3CaO.Al_2O_3$	C3A	5 - 12
Tetracalcium aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	C4AF	8 - 16
Magnesia	MgO	M	1.9 – 3.2
Gypsum	Raw material	-	4.4 – 6.7
Free lime	CaO	-	0.2 – 2.5

2.3.1.2 *Supplementary cementitious materials (cement extenders)*

Supplementary cementitious materials (cement extenders) are usually industrial waste products that have cementing properties when used together with Portland cement (C&CI, 2009). These materials are either factory-blended with Portland cement or introduced into concrete in a concrete mixer. Three main types of cement extender are used in South Africa. They are discussed in the following sections.

i. *Ground granulated blastfurnace slag (GGBS)*

GGBS is a by-product of the manufacture of iron. Blastfurnace slag which consists of silica and alumina together with lime or dolomitic lime is cooled rapidly to form a glassy reactive substance. This is then ground with water to form GGBS. The chemical composition in terms of oxides of South African GGBS is shown in Table 2.3 (Grieve, 2009). The amount of GGBS used in concrete is normally between 15% and 50% by replacement of Portland cement (C&CI, 2009).

Table 2. 3: Chemical composition of South African GGBS (Grieve, 2009).

Oxide	% by mass in GGBS
SiO_2	34 – 40
CaO	32 – 37
Al_2O_3	11 – 16
MgO	10 – 13
FeO	0.3 – 0.6
MnO	0.7 – 1.2
K_2O	0.8 – 1.3
S	1.0 – 1.7
TiO_2	0.7 – 1.4

ii. *Fly ash (FA)*

FA is a by-product of coal-fired power stations. It is extracted by electrostatic precipitation or bag filters from the flue gases of furnaces fired with pulverised coal. After extraction, only the fine fractions (about 10% retained on the 45-μm sieve) are used as an extender. The chemical composition of South African FA is shown in Table 2.4 (Grieve, 2009). The amount of FA used in concrete is normally between 15% and 30% by replacement of Portland cement (C&CI, 2009).

Table 2. 4: Chemical composition of South African FA (Grieve, 2009).

Oxide	% by mass
SiO_2	48 – 55
Al_2O_3	28 – 34
CaO	4 – 7
Fe_2O_3	2 – 4
MgO	1 – 2
Na_2O + 0.658 K_2O	1 – 2

iii. *Condensed silica fume (CSF)*

CSF is a by-product of ferrosilicon smelting process. CSF particles are extremely fine with an average particle size of 0,15µm. The particles are generally densified by means of electrostatic forces into small pellets of about 0,5 mm. The chemical composition of CSF is shown in Table 2.5 (Grieve, 2009). The amount of CSF used in concrete is normally between 5% and 15% by replacement of Portland cement (C&CI, 2009).

Table 2. 5: Chemical composition of CSF (Grieve, 2009).

Oxide	% by mass
SiO_2	92 – 96
Al_2O_3	1.0 – 1.5
Fe_2O_3	1.0 – 1.6
CaO	0.3 – 0.6
MgO	0.6 – 0.8
K_2O	1.2 – 2.0
H_2O	0.4 – 0.8

2.4 Carbonation phenomena and its associated mechanisms

Carbonation results from the interaction between cement hydration products in concrete and atmospheric carbon dioxide (CO_2). This leaves the steel reinforcement vulnerable to corrosion. It occurs when the CO_2 gas present in the atmosphere diffuses through the pore structure into the concrete and reacts with the cement hydrates such as portlandite ($Ca(OH)_2$ or CH), see Figure 2.3 (Dyer, 2014; Bohni, 2005; Bertolini, et al., 2013; Li, 2016). During carbonation, the following reactions occur (Papadakis, et al., 1991):

Uptake of CO_2 in the pore water (formation of carbonic acid)

$CO_2 + H_2O \rightarrow 2H^+ + OH^-$ Equation (2.1)

Formation of $CaCO_3$

$Ca(OH)_2 + H_2O + CO_2 \rightarrow CaCO_3 + 2H_2O$ Equation (2.2)

Further carbonation takes place if sufficient CO_2 and H_2O are available, to form soluble hydrogen carbonate

$CaCO_3 + H_2O + CO_2 \rightarrow Ca(HCO_3)_2$ Equation (2.3)

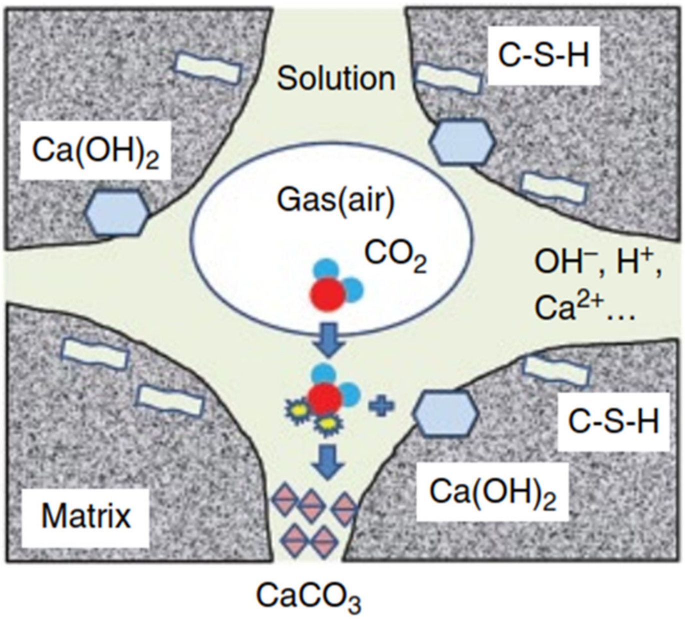

Figure 2. 3: Mechanism of carbonation at the pore level (Li, 2016).

Carbonation is a gradual process which moves as a front in the concrete. The front is referred to the line after all the $Ca(OH)_2$ at a given depth has been converted to $CaCO_3$, see Figure 2.4. This lowers the pH of the concrete pore water from a high alkaline pH of 12.5 to about 8.5

upon complete carbonation. The decrease in alkalinity causes the gamma-ferric oxide layer around the steel to become unstable and the steel is depassivated (Ballim, et al., 2009).

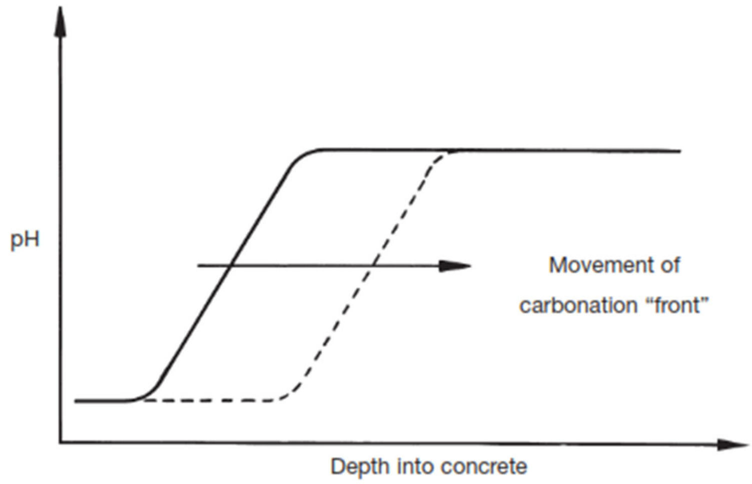

Figure 2. 4: pH reduction due to carbonation moving as a 'front' from the concrete surface (Ballim, et al., 2009).

There is no standard defined method to measure concrete carbonation. However, phenolphthalein solution is most commonly used as an indicator for the depth of carbonation, as it is quick and economical to apply. This involves spraying freshly broken or cut concrete surfaces with 1 - 2% phenolphthalein solution. The surface turns pink where the pH is greater than 9. The colourless portion represents the depth to which full or partial carbonation has occurred (Neville, 1997; Wierig, 1984).

2.5 Factors influencing carbonation

The rate of carbonation is affected by both internal and external factors to the concrete. The external factors are defined by the environmental conditions while the internal factors are defined by the concrete properties.

2.5.1 External factors

i. Relative humidity

The exposure site relative humidity determines the drying rate of concrete and thus the internal relative humidity (saturation degree of the pore structure). The relative humidity or moisture content of the concrete pore structure strongly affects the carbonation rate in concrete. Carbonation will be limited if the pores are fully saturated and will not occur when completely dry. At full saturation, water in the pores prevents CO_2 diffusion while in completely dry concrete there is insufficient moisture for the chemical reaction. Optimum relative humidity conditions for carbonation are generally between 40 to 65% (Ballim, et al., 2009; Fernandez , et al., 2004; Richardson, 2002; Kropp, 2004).

ii. Temperature

Temperature affects the carbonation rate both directly and indirectly. Carbonation is affected directly as an increase in temperature will generally enhance the CO_2 diffusion in the concrete. Furthermore, depending on the temperature, the dissolution and saturation degrees of different species in water changes. This affects the rate of reaction in the pore structure (Salvoldi, 2010; Houst & Wittmann, 2002). Carbonation is affected indirectly as temperature changes may cause micro-cracks, which is a preferential route for aggressive agents to penetrate into concrete.

iii. Cyclic wetting and drying conditions

Short frequency cycles of wetting and drying significantly reduce carbonation due to the slower rate of CO_2 diffusion through the partially saturated pores. On the other hand, long cycles of wetting and drying conditions promote carbonation as it allows CO_2 to penetrate the concrete freely. Generally, dry periods promote carbonation whereas wet periods promote corrosion if depassivation of steel has already occurred (Yunusa, 2014; Ballim, et al., 2009).

iv. Atmospheric CO_2 concentration

The rate of carbonation increases with an increase in CO_2 concentration. This is because CO_2 moves into concrete through a diffusion mechanism, which is a gradient-driven transport mechanism. Thus, the higher the concentration between the external and the internal environment of concrete the higher the CO_2 penetration (Da Silva, et al., 2009; Nischer, 1984).

2.5.2 Internal factors

The internal factors affecting carbonation are factors which have an influence on the pore structure, and the chemical and physical properties of the concrete components. A wide range of factors exist that affect the chemical and physical pore structure of concrete. The discussion below focusses on the influence of binder content, binder type, water/binder (w/b) ratio, and curing on the rate of carbonation.

i. Binder content

The binder content in concrete affects the CO_2-binding capacity. An increase in the binder content will result in an increase in the amount of hydration products (other factors constant). The carbonation rate is highly influenced by the amount of hydration products available for the reaction. Hence, for the same permeability, a concrete with greater binder content will show a slower carbonation progression (Yunusa, 2014).

ii. Binder types

The binder material in concrete consists, with very few exceptions, of Portland cement on its own or Portland cement blended with other cement extenders also known as supplementary cementitious materials (SCM). The commonly used extenders in the South African market include; ground granulated blastfurnace slag (GGBS), fly ash (FA), and condensed silica fume (CSF). Different types of binders have different microstructural and physical properties. Thus, it is essential to understand how these different binders affect carbonation.

Portland cement used with other extenders in specific proportions provide an amelioration of the pore size distribution at complete hydration (Scanlon, 1987; Baroghel-Bouny, et al., 2009). A study conducted by Tuutti (1982) revealed that this favourable effect also influences the gas permeability of the material. This beneficiary effect is attributed to the fact that products of the pozzolanic reaction of SCMs such as GGBS fill up the pores and voids resulting in a refined pore structure (Khunthongkeaw, et al., 2006; Pack, et al., 2010). Secondly, the fine particles from SCMs such as silica fume fill up the spaces and voids between larger particles, thus blocking the potential pathways of transport. However, it has also been shown that blended cements have a lower alkaline reserve i.e. less free $Ca(OH)_2$ than Portland cement concretes. As the pozzolanic materials bind with the $Ca(OH)_2$, the alkaline reserve is further reduced (Khunthongkeaw, et al., 2006). With respect to carbonation, the more favourable pore structure is faced with the lower alkaline reserve. With adequate curing, both effects are nearly compensated (Scanlon, 1987).

Many studies have reported higher carbonation in concrete containing fly ash, compared to plain PC concrete (Byfors, 1985; Ho & Lewis, 1987; Ho & Lewis, 1988; Thomas, et al., 1990; Papadakis, et al., 1992; Atis, 2003; Sulapha, et al., 2003). Other studies (Thomas & Matthews, 1992; Salvoldi, 2010) revealed that fly ash replacement of up to 30% had a similar or slightly greater degree of carbonation than PC concrete of the same strength, provided adequate curing. However, the rate of carbonation increases significantly with 50% replacement.

For silica fume, research indicates that up to 10% replacement of SF has no influence on the carbonation process of concrete but as the limit increases beyond 10%, it not only increases the carbonation depth but also increases the variation in the intensity of corrosion (Lo, et al., 2009). A study by Kulakowski, et al. (2009) on concrete samples with SF replacements of up to 20% and w/b ratios ranging from 0.3 to 0.8 provided the following conclusions. The w/b ratio governs the behaviour of the SF additions. In samples with w/b ratios between 0.45 to 0.50, the porosity of the material is the governing factor in the carbonation process whereas the consumption of $Ca(OH)_2$ has an insignificant effect on carbonation. On the contrary, for higher w/b ratios, the consumption of $Ca(OH)_2$ plays a significant role in the carbonation process.

Studies indicate that GGBS concretes have similar porosity values as plain concretes, provided adequate curing (Pigeon & Regourd, 1983). With respect to carbonation, at lower GGBS cement replacements (less than or equal to 50%), it seems that the refinement in pore structure outweighs the depletion of portlandite. But at replacements above 50%, the advantage of a more refined pore structure is not sufficient to counteract the depletion of the carbonatable material in the concrete matrix (Salvoldi, 2010; Litvan & Meyer, 1986; Osborne, 1989).

iii. Water/binder ratio

The water/binder (w/b) ratio is considered as one of the most significant parameters affecting concrete permeability. Several studies have shown that the depth of carbonation increases with an increase in w/b ratio (Fattuhi , 1986; Roy, et al., 1999; Chaussadent, et al., 2000). This is attributed to the influence of w/b ratio on the microstructural properties of concrete. In an investigation by Chaussadent, et al. (2000), where measurements were conducted on 28-day and 2-year old hydrated concrete samples with w/b ratios ranging from 0.25 to 0.60, it was observed that a wide difference exists in the microstructural characteristics for concrete with w/b ratios above 0.40 and below 0.35. Samples with w/b ratios above 0.40 showed a higher porosity and an increase in the amount of $Ca(OH)_2$. Several studies on the relationship between w/b ratios and carbonation have shown that carbonation increases with an increase in w/b ratio (Grieve, 2009; St. John, et al., 1998; Roy, et al., 1999).

iv. Curing

The influence of curing on carbonation can be in terms of curing method and in terms of initial curing periods. When it comes to the effects of different curing methods on natural carbonation of reinforced concrete (RC) structures, very few studies exist in the literature (Ekolu, 2015). Nevertheless, few studies can be found for different curing methods studied in the laboratory (Neville, 1995; Ekolu , 2006; Lo, et al., 2016).

In a study by Lo and Lee (2002) to examine the effect of curing methods on the depth of carbonation for different concrete samples, it was observed that the carbonation depth increases with an increase in w/b ratio and age of concrete. It was also observed that the differences in carbonation depths between air-cured and water-cured samples were very small, with the differences further reducing over time.

Ekolu (2015) in a critical review of the effect of long-term natural carbonation cited a study by the CSIR (Xue et al., 2015), which concluded that none of the different curing regimes caused a change in the carbonation behaviour for outdoor concrete structures. The study also revealed that the strength class was the determining factor in the carbonation behaviour, with curing methods influencing carbonation only if they cause a change in the strength class of the mixture.

The initial curing period is critical for carbonation of RC structures. Proper initial curing greatly reduces the carbonation potential of a structure. Kunal et al. (2018), citing a study conducted by Balayssac et al. (1995) explain that although the curing period affects carbonation, the greatest influence occurs from 1 to 3 days after casting of the concrete, followed by slower variation until 28 days after casting.

2.6 Existing carbonation models relevant to the South African Durability Index Approach

Several mathematical models have been developed to predict carbonation. Results from these studies show that the carbonation depth development is a function of the square root of time (Meyer, et al., 1967; Tuutti, 1982; Shigeyoshi , et al., 1986; Nagataki & Ohga, 1988; Qiu-Dong, 1987).

$$X_c = A\sqrt{t} \qquad \text{Equation (2.4)}$$

Where X_c is the carbonation depth (mm), t is the time (years) and A is the carbonation coefficient (mm/annum).

The carbonation coefficient is determined empirically and has been evaluated using different approaches by researchers. The main intent being to correlate the properties of concrete to the depth or rate of carbonation. This section reviews various carbonation models relevant to the South African Durability Index approach.

Numerous attempts have been made by researchers in South Africa to model the rate of carbonation in concrete structures. These include amongst others attempts by (Ballim, 1994; Mackechnie, 1996; Ballim & Lampacher, 1996; Lampacher, 2000; Mackechnie & Alexander, 2002; Yam, 2004; Alexander, et al., 2007; Salvoldi, 2010; Yunusa, 2014).

Ballim (1994) was the first to develop a predictive model for carbonation in South Africa, using the fluid transport properties of concrete. He developed a relationship between oxygen permeability and water Sorptivity respectively. The relations are shown in Equation 2.5 and 2.6 below.

$$X = \lambda_9 \, . \ln(k) + \lambda_{10} \qquad \text{Equation (2.5)}$$

$$X = \lambda_{11} . S + \lambda_{12} \qquad \text{Equation (2.6)}$$

Where, X is the carbonation depth (mm), k is the oxygen permeability, S the water Sorptivity, λ_9 *to* λ_{12} are coefficients dependent on the binder type

Next is a model developed by Mackechnie & Alexander (2002), using 28-day oxygen permeability values to characterize the pore structure of the concrete. In the study, plain and blended cement concretes were exposed to marine environment of South Africa for a period of 6 years, with measurements done at intervals of 1, 4 and 6 years. The relationship developed is presented in Equation 2.7.

$$X = K_c t^{0.4} \qquad \text{Equation (2.7)}$$

Where, X is the carbonation depth (mm), K_c the material coefficient and t the time (years).

Studies by Ballim, (1994), Mackechnie & Alexander (2002) represent a significant contribution in modelling carbonation. The models developed by these authors however do not allow for predicting the performance of concrete in environments other than marine environments. In addition, the models do not consider the chemistry of carbonation in terms of concrete chemical composition.

Lampacher (2000), in a continuation of the study initially developed by Ballim & Lampacher (1996) to determine the durability performance of existing structures in the inland environment of South Africa, proposed an average carbonation rate of $3.16\text{mm}/\sqrt{\text{years}}$. The empirical value does not relate the parameters that influence carbonation and equally fails to provide a relationship between the carbonation depth and either the OPI or the WSI value.

Salvoldi (2010) assessed the oxygen permeability index of plain and blended cement concretes, as well as accelerated carbonation test on companion concretes. The purpose was to develop a performance-based model for predicting carbonation of RC structures that can be applied to various environments. Using OPI values, relative humidity, CO_2 concentration and the chemical composition, Salvoldi (2010) proposed the regression model shown in Equation 2.8.

$$X_c(t) = \sqrt{\frac{2Dc\beta t_e}{a}} \qquad \text{Equation (2.8)}$$

Where, $X_c(t)$ is the carbonation depth (mm), t_e the effective exposure time (years), β the relative humidity factor, D the effective diffusion coefficient, c is the ambient CO_2 concentration (mol/m^3) and *a* the amount of carbonatable material (mol/m^3)

Salvoldi's model covers a wide range of factors that influences the carbonation process. The model can be used for different exposure conditions and considers different concrete types. Though the model was based on accelerated carbonation measurements, a factor is introduced to correlate the results to natural carbonation. Salvoldi's model was used as a basis model for this study.

Yunusa (2014) developed a series of empirical relationships for predicting carbonation of RC structures in inland environments. The developed models consider the concrete materials, processing as well as the climatic exposure conditions. The correlation of carbonation rate with early age concrete mix-design parameters gave rise to the carbonation equations presented in Table 2.6.

Table 2. 6: Carbonation model for service life design of RC structure (Yunusa, 2014).

Model	**Prediction equation**
Model 1: Mixture Design	$K_{nat} = 5.63 + 10.01(w/b) - 0.02(C) + \alpha_1 + \beta$
Model 2: Compressive Strength	$K_{nat} = 14.26 - 0.18(f_c) + \alpha_2 + \beta$
Model 3: Oxygen Permeability Index	$K_{nat} = 41.24 - 0.07(f_c) - 2.53(OPI) - 0.57(CH) + \beta$
Model 4: Water Sorptivity	$K_{nat} = 13.75 - 0.06(f_c) + 0.34(WS) - 0.63(CH) + \beta$
Model 5: Accelerated Carbonation Rate	$K_{nat} = 1.44 + 1.15(K_{acc}) + \alpha_3 + \beta$

Where, K_{nat} is the natural carbonation rate (mm/$\sqrt{\text{years}}$); w/b the water/binder ratio; C the cement content, in kg/m^3; f_c the 28-day compressive strength of concrete, in MPa; OPI the 28-day Oxygen Permeability Index; WS the 28-day Water Sorptivity Index; CH the 28-day calcium hydroxide content; K_{acc} the 28-day accelerated carbonation rate; $\alpha_{1,2,3}$ relates to the curing duration (Table 2.7) and β to the exposure condition (Table 2.8).

Table 2. 7: Model coefficients for curing duration

Curing duration	α_1	α_2	α_3
28 days	0	0	0
7 days	+1.61	-1.56	+0.74
3 days	+2.90	-1.79	+1.31

Table 2. 8: Model coefficients for exposure condition

Exposure condition	β_1
Indoor	0
Outdoor sheltered	+1.07
Outdoor exposed	+2.07

The series of model equations developed by Yunusa (2014) is quite comprehensive, with the advantage that the study was developed based on natural carbonation processes. Nevertheless, his study provides different model equations which give different carbonation depth values for a single concrete sample.

2.7 Probabilistic approach to durability design

Current standards (SANS 10100, 2014; EN 206-1, 2013) deal with durability of new RC concrete structures by specifying that such structures satisfy certain minimum requirement for concrete composition and cover (prescriptive approach). The past two decades have seen the development of probabilistic design concepts to address the carbonation and chlorides induced corrosion problems. This can be seen for example with the DuraCrete project, Probabilistic Performance-based Durability Design of Concrete Structures (DuraCrete, 2000) and CEB FIB Model Code (fib, 2006). The probabilistic approach permits durability design which accounts for uncertainties and variabilities in the physical and material parameters.

Modelling the service life of RC structures using the probabilistic approach is made possible through the application of reliability theory as that applied in structural design (Melchers, 1999). That is, using statistical databases and probability theory to give a relative measure of the likelihood of the structure to perform its function during the designed service life (ISO 13823, 2008). The probabilistic methods make use of the limit state methodology (LSM), which incorporates the use of mathematical models that describe the deterioration mechanism. The limit-state is defined as the border that separates the desired state from the undesired state.

ISO 13823 (2008) recognises three types of limit state; the ultimate limit state (ULS), which depicts the point in which the safety of the structure is deemed at risk. The serviceability limit state (SLS), which defines the situation where the functionality of the structure is affected (e.g. deflection, cracking) and the initiation limit state (ILS), which precede both the ULS and the SLS and describes the onset of quantifiable deterioration (e.g. corrosion initiation). This study focuses on the durability design of RC structures against carbonation; hence the relevant limit-state is the ILS. However, in practice the relevant limit state is specified based on technical, economical, functional and social considerations.

2.7.1 Probability limit state

A limit state is the border that separates the desired state from the undesired state (ISO 13823, 2008). The limit state should satisfy the primary demand of the structure, set by the owner or the authorities (e.g. building Codes). When the primary demand is specified, it is translated into a functional demand by the architect and/or engineer. From the functional demand, the performance of the structure can be expressed with a performance function. The performance function is expressed as a number of basic parameters and then compared with a defined limit state (Equation 2.9). For carbonation, the variables define the material properties, geometric properties and the environmental properties.

$$G\{X_1, X_2, \ldots, X_i, \ldots, X_n\} \leq L$$

Or Equation (2.9)

$$G\{X_1, X_2, \ldots, X_i, \ldots, X_n\} \geq L$$

Where G is the performance function, X_i the i^{th} parameter, n the total number of parameters; and L is the limit state.

For engineering problems, it is convenient to divide the performance function (Equation 2.9) into two parts representing the 'action effect' and the 'resistant effect' respectively, see Equation 2.10. This function is called the limit state function (ISO 13823, 2008).

$$S(X_1, X_2, \ldots, X_i) - R(X_{i+1}, X_{i+2}, \ldots, X_n) \geq 0 \qquad \text{Equation (2.10)}$$

Where, S is a function (model) that describes the deterioration process, R is a function (model) that describes the resistance of the RC structure, X_i is a basic variable.

The action effect *S* that acts on the structure is random in nature and varies with time. Similarly, the resistant effect R is also a function of time. Therefore, the probability of failure is also a time-dependent quantity. The measure of risk associated with the specific event of $S(t) \geq R(t)$

can be expressed as the probability of failure P_f (t) of that event, or generically as:

$$P_f(t) = P[R(t) - S(t) \leq 0] \qquad \text{Equation (2.11)}$$

The basis of the limit state approach is that when a structure goes into the undesired state (adverse state), it does not meet its durability requirements. With this approach, every structure can be put into two different classes:

- Safe state (S < R): If R - S > 0, the structure is in the safe state and if R - S = 0, the structure is in a state on the border between the safe and unsafe state.
- Unsafe state (R > S): If R - S < 0, the structure is in an unsafe state.

According to Equation 2.11, the probability of failure is a time dependent function as schematically represented in Figure 2.5. The resistance R is represented by a distribution that varies with time. The same applies to the load S. With time, the two distributions approach each other forming an over lapping area of increasing size that can be graphically interpreted as the probability of failure.

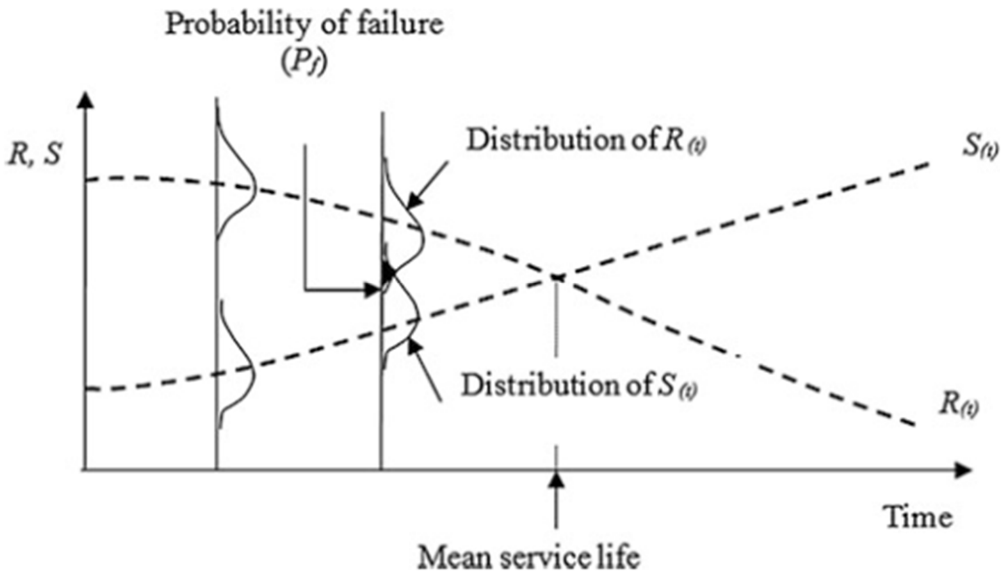

Figure 2. 5: Illustration of the decrease of ‘‘resistance’’ with time and increase of ‘‘load’’ with time (DuraCrete, 2000; Sarja & Vesikari, 2005; ISO 13823, 2008)

There are two approaches to durability problems. The two approaches are equivalent in principle: intended service life and; lifetime design (DuraCrete, 1998).

The intended service life considers that the probability of exceeding a specified limit state should not be greater than a target probability, within the intended service life period. The concept of the intended service life period described by Equation 2.12.

$P_{f,T}\ [R(t) - S(t)]_T \leq P_{target} = \emptyset(-\beta)$ Equation (2.12)

Where $P_{f,T}$ is the probability of failure of the structure within the service life period T.

T is the intended service life period, P_{target} the target probability of failure, ∅ the standard normal distribution function, and β the reliability index.

In the lifetime design the reliability of the structure considers the probability that the design service life is exceeded. The service life is ended once the limit state is reached.

2.7.2 Reliability index β

The probability of failure can be expressed through a performance function G. As seen in the previous section, if R is the resistance effect and S the action effect, the performance function is G = R - S. The variable G represents the reliability with which the component fulfils the required limit state. When the resistance and the load variables are independent and each described by a known probability density function, the resulting performance function will be normally distributed (Rui, 2004). If G = R - S is normally distributed, the reliability index β is taken as:

$$\beta = \frac{\text{Mean value of G}}{\text{Standard deviation of G}} = \frac{\mu_G}{\sigma_G} \qquad \text{Equation (2.13)}$$

$$\mu_G = \mu_R - \mu_S$$

$$\sigma_G = \sqrt{\sigma_R^2 + \sigma_S^2}$$

Where β = reliability index, μ = mean value and σ = standard deviation value.

The reliability index may be transformed into a standard normal distributed one, for which μ = 0 and σ = 1. The failure probability is then equal to the probability distribution function of the standard-normal distribution of β (Faber & Sørensen, 2002).

$$P_f = \emptyset\left(-\frac{\mu_G}{\sigma_G}\right) = \emptyset(-\beta) \qquad \text{Equation (2.14)}$$

Where P_f is the probability of failure, Φ(·) the probability distribution function of standard normal distribution, β the reliability index, μ_G the mean value of reliability G, σ_G the standard deviation of the reliability G.

2.7.3 Target reliability index β

EN 1991-1, (2002), NS 3490 (1999), provides reliability values for typical cases like:

- ULS (residential and office buildings where consequences of failure are medium) – β is 3.8 for a 50 years reference period.
- SLS – β equals 1.5 for a 50 years reference period.

Table 2.9 and 2.10 present reliability indexes for service life design and depassivation respectively.

Table 2. 9: 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	1.5

Table 2. 10: Reliability index β_{SLS} for depassivation of reinforcement according to TG 5.6 of (fib, 2006).

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

2.8 Methods of calculating failure probability

The probability of failure can be assessed using several reliability methods. The First-order reliability method (FORM), second-order reliability method (SORM) and the Monte Carlo Simulation (MCS) method (Hasofer & Lind, 1974; Liu & Der Kiureghian, 1990; Mackechnie, 1996). In the past decades, the FORM and SORM methods were the methods used for reliability analysis (Ang & Tang, 1984). These methods transform a reliability problem into an approximate optimization problem, which makes use of mean and standard deviation values of the random variables together with the linearized form of the performance function. The

FORM is generally suitable if the performance function is linear. For nonlinear functions, the SORM is used, as it improves the assessment given by the FORM (Emilio, Abdul, 2014). Using the FORM and SORM generally comes with a number of assumptions and trade-offs such as: a) limitation in the number of variables; b) requires a knowledge on solving non-linear optimization problems.

Limited number of variables (model parameters) considered is difficult to meet, when it comes to deterioration problems. This is because such problems are generally associated with numerous random variables. Also, engineers may not have the knowledge and skills for solving non-linear optimization methods. This makes the FORM and SORM not easy to implement, even for simple reliability problems. With the availability nowadays of personal computers (PCs) and laptops, massive computations are possible. Specifically, the Monte Carlo Simulation method (MCS) can now be implemented for the purpose of reliability analysis.

2.8.1 Monte Carlo Simulation (MCS) method

Monte Carlo simulation (MCS) methods are statistical simulation methods that utilize sequences of random numbers to perform the simulation. It consists of drawing samples of each random variables several times to represent their real distribution according to their probabilistic characteristics and feeding into the performance function. The performance function is then solved deterministically for each realization. This is termed a simulation cycle or a trial. Using several simulation cycles gives the overall probabilistic characteristics of the problem, particularly when the number of cycles N tends to infinity.

MCS method has evolved as a very powerful tool for engineers to evaluate the risk of complex engineering systems (Ching, 2011). The method allows for the physical process to be simulated directly without having to derive the differential equations that describe the behaviour of the system. This makes it a suitable method to implement for complex systems with many variables; hence, the limitations of the FORM and SORM methods can be overcome. The only requirement of the MCS method is that the physical system be described by probability density functions (PDF's). Such a function is in many cases characterized by a particular type of distribution (e.g. a normal distribution or a log-normal distribution), a mean value and a standard deviation. The procedure for MCS is described in Figure 2.6.

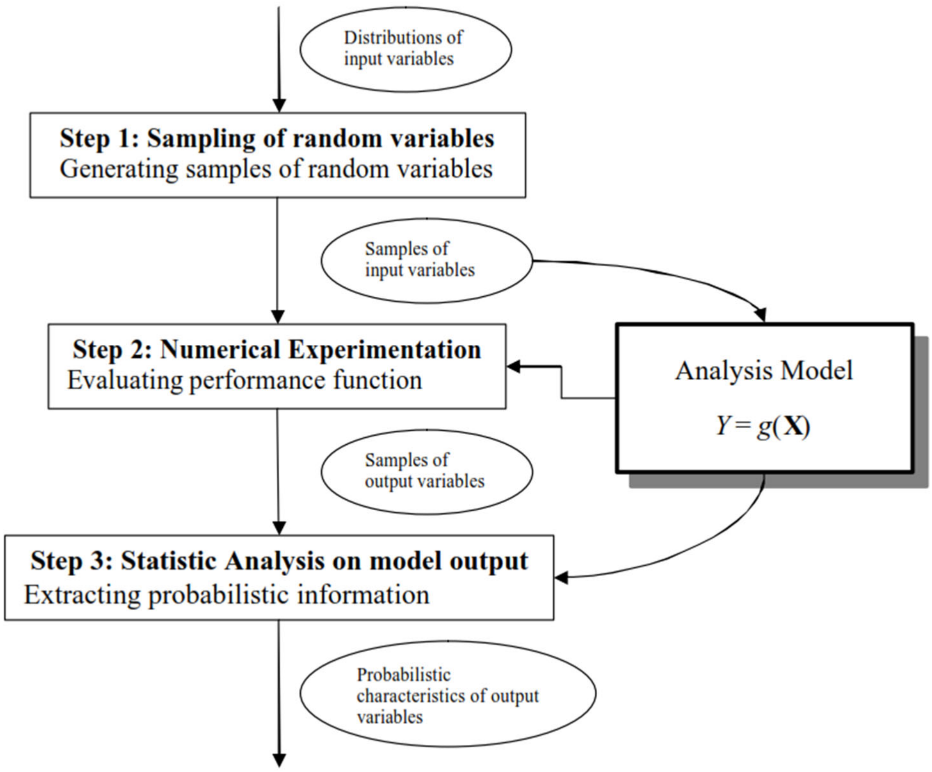

Figure 2. 6: The principle of Monte Carlo simulation (CSEP 1995)

MCS techniques involve sampling at random to simulate a large number of experiments artificially and to observe the results. In the case of reliability analysis, this means sampling each random variable X_i to give a sample value x_i. The limit state function $G(x) = 0$ is then computed. If $G(x) < 0$, the limit state has been violated (depassivation has occurred). The experiment is repeated several times. If N trials are conducted, the probability of failure is given by:

$$P_f = \frac{n(G(x_i) < 0)}{N} \qquad \text{Equation (2.15)}$$

Where P_f is the failure probability, $n(G(x_i) < 0)$ denotes the number of trials for which $G(x_i) < 0$. N is the total number of trials.

The standard error of the probability of failure is estimated by:

$$s = \sqrt{\frac{P_f(1 - P_f)}{N}} \qquad \text{Equation (2.16)}$$

Where P_f is the failure probability, and N denotes the total number of trials.

Here, the physical process is simulated directly using the models mentioned earlier to describe the behaviour of the phenomena. The main requirement here is that the physical system be described by PDF's.

2.9 Summary

A review of the South African durability index approach and the OPI test method has been presented in this chapter. A review of the common cementitious materials available in the South African market has been presented. This include; Portland cement, masonry cement and cement extenders which are sold separately. Their composition, types and properties have also been presented.

Carbonation is a process that results from the interaction between cement hydration product in concrete and the atmospheric CO_2, which leaves the steel reinforcement vulnerable to corrosion. A detailed review discussion on the carbonation mechanism and the factors that influence carbonation has been discussed in this chapter. The numerous attempts made by researchers in South Africa to model the rate of carbonation in RC structure was further presented in Section 2.6.

The past two decades have seen the development of probabilistic design concepts to address the carbonation induced corrosion problems. The probabilistic approach permits a sound durability design, which accounts for the uncertainties and variabilities in the physical and material parameters. A review of the reliability theory, the limit state approach and the MCS method used to assess the probability of failure of RC structures has been presented in this chapter.

Although several carbonation models have been developed to predict carbonation in RC structures, these models are generally too detailed and require much knowledge on the carbonation process and the cement chemistry to be used. Furthermore, the models are deterministic in nature and fail to account for the variability in the different carbonation parameters. There is therefore a need to develop a carbonation model that can easily be used by engineers in practice. The proposed model presented in the sections to follow is based on the existing carbonation model by Salvoldi (2010), but with alterations made such that the model is made simple but physically and chemically correct. From a probabilistic point of view, the input parameters are described using distribution functions to account for variability.

CHAPTER 3

3. Proposed probabilistic model for service life design of carbonating structures

3.1 Introduction

The introduction of probabilistic-based service life assessment of RC structures has shown to be valuable for durability design (DuraCrete, 1998; DuraCrete, 2000; McGee, 1999). Time-dependent carbonation models relevant to the South African DI approach were reviewed in the previous chapter. However, the proposed models are deterministic and generally too detailed, requiring too much data to be used by the engineer. This study was limited to providing a practical design tool based on an existing model, that can be used for service life design of carbonating structures while incorporating probabilistic principles in the model predictions.

The proposed probabilistic model is based on the same background as the carbonation model developed by Salvoldi (2010), with some alterations made in order to make it user friendly and functional. The model was developed using MS Excel. The work was divided into two phases. Phase one comprised the determination of parameters relevant for carbonation and the relevant statistical information. Phase two comprised the modelling of service life and the probability of exceeding the limit state in an MS Excel spreadsheet. The Monte Carlo simulation method was used to compute the probability of failure. The tasks performed in are summarised in Figure 3.1.

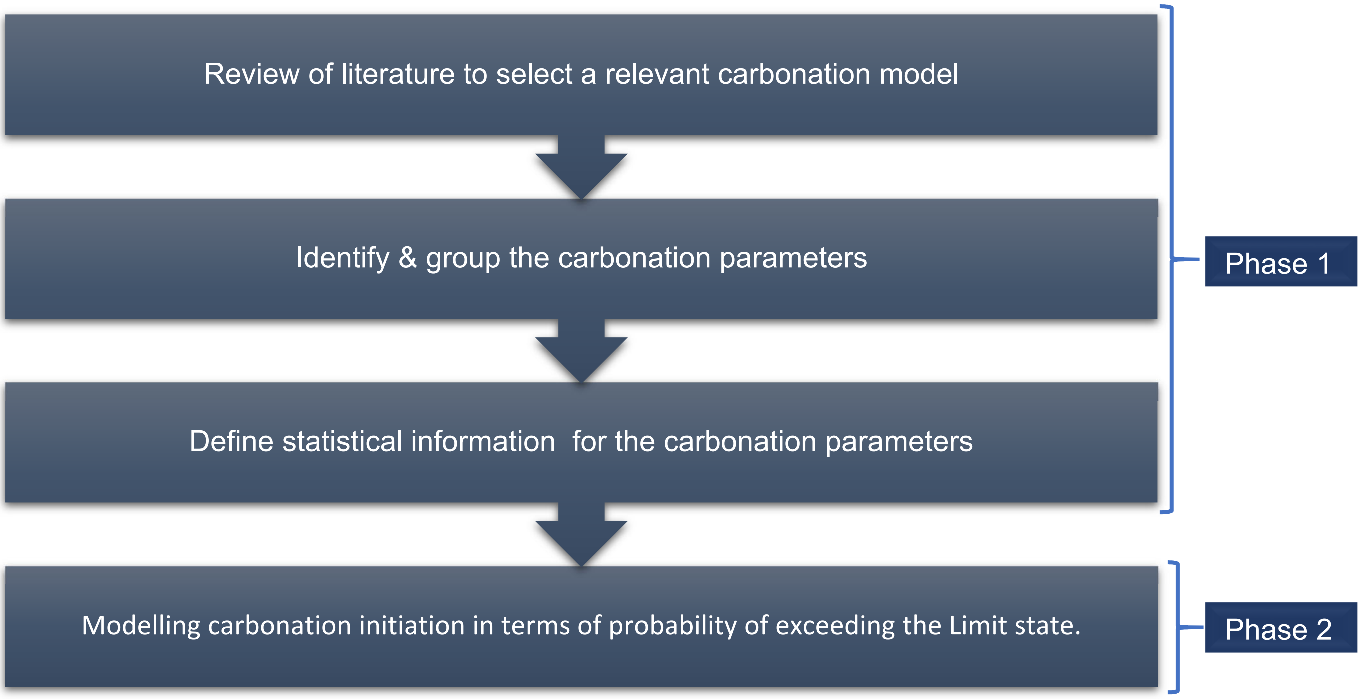

Figure 3. 1: Flow chart of methodology for the research

3.2 Carbonation modelling

The carbonation model developed by Salvoldi (2010) is used as background for this study. The model is based on Fick's 1st law of diffusion, commonly used to model CO_2 penetration in concrete. Fick's 1st law of diffusion is given by:

$$J = -D\frac{dc}{dx}\left(\frac{mol}{m^2s}\right) \qquad \text{Equation 3.1}$$

Where J is the flux, D is the diffusion coefficient, c is the CO_2 concentration and x is the carbonation depth.

Assuming carbonation is a front that moves through the concrete matrix with a gradient of $c_2 - c_1$, where c_1 is the concentration at the concrete surface and c_2 is the concentration at the front, Equation 3.1 can be written as:

$$J = D\frac{(c_1 - c_2)}{x}\left(\frac{mol}{m^2s}\right) \qquad \text{Equation 3.2}$$

The flux of CO_2 diffusion through an area A of concrete can equally be written as:

$$J = \frac{Q}{At}\left(\frac{mol}{m^2s}\right) \qquad \text{Equation 3.3}$$

Where Q (mol) is the amount of CO_2 diffusing and t is the time.

The amount of CO_2 needed to carbonate the carbonatable material a is given by:

$$Q = aAdx \quad (mol) \qquad \text{Equation 3.4}$$

Substituting Equation 3.4 into Equation 3.3 and equating to equation 3.2 gives:

$$xdx = \frac{D(c_1 - c_2)}{a} \times t \qquad \text{Equation 3.5}$$

Integrating Equation 3.5 gives:

$$x^2 = \frac{2D(c_2 - c_1)}{a} \times t \qquad \text{Equation 3.6}$$

Assuming CO_2 concentration at the front is close to zero and taking the square root of each side gives:

$$x = \sqrt{\frac{2Dc}{a}} \times \sqrt{t} \qquad \text{Equation 3.7}$$

Where, x is the carbonation depth, D the diffusion coefficient, c the ambient CO_2 concentration (mol/m^3), a the amount of carbonatable material in the concrete matrix (mol/m^3), and t is the time.

Salvoldi's model makes some adjustment on the general diffusion equation above (Equation 3.7) by including a relative humidity factor β to account for the effect of relative humidity on the rate of carbonation. When adding the relative humidity factor, Equation 3.7 becomes (Salvoldi, 2010):

$$x = \sqrt{\frac{2Dc\beta}{a}} \times \sqrt{t} \qquad \text{Equation 3.8}$$

Where, x is the carbonation depth, D the diffusion coefficient, β the relative humidity factor, c the ambient CO_2 concentration (mol/m^3), a the amount of carbonatable material in the concrete matrix (mol/m^3), and t is the time.

Transposing Equation 3.8 to make t the subject of the formula gives Equation 3.9. Equation 3.9 can be used to estimate the service life of a structure, when given the cover depth, diffusion coefficient, molar concentration of CO_2, the relative humidity factor and the molar concentration of carbonatable material.

$$t = \frac{x^2 a}{2Dc\beta} \qquad \text{Equation3.9}$$

Where, x is the carbonation depth, t is the time required for the carbonation front to reach a depth x, D the diffusion coefficient, β the relative humidity factor, c the ambient CO_2 concentration (mol/m^3), a the amount of carbonatable material in the concrete matrix (mol/m^3).

The following simplification were made in developing the model (Salvoldi, 2010).

1. The depth of carbonation is given by a clear reaction front between carbonated and uncarbonated material
2. The amount of carbonatable material is constant throughout the concrete matrix.

3.3 Carbonation parameters

The carbonation model developed by Salvoldi (2010) considers several parameters (diffusion coefficient, ambient CO_2 concentration, relative humidity factor and the amount of carbonatable material in the concrete matrix) in estimating service life for a given cover depth value. For this study, the various parameters will be categorized into concrete mix parameters, exposure parameters, geometric parameters and parameters describing the concrete micro-structure. These parameters are summarized in Table 3.1.

Table 3. 1 : Carbonation parameters

Parameters	**Input data**	**Output parameter**
Concrete mix parameters	Binder type, binder content	Molar concentration of carbonatable material
Exposure parameters	Land use	Molar concentration CO_2
	Project location	Relative humidity factor
Geometric parameters	Concrete cover	Concrete resistance depth
Concrete micro-structure parameter	OPI value	Concrete diffusion coefficient

3.3.1 Concrete mix parameter

The concrete mix parameter describes the amount of carbonatable material in the concrete matrix (mol/m^3). For the purpose of the model, it was assumed that the molar concentration of carbonatable material is constant throughout the matrix.

In natural conditions CO_2 reacts with both calcium hydroxide (CH) and calcium silicate hydrate (CSH), with the reaction layer of CSH lagging behind the reaction layer of CH. As the carbonation of CSH is not associated with the drop of pH, the reaction layer of CH is the more critical one for durability.

When pozzolans or slags are added to concrete the portlandite is consumed to further produce CSH gel. Borges (2010) states that other researchers found that the extent of carbonation of CSH gel is dependent on the permeability of the concrete, with carbonation of CSH gel more rapid at high permeability. Given that the concrete used for structural projects is generally concrete of low permeability, the carbonation of CSH gel was assumed to be minimal and not to influence the progress of the CH carbonation front.

However, the extent of the CSH gel carbonation at lower permeabilities in natural conditions needs to be investigated further for accurate calculation of the amount of carbonatable material in concrete structures.

Using the binder content and composition, the age, the curing humidity and the average ambient temperature, the amount of carbonatable material (portlandite) available in concrete can be estimated. Typical compound composition of South African PC and cement extenders presented in Section 2.4 were used in this study. Knowing the binder composition and the mass of binder used per cubic metre enables calculation of the number of moles of each compound available in each cubic metre of concrete (Equation 3.10). A mass balance equation can then be evaluated to estimate the amount of portlandite available after hydration.

$$[\,i\,] = \frac{\%\text{ composition of i} \times \text{binder content}}{M} \qquad \text{Equation 3.10}$$

Where [i] is the molar concentration of a given compound i, M is the molar mass of the compound.

3.3.1.1 Mass balance of portlandite in the system

Calcium hydroxide (CH) is created by the hydration of Alite and Belite and is partially consumed by the hydration of the other cementitious compounds. Furthermore, calcium hydroxide is consumed by pozzolanic reactions that are initiated in the presence of calcium hydroxide. Thus, a mass balance equation of the CH in the system is equal to CH produced by hydration less CH consumed by hydration and pozzolanic activity (Papadakis, et al., 1991). The mass balance of portlandite for both Portland cement and any blended cement can be written as (Papadakis, et al., 1991):

$$[CH] = 1.5[C_3S] + 0.5[C_2S] - 4[C_4AF] - [C_3A] + [C\hat{S}H_2] + [C] - 4[A] - 1.5[S] \qquad \text{Equation (3.11)}$$

3.3.1.2 Hydration and pozzolanic activity

Only a certain amount of the compounds in Portland cement are hydrated, depending on the degree of hydration. Similarly, only a certain amount of compounds in pozzolans and slags react depending on the pozzolanic activity. Defining the degree of hydration as F_i, where i denotes the species, and the degree of pozzolanic activity as P_j, where j denotes the species, the Equation (3.11) becomes:

$$[CH] = 1.5[C_3S]F_{C_3S} + 0.5[C_2S]F_{C_2S} - 4[C_4AF]F_{C_4AF} - [C_3A]F_{C_3A} + [C\hat{S}H_2] + [C]P_c - 4[A]P_A - 1.5[S]P_S \qquad \text{Equation (3.12)}$$

Where [CH], $[C_3S]$, $[C_2S]$, $[C_4AF]$, $[C_3A]$, $[C\hat{S}H_2]$, [C], [A], [S] are the molar concentration of calcium hydroxide, tricalcium silicate, dicalcium silicate, tetracalcium aluminoferrite , tricalcium aluminate, gypsium, Calcium oxide (CaO), Aluminium oxide (AL_2O_3), and Silicorn dioxide (SiO_2) compounds respectively.

F_{C_3S}, F_{C_2S}, F_{C_4AF}, F_{C_3A} indicates the degree of hydration of tricalcium silicate, dicalcium silicate, tetracalcium aluminoferrite and tricalcium aluminate compound respectively.

P_C, P_A, P_S indicates the degree of pozzolanic activity of Calcium oxide (CaO), Aluminium oxide (AL_2O_3) and Silicon dioxide (SiO_2) compounds respectively.

The hydration model developed by Papadakis, et al. (1991) is used to evaluate the degree of hydration F_i (Parrott , 1988; Kada-Benameur, et al., 2000).

$$F_i = \alpha_{RH} \cdot \left[1 - \left(1 - K_i \times \alpha_T \times t \,.\, (1 - n_i)^{\frac{1}{(1 - n_i)}}\right)\right] \qquad \text{Equation (3.13)}$$

The values of K_i and n_i are empirical factors evaluated experimentally (Table 3.2) (Papadakis, et al., 1991).

Table 3. 2 : Values for n_i and K_i for different compounds in cement. Adapted from (Papadakis, et al., 1991)

Compound	**C_3S**	**C_2S**	**C_2AF**	**C_3A**
n_i	2.65	3.10	3.81	2.41
K_i(day-1)	1.17	0.16	1.00	2.46

The degree of hydration is dependent on the ambient humidity, temperature and the age of concrete. The coefficient α_{RH} and α_T were developed to account for the effect of humidity and temperature on the degree of hydration (Parrott , 1988).

$$\alpha_{RH} = \left[\frac{RH - 0.55}{0.45}\right]^4 \qquad \text{Equation (3.14)}$$

$$\alpha_T = e^{\left[\frac{E_o}{R}\left(\frac{1}{293} - \frac{1}{T}\right)\right]} \qquad \text{Equation (3.15)}$$

Where T = temperature in kelvin, R = universal gas constant (R = 8.314 J/mol), E_0= average activation energy (E_o= 38.2KJ/mol), RH = ambient relative humidity (%)

Similarly, the degree of pozzolanic activity P_i is evaluated using Equation 3.16 (Bahador & Cahyadi, 2009).

$$P_i = \alpha_b \, . \, \alpha_{RH} \, . \left[1 - \left(1 - K_j \times \alpha_T \times t \, . \, (1 - n_j)^{\frac{1}{(1 - n_j)}}\right)\right] \qquad \text{Equation (3.16)}$$

α_b is a factor that accounts for the pozzolanic activity of mineral admixtures and is calculated according to Equation 3.17.

α_b = 1 - Crystalline phase content Equation (3.17)

The crystalline phase content is the fraction of minerals in the additives that are unreactive. The values of n_j and K_j are empirical factors evaluated experimentally (Table 3.3) (Bahador & Cahyadi, 2009).

Table 3. 3 : Values for n_j and K_j for different mineral compounds. Adapted from (Bahador & Cahyadi, 2009).

Mineral	**Silica fume**	**Slag**	**High CaFA**	**Low CaFA**
n_j	2.9	3.6	2.8	6.0
K_j	$\frac{1}{\sqrt{250d_n}}$	$\frac{1}{\sqrt{50d_n}}$	$\frac{1}{\sqrt{100d_n}}$	$\frac{1}{\sqrt{100d_n}}$
α_b	0.5	45	20	20

In practice, information about the binder composition and binder content are generally well documented and easy to check for compliance as in each concrete batch, the mass of the binder is measured using a scale balance. Added to this is the high-quality control measures usually implemented in ready mixed concrete plants. This results in less variability in the documented binder content and the as-mixed binder content. Following the above discussion, the amount of carbonatable material is a value mainly dependent on the binder type, the binder content, the age of concrete and the curing temperature.

3.3.2 Exposure parameters

The rate of carbonation is closely related to the exposure environment. The exposure parameters here define the conditions to which the structure will be subjected to throughout its service life. The variables which define the exposure conditions are; the relative humidity of the carbonating layer and the ambient carbon dioxide concentration.

3.3.2.1 Relative humidity

The relative humidity (RH) of the carbonating layer was considered as it affects the carbonation rate. Due to the difficulty in obtaining such data and since carbonation takes place in the outermost layer (cover concrete), the RH of the ambient air around the structure was considered. The average RH values for different locations in South Africa based on mean hourly values for the period between 1932 and 1950 presented in Table 3.4 was used (Alexander & Beushausen, 2009). Average RH value was chosen as it is more representative of the humidity value around the structure. The impact of RH on the carbonation depth is described by the relative humidity factor β (Salvoldi, 2010).

$$\beta = 23.32 \cdot (1 - RH)^2 \cdot (RH)^{2.6} \qquad \text{Equation (3.18)}$$

Where RH is the ambient relative humidity.

Table 3. 4 : Average relative humidity in different localities in South Africa (based on mean hourly values for the period between 1932 and 1950) (Alexander & Beushausen, 2009).

Location	**Average relative humidity (%)**
Bloemfontein	53
Cape Town	78
Durban	78
Johannesburg	59
Kimberley	46
Port Elizabeth	79
Pretoria	60
Upington	39

3.3.2.2 Ambient carbon dioxide concentration

The CO_2 concentration of the ambient air is an important exposure parameter affecting the carbonation rate (Parrott, 1987). However, very few studies exist in literature on its effect on carbonation, most especially with regard to outdoor natural carbonation. The CO_2 concentration levels vary geographically across cities and rural areas. This variation came to the attention of scientists in 1998, with the discovery and characterization of the urban CO_2 dome of Phoenix, Arizona, USA by Idso et al. (2001). In this study, six exposure environments (See Table 3.7) were selected with varying CO_2 concentration as follows:

i. Indoor exposure

Indoor levels of CO_2 concentration are usually higher than outdoor concentration (Szczurek, et al., 2015). It is difficult to adequately characterise indoor CO_2 concentration, since it is a function of several factors; (1) the number of people present, (2) how long it has been occupied, (3) the amount of ventilation, and (4) the outdoor CO_2 concentration (Szczurek, et al., 2015). However, typical CO_2 concentration levels normally observed in indoor environments range from 350 to 2,500 ppm (Seppänen, et al., 1999). Currently, the American Society of Heating, Refrigeration, and Air-conditioning Engineers (ASHRAE) recommends a minimum office building ventilation rate of 10 Ls^{-1}per person, corresponding to an approximate steady state indoor concentration of 870 ppm (ASHRAE, 1999). Due to the lack of data on the indoor CO_2 concentration in South Africa, the value recommended by the ASHRAE was used for this study.

ii. City/urban area

Cities due to their dense population, settlement structure, transportation networks and energy use can alter the state of the atmosphere and climate (Mills 2007). The average outdoor air concentration of CO_2 is in the order of 300 to 400 ppm. Research regarding the urban atmospheric CO_2 concentration has been performed in many parts of the world (Table 3.5). The results obtained from various studies conducted at different locations round the world show some commonalities. Data provided by the Commonwealth Scientific and Industrial Research Organisation (CSIRO, 2018) indicates that typical concentrations of CO_2 in urban areas is between 350 and 600 ppm. This can equally be observed as values in the same range have been reported from measurements conducted in different cities round the world. An average CO_2 concentration of 475 ppm was used in this study. This value was obtained by taking the average of the average of the ranges of CO_2 concentrations from the different studies conducted in the different cities around the world (Table 3.5) (Iovanca & Mihail , 2019).

Table 3. 5 : Typical carbon dioxide concentration in some cities around the world (Iovanca & Mihail , 2019).

Source	Place of measurement	Location	CO2 conc ppm
Coutts A. M. (2004)	Melbourne, Australia	City	355 - 380 ppm (daily mean concentration).
Day T. A. et al. (2000)	Phoenix, USA	City	377 - 396 ppm (daily mean concentration)
George K. et al. (2007)	Baltimore, USA	City	488 in urban area, 442 in sub-urban area
		Rural	422 in rural area
Ghauri B. (2003-2004)	Pakistan	City	270 - 325 ppm in Islamabad
		City	289 - 389 ppm in Quetta
		City	317 - 360 ppm in Karachi
		City	324 - 380 ppm in Lahore
		City	295 - 356 ppm in Rawalpindi
		City	312 - 382 ppm in Peshawar
Gratani L. et al. (1995-2004)	Rome, Italy	Rural	414 ± 25 ppm green zone
		City	505 ± 28 Centrale zone
Grimmond et al. (1995)	Chicago, USA	City	338 - 370 ppm (diurnal variation)
		City	405 - 441 ppm (nightly variation)
Idso S. B. et al. (2000)	Phoenix, USA	City	390 ± 0.2 ppm (minimum daily concentration)
		City	424 - 490 ppm (maximum daily concentration)
Kuc T. et al (2000)	Kasprowy Wierch	Rural	370 ppm (monthly mean variation)
	Krakow, Poland	City	370 - 430 ppm (monthly mean variation)
Moriwaki R. et al. (2005)	Tokyo, Japan	City	380 - 580 ppm (daily mean concentration
Nasrallah H.A. et al. (1996-2001)	Kuwait City, Kuwait	City	368 - 371 ppm (daily mean concentration at 7m high)
George K. et al. (2007)	Mexico City, USA	City	398 – 444 ppm (daily variation) 421 ppm (daily mean)

iii. Rural areas.

Atmospheric CO_2 concentration in rural area is relatively lower compared to urban areas. Studies by Idso, et al. (2002) found a 200-ppm difference between urban and rural (countryside) areas. An average CO_2 concentration of 275 ppm was used in this study. This value was obtained by subtracting 200 ppm from the CO_2 concentration value adopted the city/urban area.

iv. Seaside

An air-sea exchange of CO_2 over the world's coastal seas has been reported (Chen, et al., 2013). Approximately 25% of CO_2 emissions are estimated to be absorbed by the world's oceans (Ciais, et al., 2013; Takahashi, et al., 2009; Sabine, et al., 2004). The CO_2 concentration between oceans and the atmosphere has been evaluated by several authors (Table 3.6). No proper study was found in the literature relevant to the South African coastal areas. A value of 330 ppm, which represents the extreme value obtained from the different studies in literature was used in this study.

Table 3. 6 : Typical CO_2 concentration around coastal areas.

Source	**Location**	**CO_2 level ppm**
Schneider et al. (2003) and Kuss et al. (2004)	Gotland Sea (Baltic)	160
Omar et al. (2010)	Northern North Sea	225
Omar et al. (2010)	Southern North Sea	125
Borges and Frankignoulle (2003)	English Channel	320
Borges et al. (2006)	Celtic Sea	295
Copin-Montégut et al. (2004)	Bay of Angels (Mediterranean)	330
Wang et al. (2000)	East China Sea	280
Bates (2006)	Chukchi Sea	70
DeGrandpre et al. (2002)	US Middle Atlantic Bight	200

v. Parking, tunnels and industrial areas

The CO_2 concentration in parking lots is mainly related to the emissions from vehicles. In open spaces with sufficient O_2 present, CO_2 is usually dominantly a product of combustion. A study in Hong Kong revealed that the levels of CO and CO_2 of indoor parking lots are relatively higher compared to outdoor parking area (Wong, et al., 2002). A measure of CO_2

concentration in parking lots is generally a measure of the total amount of accumulated exhaust, and consequently, refers to ventilation requirements. 1330 ppm CO_2 concentration is the limiting value suggested after a study of diesel exhaust (US Department of interior, 1994, ASHRAE, 1999).

Values in the same order were observed in studies around city tunnels. A study conducted by Maria et al. (2009) shows a steady rise in pollutant concentration along the tunnel in the traffic direction, with CO_2 levels reaching values above 1000 ppm.

An industrial zone refers to a designated area where industrial factories are concentrated (Hsu & Chiang, 2001). Very few studies exist in literature on the CO_2 concentrations around industrial zones. In this study, values for CO_2 concentration around industrial zones was assumed similar to that around parking structures and tunnels.

3.3.2.3 Summary of CO_2 concentration

Following the above discussion, a summary of the ambient CO_2 concentration for different environmental land use is presented in Table 3.7.

Table 3. 7 : Summary of carbon dioxide concentration according to key locations

Key location	**CO_2 Conc. ppm**
Indoor	870
Urban area (city)	475
Rural area	275
Industrial zone	1330
Seaside	330
Tunnels and parking areas	1330

In the carbonation model developed by Salvoldi (2010), the CO_2 concentration is estimated in mol/m^3. Atmospheric CO_2 is measured in parts per million volume (ppmv). The relationship between the CO_2 concentration in mol/m^3 and ppmv is described using Equation 3.19.

$$c = \frac{CO_2\% \,.\, n}{V} = \frac{ppmv_{CO_2} \,.\, n}{1000 \,.\, V} \qquad \text{Equation (3.19)}$$

Where c is the CO_2 concentration (mol/m^3), $ppmv_{CO_2}$ is the parts per million volume concentration of CO_2, $\frac{n}{V}$ is the molar volume of CO_2. At sea level at 25^{o}C the molar volume of carbon dioxide is 24.79 litre/mol.

3.3.3 Geometric parameter

The concrete cover thickness denoted by d, is an important durability indicator for carbonation-induced corrosion. In theory, the carbonation front is approximated with the square root of time equation (RILEM report 40, 2007). This means that the time to corrosion initiation is proportional to the square of the cover depth. Unfortunately, a remarkably large scatter is often observed in the concrete cover depth of a single structure. Many cover depth survey studies done on structural elements (bridges and buildings) have shown that most structures have inadequate cover depths and show significant variability from the design value (Nganga, 2011; Marosszeky and Chew, 1990; Clark et al., 1997; Sharp, 1997; Ronne, 2005).

Nganga (2011), collected data on concrete cover measurements for various structures in the Gauteng freeway Improvement project (GFIP). Then a set of statistical information to describe the concrete cover depth (distribution function, mean and standard deviation values) was proposed by considering the goodness-of-fit test. However, the data was found to be inappropriate for this study as the concrete cover depth value should be limited within a practical range (positive values). For example, with the normal distribution proposed at the end of the study by Nganga (2011), although the probability of occurrence is low, a negative value for concrete cover depth might be obtained from numerical sampling. If this occurs, computation cannot be complete.

Other statistical characterisations were considered in this research. The statistical characterisation for concrete cover depth proposed by the international concrete federation (fib) for the probabilistic carbonation model was adopted (fib, 2006). fib (2006) assumes the concrete cover depth values as a random variable that follows a log-normal distribution with mean equals the design or measured concrete cover d [mm] and standard deviation equals 6 mm. The log-normal distribution is suitable as it eliminates all possibility for negative values for concrete cover.

3.3.4 Concrete micro-structure parameter

The concrete micro-structure parameter describes the resistance of concrete to carbonation. This parameter is characterized by the Oxygen Permeability index (OPI), which is strongly influenced by the concrete composition (w/b ratio and binder composition). The OPI test method measures the oxygen permeability of concrete. Permeability coefficient for concrete can be used to describe the pore structure and therewith estimate the diffusion coefficient for concrete. Equation 3.20 was derived via regression analysis relating the CO_2 diffusion coefficient and the oxygen permeability coefficient (k) obtained from OPI test (Salvoldi, 2010).

$$D_{CO2} = \left[1.4\times\left(\frac{K}{10^{-11}}\right)^{2.2}\right]\times10^{-11} \qquad \text{Equation 3.20}$$

Where D_{CO2} is the carbondioxide diffusion coefficient and k is the oxygen permeability coefficient obtained from the Oxygen Permeability Index Test.

A study by Nganga (2011), were OPI testing was done on core samples from a variety of structures in the Gauteng Freeway Improvement Project (GFIP) revealed a large scatter in the measured OPI values in single structures. Studies by Imamoto et al. (2016) in Japan, Portugal and Switzerland equally showed variability in the measured air permeability values for a single structure. Nganga (2011) proposed a set of statistical information (distribution function, mean and standard deviation values) to describe the OPI measurements in real life structures (see Table 3.5).

Based on the results from the five different projects examined by Nganga (2011), an appropriate description for OPI variability was obtained by averaging the results of standard deviations from the five projects. For this study, the OPI parameter was approximated by a normal distribution with mean values equalling the design or measured OPI value, and the standard deviation equalling 0.25.

Table 3. 8 : Summary statistics of OPI (Nganga, 2011).

Project	OPI (log scale)					
	Distribution	Mean	Max	Min	SD	CoV (%)
1	Normal	9.75	10.41	9.07	0.28	2.84
2	Normal	9.91	10.42	9.37	0.22	2.24
3	Normal	9.87	10.40	9.39	0.23	2.33
4	Normal	10.06	11.10	8.83	0.46	4.60
5	Normal	10.25	10.70	9.85	0.18	1.75

3.4 Reliability modelling

As seen above, the carbonation process in RC structures is complex and subject to several uncertainties. In this context, a reliability approach as compared to a deterministic approach seems appropriate to model such a problem. A reliability approach helps to assess the probabilities of failure of an event represented by a physical model. Probabilistic-based service life assessments of RC structures have shown to be valuable and have been widely adopted in design codes replacing deterministic approaches (Siemes & Edvardsen, 1999; DuraCrete, 2000; fib, 2006).

3.4.1 Formulating a limit-state function for corrosion initiation

As seen previously, a limit-state is the event in the life of the structure that marks the boundary between the desired and the failed state. Failure here refers to the violation of any specified performance requirement during the service life of the RC structure. For example, the event when the carbonation front reaches the reinforcing steel.

Based on the variables of interest in the carbonation process, the corresponding performance requirement can be formulated as the difference between a "resistance term" and a "load term".

Resistance term: Concrete cover depth, d.

Load term: Carbonation progression (front), X_c.

Therefore, the limit-state function for corrosion initiation can be formulated as follows:

$$g[d,X_c] = d - X_c(t) \qquad \text{Equation (3.21)}$$

Hence:

$g[d,X_c] = 0$ Limit state

$g[d,X_c] > 0$ Desired state or passivated state

$g[d,X_c] \leq 0$ Failure state or depassivated state

Using the carbonation model equation proposed by Salvoldi (2010) (Equation 2.8), the limit-state function for corrosion initiation can be formulated as a function of six variables (cover depth, effective diffusion coefficient, ambient carbon dioxide concentration, amount of carbonatable material, environmental factor and time) according to Equation 3.22.

$$g[d, D, c, a, \beta, t] = d - \sqrt{\frac{2Dc\beta t}{a}} \qquad \text{Equation (3.22)}$$

Where, g [d, D, c, a, β, t] is the Limit-state function, t the service life (years), β the relative humidity factor, D the effective diffusion coefficient (mm/days), c the ambient CO_2 concentration (mol/m^3), and a the amount of carbonatable material in the concrete matrix (mol/m^3).

3.4.1 Estimating the probability of failure using Monte Carlo simulation

The Monte Carlo Simulation (MCS) method was used to compute the probability of exceeding the limit-state function. The statistical information and distribution functions pertaining to the carbonation parameters have been described in Section 3.3. Random values were then sampled from each of the defined distributions. For each trial, MCS analysis was applied to solve for the time to steel depassivation. This is termed the bootstrapping technique (Efron, 1979; Dudewicz, 1992). The process was repeated several times to define a distribution to corrosion initiation time and to reduce the inherent error involved in the MCS process. Due to limited computing capacities, the number of trials where limited to 10,000 trials. The accuracy of the results increases as the number of trials increases (Ang & Tang, 2007).

The probability of failure (P_f) is given by the ratio between the number of trials resulting in the negative performance of the limit-state function and the total number of trials (N) (Equation 3.23).

$$P_f = \frac{n}{N} \qquad \text{Equation (3.23)}$$

The accuracy of the estimated P_f (standard deviation) is given by Equation 3.24.

$$\sigma(P_f) = \sqrt{\frac{P_f\,(1 - P_f)}{N}} \qquad \text{Equation (3.24)}$$

3.5 Summary

The carbonation models relevant to the South African durability index in literature are deterministic in nature and present the user with the requirement to have a lot of data at their disposal. The proposed probabilistic model is based on the same background as the model proposed by Salvoldi (2010) and is implemented in a software based on MS Excel in order to make it simple, user friendly and functional.

Improvements to Salvoldi's model include the ability to consider standard binder composition and cement replacements proportions adopted in industry, different locations in South Africa in terms of varying relative humidity values and different environmental land use with varying CO_2 concentration.

From a probabilistic point of view, the use of distribution functions to account for the variability in the model input parameters is also an advantage.

CHAPTER 4

4. Model validation and sensitivity analysis

4.1 Introduction

This chapter presents a brief explanation of the developed software program. The program was implemented in an MS Excel spreadsheet and provides an estimate of the service life of carbonating RC structures, i.e. the time when the carbonation front reaches the depth of the reinforcing steel. The software represents the implementation of the probabilistic model proposed in the previous chapter. A detailed user guide for the developed MS Excel program can be found in Appendix B.

The probabilistic carbonation model was developed from existing carbonation models by refining the input parameters, such that standard binder compositions and cement replacement proportions adopted in industry, different locations in South Africa with varying RH and different environmental land uses with varying CO_2 concentration can be considered. Model validation with long term data was not possible. However, the output from the model was compared with the existing OPI - carbonation model as well as a sensitivity analysis of the input parameters to check the validity and sensitivity of the developed model.

4.2 The service life design MS Excel program

The previous chapter presented the probabilistic model for carbonation as well as the mathematical solution adopted for the Monte Carlo Simulation. The developed service life design program is intended to be used by engineers in practice at the design phase for estimating long-term performance of new RC structures and as an improved basis for conformity assessment (quality control) of newly built RC structures. The program requires few but critical input parameters that can easily be measured using well known test methods or obtained from commercially available concrete producers. Other parameters, such as expected carbon dioxide concentrations and environmental characteristics are already integrated in the program and can easily be identified for the prevailing land use and location of the structure.

The service life design program is composed of four sheets:

1. The About sheet, which gives an overview of the program.
2. The SL-Model sheet, which contains the design interface with both input and output information displayed.
3. The Randomotor sheet, which performs the MCS analysis.
4. The Portlandite sheet, which computes the amount of portlandite (carbonatable material) for the given input parameters.

4.3 Model validation

To validate that the program computed realistic carbonation depths, carbonation was modelled and compared to the existing carbonation model currently used in industry. The existing OPI - carbonation model used in industry relies on regression correlation values of carbonation depth and OPI developed by Mackechnie & Alexander (2002). The model assumes a power relationship between carbonation depth and time, with a 0.33 power factor for both PC and GGBS concretes and a 0.4 power factor for both FA and SF concretes. The existing model used requires the following as input parameters: The OPI value, the binder type with five standard cement replacements provided (100% PC, 10% SF, 30% FA, 50% GGBS and 50% GGCS) and the exposure environments where three exposure environment with different RH values are provided (dry Inland with 60% av. RH, coastal with 80% av. RH and partly wet with 90% av. RH). Figure 4.1- 4.4 show the comparison of the model developed in this dissertation to the existing OPI – carbonation model. For the comparison the ambient CO_2 concentration was taken as 475 ppmv, corresponding to the CO_2 concentration in urban areas. A total binder content of 350 kg/m^3 was assumed for the different concretes. These values were chosen as the study by Mackechnie & Alexander (2002) was conducted on plain and blended cement concretes exposed to the outdoor environment around the city of Cape Town, with the mix design for the concrete samples chosen to simulate standard concretes used in industry. Three OPI values (9.5, 9.7, 10) were selected for comparison to cover a range of OPI values typical in structural applications. The outcomes of the various scenarios are summarized in Figures 4.1- 4.4.

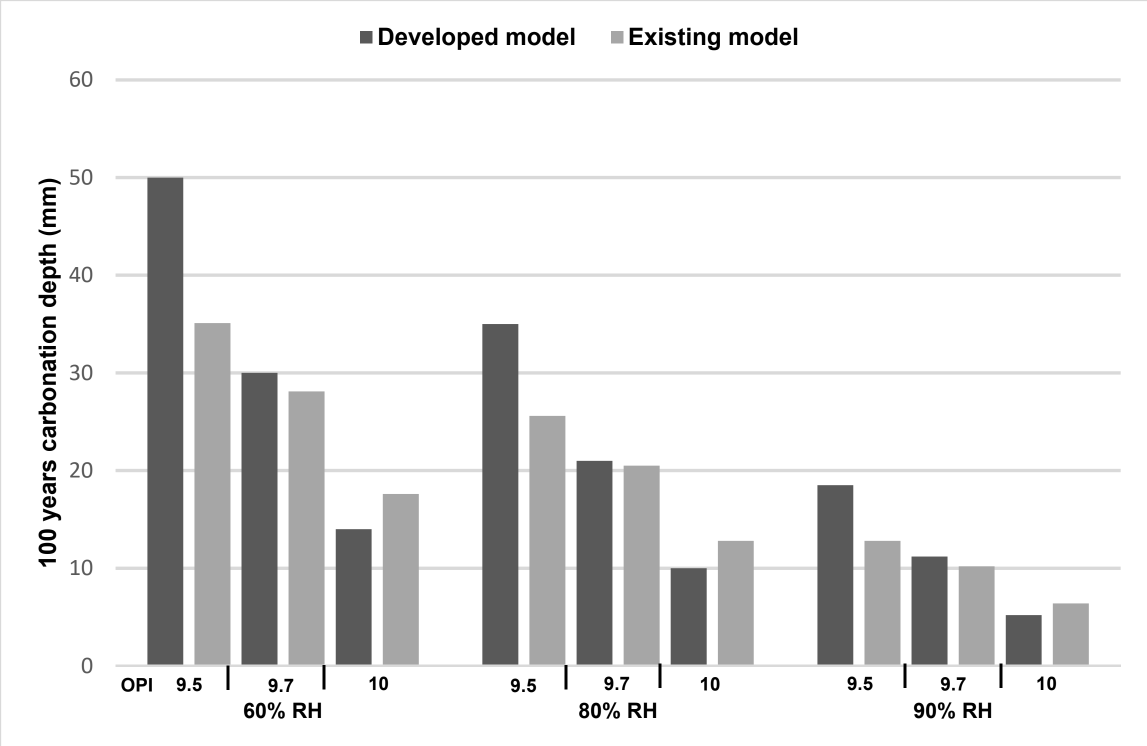

Figure 4. 1: Comparison of carbonation depth results obtained with current model to the results obtained with the existing model (PC concretes)

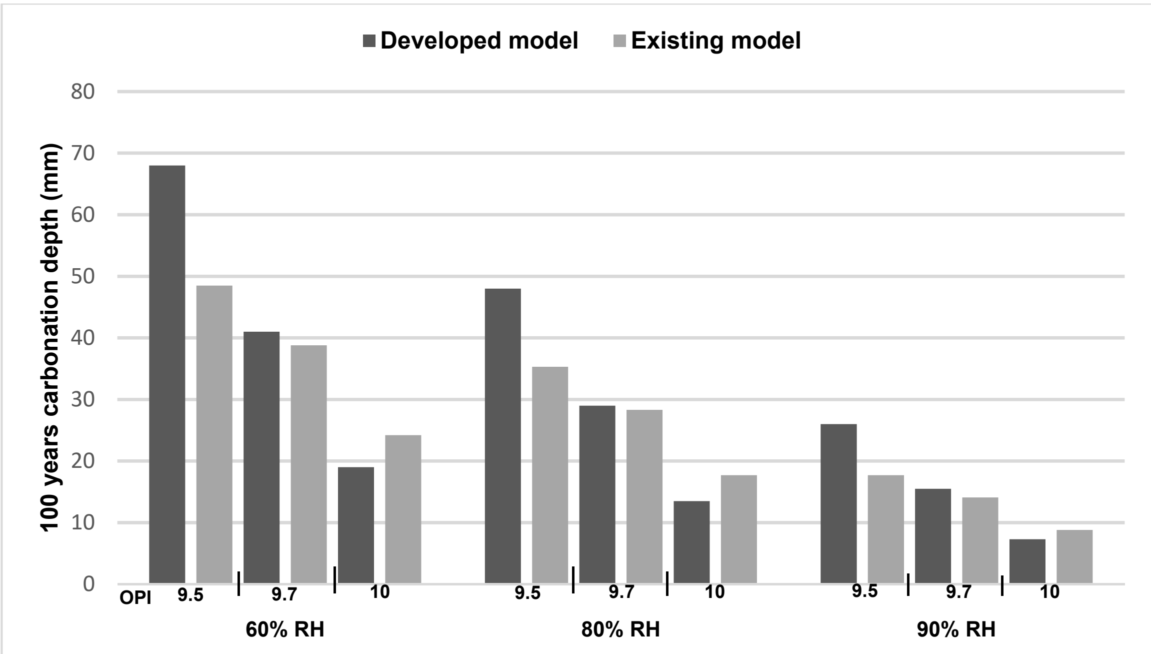

Figure 4. 2: Comparison of carbonation depth results obtained with current model to the results obtained with the existing model (SF concretes)

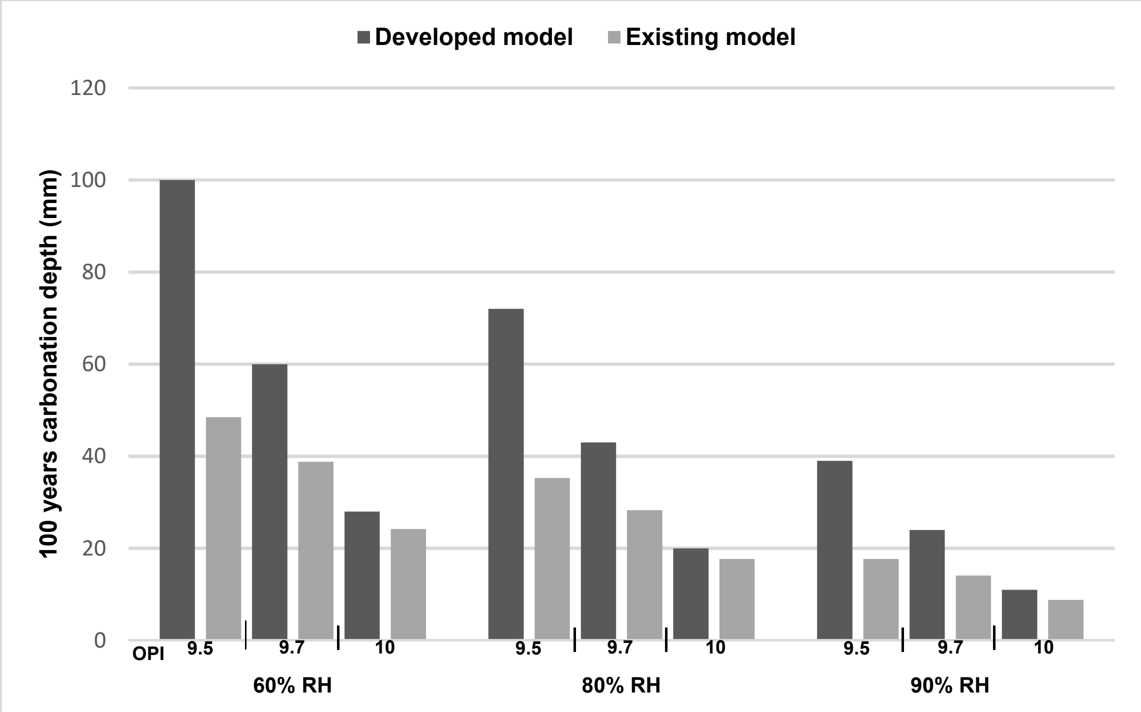

Figure 4. 3: Comparison of carbonation depth results obtained with current model to the results obtained with existing model (FA concretes)

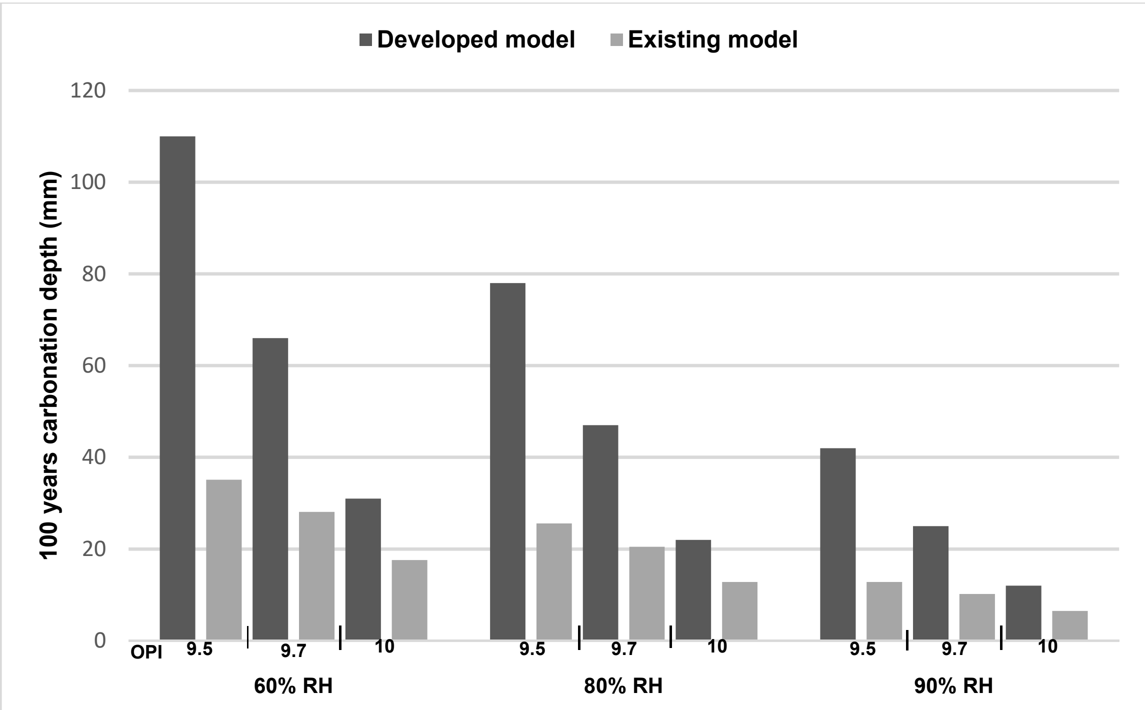

Figure 4. 4: Comparison of carbonation depth results obtained with current model to the results obtained with existing model (GGBS concretes)

From the comparisons of carbonation depth after 100 years of exposure from both models, the following points can be made:

i. Based on both models, the calculated carbonation depths generally increases with a decreasing OPI, which was observed for all concrete types.
ii. For PC concretes and SF concretes, both models compute similar carbonation depths, with the developed model being considerably more conservative for lower OPI values (9.5) and less conservative for high OPI values of 10.
iii. For FA concretes, both models predicted similar carbonation depths for concretes with an OPI value of 10. However, there are significant differences in the predicted carbonation depth for OPI values below 10. In general, the developed model is more conservative.
iv. For GGBS concretes, a large variation exists in the 100 years carbonation depth predicted by both models, with the developed model generally yielding more conservative results.

The reasons for the closeness in the predicted carbonation depth values for PC and SF concrete by both models and the consistently higher carbonation depth prediction made by the developed model for FA and GGBS concretes is discussed below. The analysis of the similarities and differences was approached from two perspectives. First, the existing OPI – carbonation model was developed from natural carbonation studies while the correlation between OPI and carbonation diffusion coefficient in the developed model is based on accelerated carbonation experiments. Secondly, while laboratory carbonation studies by Salvoldi (2010) were carried out in 2010, with casting of the concrete samples at the same period, the carbonation studies by Mackechnie & Alexander (2002) were performed more than a decade earlier with natural carbonation measurements done at intervals of 1, 4 and 6 years.

4.3.1 Accelerated and natural carbonation model

Natural carbonation is a more realistic means of examining the performance of concrete under carbonation. Since natural carbonation processes are slow and take a very long time to produce a significant effect, accelerated carbonation studies are commonly used in the laboratory to conduct carbonation experiments. The accelerated carbonation method has in the outdated British Standard been standardized by using CO_2 concentration of 4% at 20°C and 55% RH (BS 1881-210, 2013). fib, (2006) recommends a 2% CO_2 concentration in accelerated carbonation test. In the model by Salvoldi (2010), accelerated carbonation was performed at 20°C ± 2°C, 65% RH ± 5% and CO_2 concentration of 2% ± 0.1% as specified by fib bulletin 34. The typical atmospheric carbon dioxide concentration for natural carbonation around the cities is in the order of 400 ppmv or 0.04%.

It can be observed from Figure 4.1 - 4.4, that the developed model provides higher carbonation results relative to the existing OPI - carbonation model. This difference in the carbonation depth prediction between both models could be due to the fact that while the model by Salvoldi (2010) was developed from accelerated carbonation studies, the model by Mackechnie & Alexander (2002) was developed from natural carbonation studies. In comparing the two models, it is evident that the model developed from accelerated carbonation measurements is generally more conservative than the models developed from natural carbonation studies. This is because accelerated carbonation studies use higher CO_2 concentration of about 2% (Salvoldi, 2010) compared to a typical atmospheric CO_2 concentration for natural carbonation around cities in the order of 400ppmv or 0.04%.

Though it is well established that SCMs concretes give higher carbonation relative to plain cement concretes, a study by Leemann et al. (2015) found that accelerated carbonation studies unduly penalise cementitious systems containing SCMs, with accelerated test giving poorer performance for SCM concretes than their real performance under natural carbonation.

Accelerated carbonation tests used in the derivation of the developed model makes use of short-term concrete carbonation data (12 weeks), which could explain the conservative predictions observed. Carbonation results in pore densification due to the formation and deposition of $CaCO_3$, which would be more pronounced in long-term studies as opposed to accelerated tests and therefore, increasingly less ingress of CO_2 in long-term field concrete. Hence the higher carbonation in the values predicted with a model based on short-term testing. Similarly, at early-ages, concrete cover is generally more porous because of the reduced curing time.

The difference observed could also be explained by the fact that in the field data, carbonation does not actually take place throughout the year. In Cape Town for example, the CO_2 ingress mostly in summer, when the concrete is dry. In winter, less CO_2 ingress is observed and hence less carbonation because the pores are often saturated. Therefore, there is less “effective exposure” time in a year, in the field, compared to the accelerated carbonation studies which assumes a constant humidity and no pore-blocking effect from rain.

The difference in carbonation depth measurements between accelerated and natural carbonation studies verifies with results of Sanjuan, et al. (2003) where both the accelerated and natural carbonation tests conducted on different concretes types, with water/binder (w/b) ratio varying from 0.33 to 0.69 reported that 5% CO_2 and 100% CO_2 concentrations in the accelerated test led to 5 and 40 times higher carbonation respectively, relative to the natural test conducted at 0.03% CO_2. They found a direct correlation between accelerated and natural test for carbonation; and suggested that 7 to 15 days of accelerated carbonation are equivalent to one year of natural carbonation. Dhir, et al. (1989) also advanced that one week of accelerated carbonation test relates to about 15 months of natural exposure, basing their analysis on results of Ho & Lewis (1983) and their own work.

4.3.2 Change in binder characteristics

The current OPI-carbonation model was calibrated with binder materials produced more than 2 decades ago. The model assumes a power relationship between carbonation depth and time and does not consider the chemical composition of the material. The binder characteristics influence the microstructure of the concrete. The properties of cement and cement extenders have changed over the years mainly due to economic reasons, with present day cements being significantly finer with higher C_3S contents (Yunusa, 2014; Salvoldi, 2010). This implies that its rate of hydration is high, with less later-age hydration or further pore refinement. Additionally, the more reactive binder materials will increase the heat of hydration and thus the drying shrinkage of the concrete with a possibility of micro cracks and voids. This may create increased access for CO_2 to penetrate the concrete.

Another reason could be the fact that the actual binder contents in the concrete from the field studies was in fact higher or lower than the 350 kg/m^3 assumed. If the field concrete tested by Mackechnie & Alexander (2002) had binder contents less than 350 kg/m^3, then of course it will result in a decrease in the amount of hydration products. Hence, for the same permeability, a concrete with lesser binder content will show faster carbonation progression.

4.3.3 Summary

From the analysis of the differences in the predicted carbonation depths by both models, the following reasons have been discussed. Firstly, the accelerated carbonation test results used in the derivation of the developed model makes use of short-term concrete carbonation data which fails to account for the effect of pore densification due to the formation and deposition of $CaCO_3$. Secondly in the field data used by Mackechnie & Alexander (2002), carbonation does not actually take place throughout the year, resulting in less effective exposure time compared to accelerated carbonation data which assumes a constant humidity and no pore-blocking effect from rain. Another reason could be the fact that the current existing model was calibrated with binder material produced more than 2-decades ago, while cement and cement extenders with different properties are being used nowadays. Lastly, the actual binder content from field studies could be different from the assumed 350 kg/m^3. Nevertheless, the difference in the outcome observed from both models is a combination of all these factors and others. However, both models show the same trends, which is taken as a positive validation of the developed model. It is not within the scope of this research to investigate further into the above observed differences. This can be further explored in future research, based on verification with long-term field results. For this research, it is accepted that the developed model is more conservative than the existing empirical model.

4.4 Sensitivity analysis of the model parameters

The sensitivity analysis of the input parameters was performed by systematic analysis of the effect of change in a variable on the outcome of the simulation. The input parameters were varied and the associated change in the service life of the structure plotted. The sensitivity analysis was applied to the model parameters described in Table 4.1, with the parameters grouped as either deterministic or random parameters. Two distinct reference case definitions are defined. The analysis of the model parameters in these reference cases allows for conclusions to be drawn regarding the effects of varying the parameters on the structure's performance.

Table 4. 1: Input parameters for the sensitivity analysis

Deterministic parameters	Random parameters
Binder content and type	OPI
CO_2 concentration	Cover depth
Location	-

4.4.1 Reference Case 1

In the first reference case, the deterministic parameters were varied and their effect on the service life of the structure observed. In order to verify the model's sensitivity to both concrete of good-quality and poor-quality concrete and to provide realistic service life prediction the analysis was done for both high and low cover depths and OPI values. A baseline of 30 mm and 25 mm for cover depth, 9.7 and 9.4 for OPI was chosen to simulate concrete of good quality and poor-quality concrete respectively.

4.4.1.1 Good quality concrete

For the sensitivity analysis of the input parameters with concrete of good quality, a baseline was chosen as a concrete with 30 mm cover depth and OPI of 9.7. Table 4.2 shows the parameters used for the simulation.

Table 4. 2: Values of variables for good quality concrete

Figure	Binder content (kg/m^3)	Supp. Cementitious material	Cover depth (mm)	OPI	Location / RH (%)	Land Use / CO_2 conc, (ppmv)
Figure 4.5	200 - 450	No	30	9.7	65	475
Figure 4.6	350	No	30	9.7	65	400 - 1000
Figure 4.8	350	No	30	9.7	0 - 100	475

Figure 4.5 shows the change in the service life with increase in binder content. The OPI, CO_2, RH and cover depth are assumed to be constant.

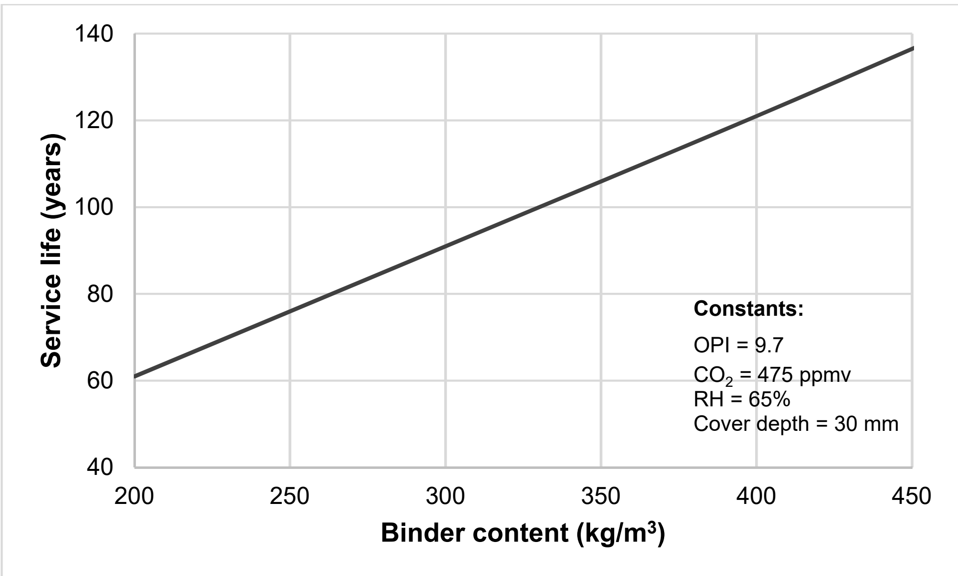

Figure 4. 5: Change in service life with variation of binder content

In Figure 4.5, it can be seen that the service life increases with an increase in binder content. This is because, in the model the service life is linearly related to the amount of carbonatable material. An increase in the binder content results in an increase in the amount of carbonatable material (other factors constant) and therefore and increase in service life. The prediction of a service life longer than 100 years is a theoretical exercise. This is a result of the high OPI values and cover depths used in this analysis. More realistic values would have been obtained with poorer quality concrete and lower cover depths.

On a more practical note, an increase in service life with binder content implies that higher binder contents result in higher durability. However, Figure 4.5 is based on a constant OPI value. In practice, an increase in binder content could result in a lower (or higher) OPI depending on the mix composition, leading to different service life values (Angelicci, 2013). Hence the overall effect of increasing the binder content is not shown in this figure.

Figure 4.6 shows the effect that the ambient CO_2 concentration has on the service life. Figure 4.7 shows the difference between concrete exposed to ambient CO_2 concentration in urban areas (275 ppmv), concrete exposed to a city environment (475 ppmv), indoor concrete (870 ppmv), concrete around sea side areas (330 ppmv), and concrete around parking areas and tunnels (1330 ppmv).

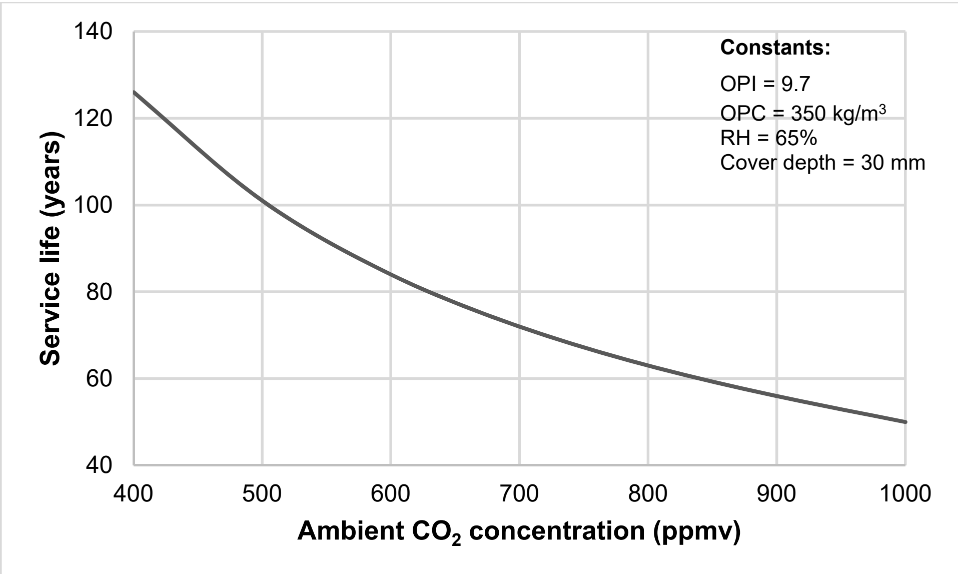

Figure 4. 6: Change in service life with variation in CO_2 concentration

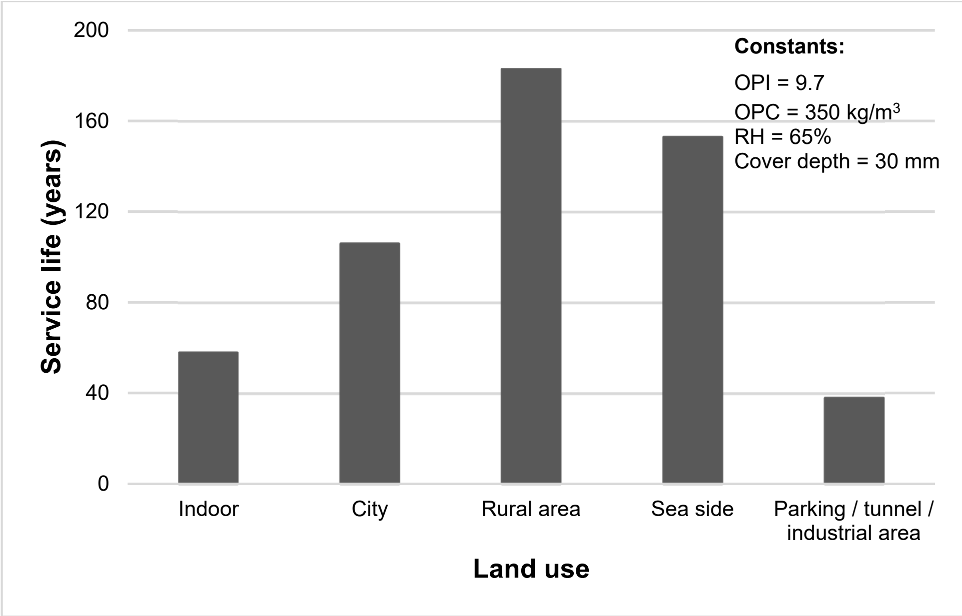

Figure 4. 7: Change in service life with variation in the land use around the structure

The developed model accounts for the effect of RH on the service life (defined as the time when carbonation depths reach the reinforcing steel) in two ways. Firstly, the effect on the diffusion of CO_2 with the relative humidity factor β, and secondly, the effect on the degree of hydration. Figure 4.8 shows the effect of relative humidity on the service life with higher service life at low and high relative humidities and the lowest service life values at medium range RH. This can be explained as diffusion of CO_2 within concrete is facilitated through the air-filled pores but is very slow through those filled with water. The diffusion of carbon dioxide in water-filled pores is about four orders of magnitude slower than that in air-filled pores (Bertolini, et al., 2013). The carbonation reaction on the other hand occurs in the presence of water, so that it is negligible in dry concrete. On a more practical note, the interval of RH most critical for the service life of the structure is 50% to 60% (Figure 4.9). This corresponds to the annual average RH of the atmosphere in the provinces of Bloemfontein, Johannesburg and Pretoria. Figure 4.10 shows the difference in service life for concretes in different provinces in South Africa. As expected, the provinces of Johannesburg, Bloemfontein and Pretoria with medium RH values (50 - 60%) have a lower service life.

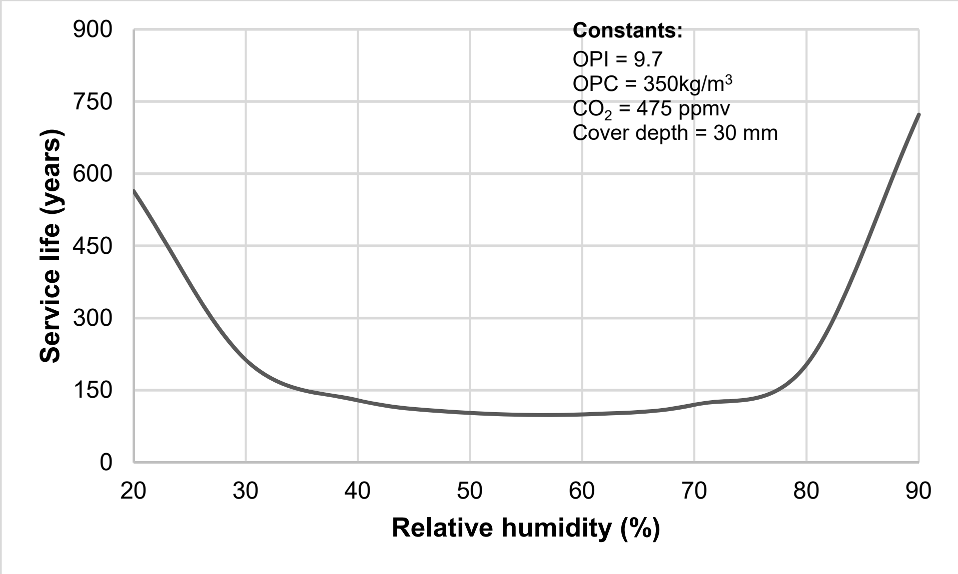

Figure 4. 8: Change in service life with variation of RH

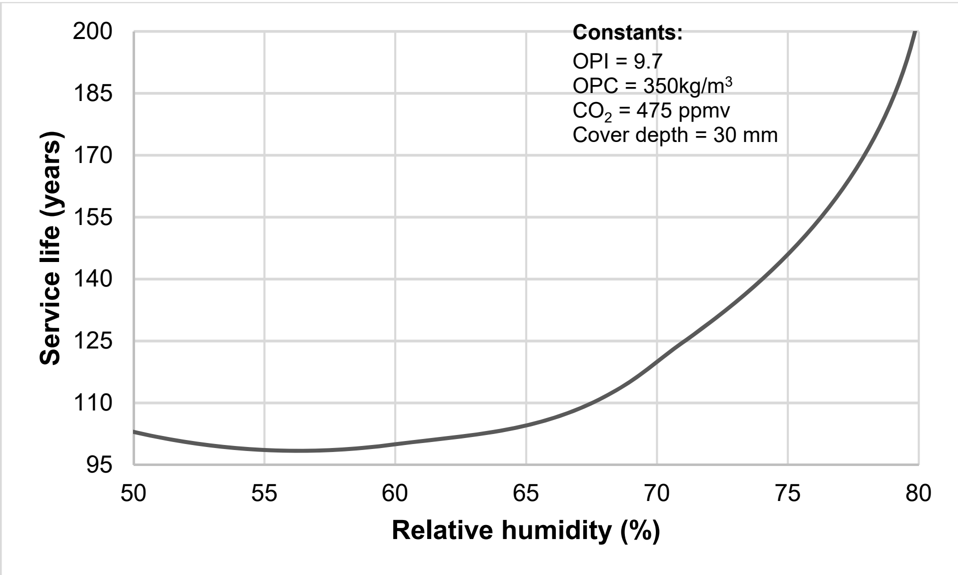

Figure 4. 9: Change in service life with variation of RH for common range of RH (50-80%)

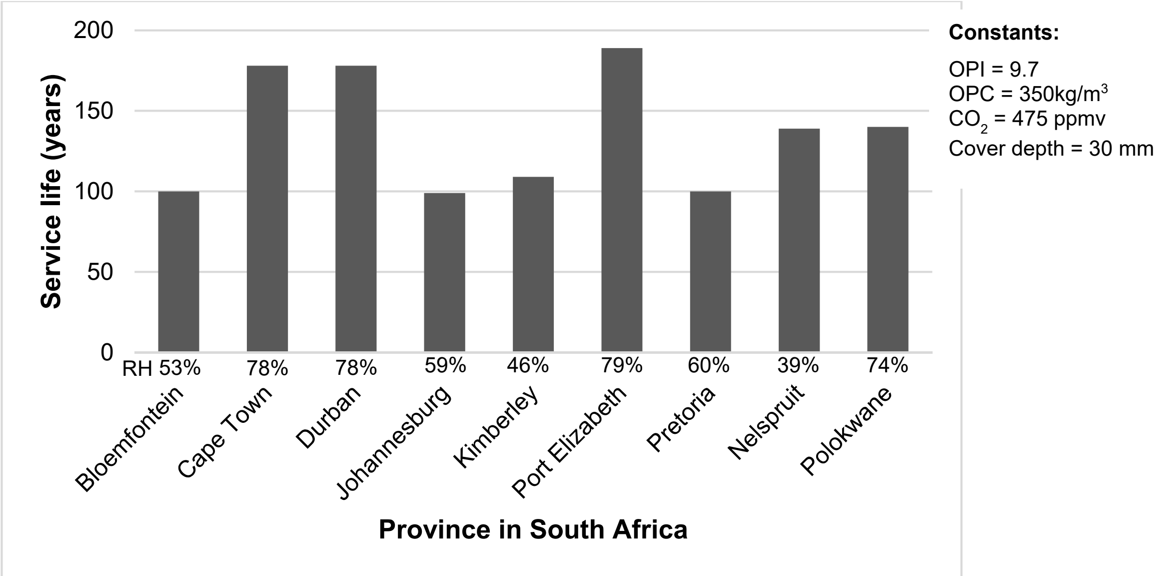

Figure 4. 10: Change in service life with variation in the location of the structure around different provinces in South Africa

4.4.1.2 Poor quality concrete

From the sensitivity analysis with concrete of good quality, it can be observed that the model predicts service life values longer than 100 years. The high service life value is the result of a theoretical exercise with high cover depths and OPI values used for the analysis. To obtain more realistic values, the same exercise was done with more conservative values (poorer quality concrete and cover depths). For the sensitivity analysis of the input parameters with poor quality concrete, a baseline was chosen as a concrete with 25 mm cover depth and OPI of 9.4. Table 4.3 shows the parameters used for the simulation.

Table 4. 3: Values of variables for poor quality concrete

Figure	Binder content (kg/m^3)	Supp. Cementitious material	Cover depth (mm)	OPI	Location / RH (%)	Land Use / CO_2 conc, (ppmv)
Figure 4.11	200 - 450	No	25	9.4	65	475
Figure 4.12	350	No	25	9.4	65	400 - 1000
Figure 4.14	350	No	25	9.4	0 - 100	475

Figure 4.11 shows the change in the service life with increase in binder content. The OPI, CO_2, RH and cover depth are assumed to be constant.

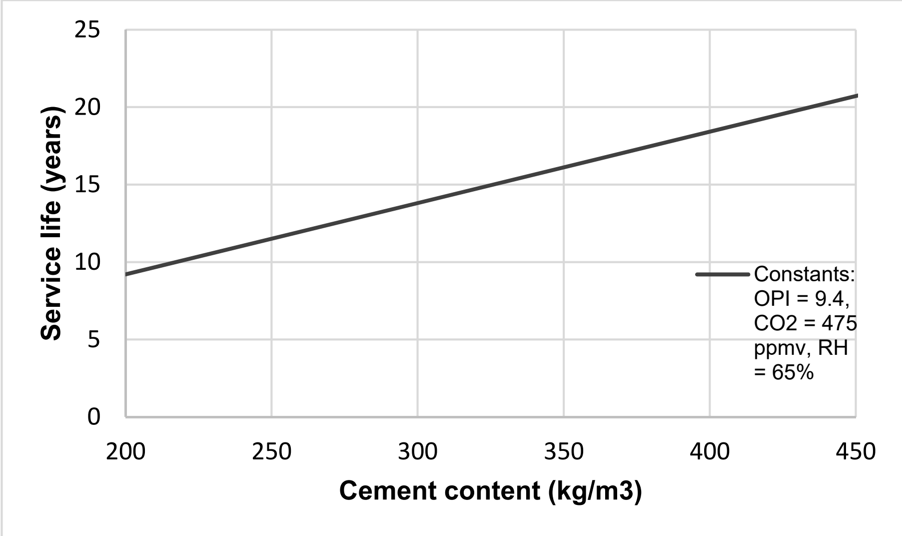

Figure 4. 11: Change in service life with variation of binder content

In Figure 4.11, it can be seen that the service life increases with an increase in binder content. The graph shows a similar trend to that observed with concrete of good quality. Therefore, all the previous discussions are still valid for poor quality concrete. However, the prediction of service life is shorter than 100 years, with the results being more realistic. This is the result of lower OPI values and cover depth for poor quality concrete.

Figure 4.12 shows the effect that the ambient CO_2 concentration has on the service life. Figure 4.13 shows the difference between concrete exposed to ambient CO_2 concentration in urban areas (275 ppmv), concrete exposed to a city environment (475 ppmv), indoor concrete (870 ppmv), concrete around sea side areas (330 ppmv), and concrete around parking areas and tunnels (1330 ppmv).

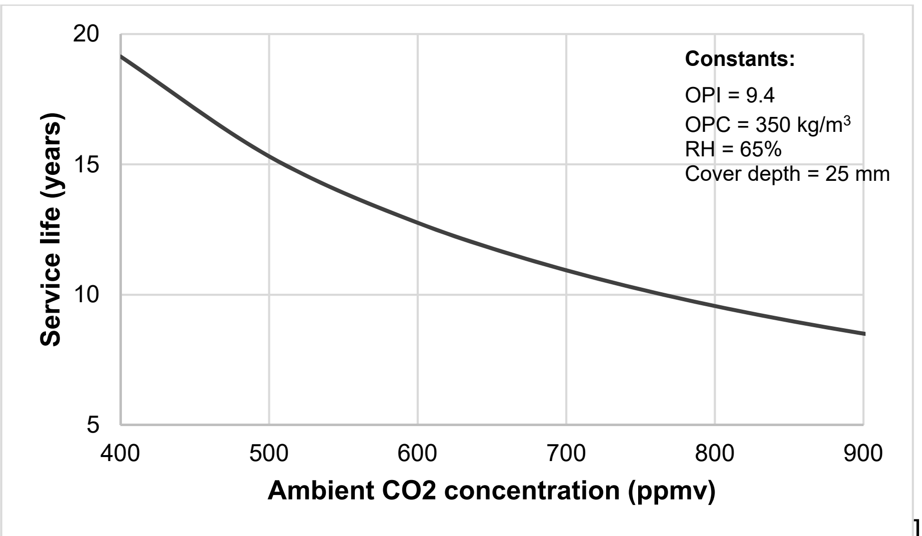

Figure 4. 12: Change in service life with variation in CO_2 concentration

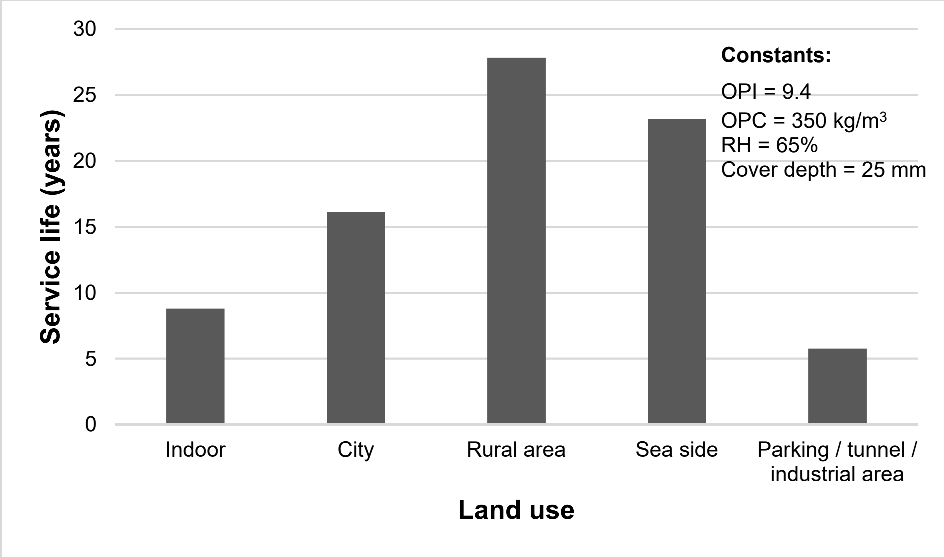

Figure 4. 13: Change in service life with variation in the land use around the structure

Figure 4.14 shows the effect of relative humidity on the service life for RH between 50% to 80%. The graph shows a similar trend to that observed with concrete of good quality. Therefore, all the previous discussions are still valid for poor quality concrete. However, the prediction of service life is shorter than 100 years, with the results being more realistic. Figure 4.15 shows the difference in service life for concretes in different provinces in South Africa.

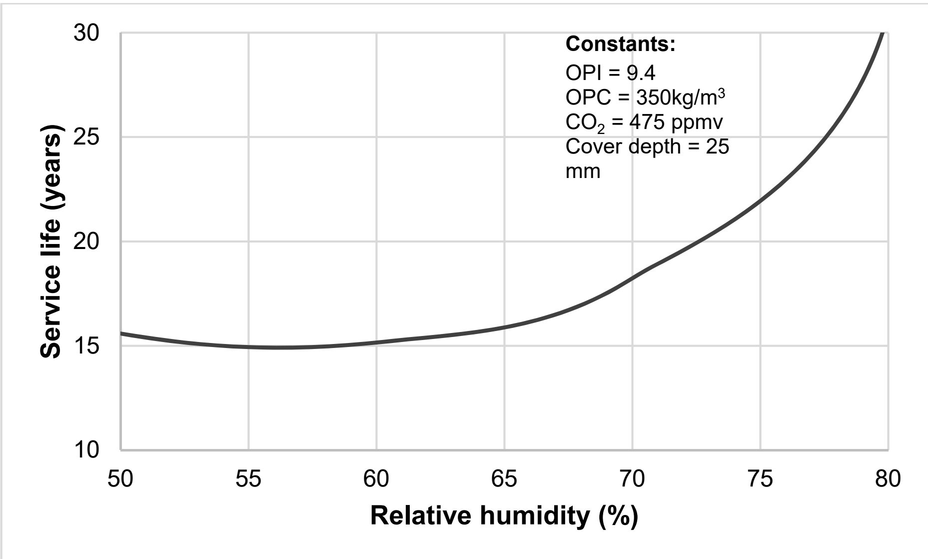

Figure 4. 14: Change in service life with variation of RH for common range of RH (50-80%)

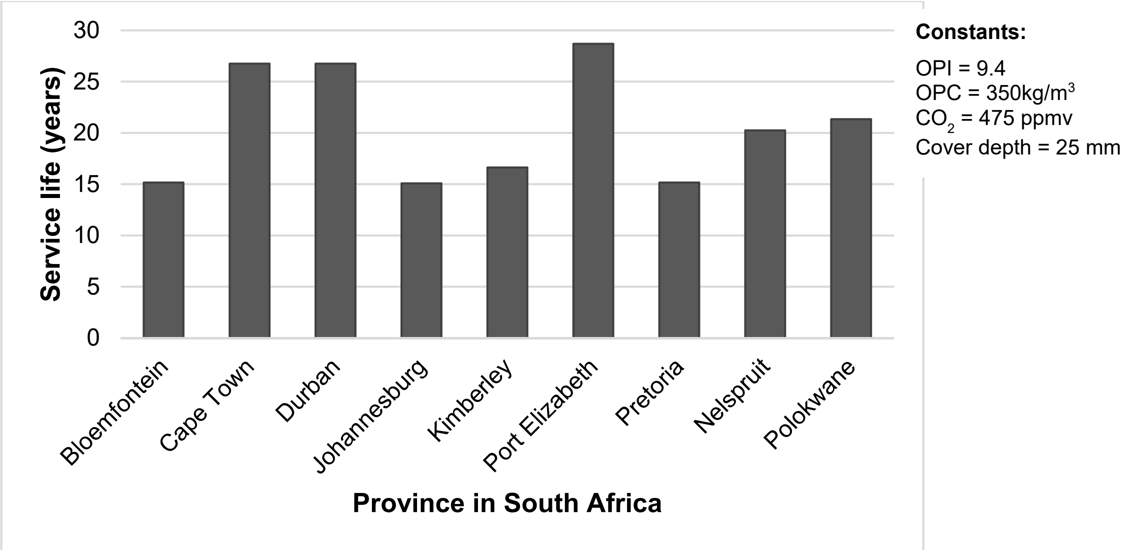

Figure 4. 15: Change in service life with variation in the location of the structure around different provinces in South Africa

4.4.2 Reference Case 2

The second reference case studies the sensitivity of the random parameters in the model (OPI and concrete cover depth). Here, the mean value of the parameter is varied over a valid range while the standard deviation is constant. Table 4.4 shows the parameters used for the simulation. In bold are the initial values of the parameters for each simulation. For the analysis, the ambient CO_2 concentration was fixed as 475 ppmv, corresponding to the CO_2 concentration in urban areas. A total binder content of 350 kg/m^3 was assumed for the different simulations.

Table 4. 4: Values of variables for Reference Case 2

Variable	Distribution	Mean	Standard deviation
Cover depth (mm)	Log - normal	20,**30**,40,50	6
OPI ($\times 10^{-10} m/s$)	Normal	9.1, 9.4, **9.7**, 10	0.25

i. Concrete cover depth

The average concrete cover parameter was varied from 20 mm to 50 mm with 6 mm standard deviation. From Figure 4.16, the effect of concrete cover on the failure probability for a given service life is clearly noticeable, with an increase in concrete cover having significant positive effects on the performance of the structure. For example, at 50 years the probability of failure for a concrete cover of 50 mm is 12% compared to a 52% chance of failure for a 20 mm concrete cover.

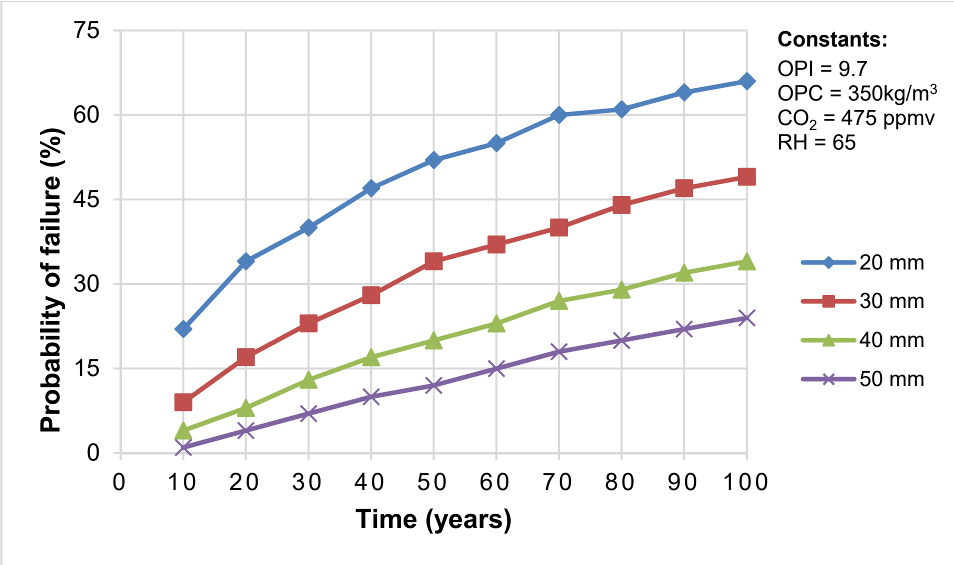

Figure 4. 16: Change in failure probability with time for different concrete cover thicknesses

ii. Oxygen permeability index

The average OPI was varied from 9.1 to 10 with 0.25 standard deviation. Figure 4.17 shows the effect of OPI on the failure probability for a given service life. The figure shows a very significant difference in performance between an OPI value of 9.1 and 10. For example, at 50 years the probability of failure for concrete with OPI value of 9.1 is 92% compared to 10% chances of failure for concrete with OPI value of 10.

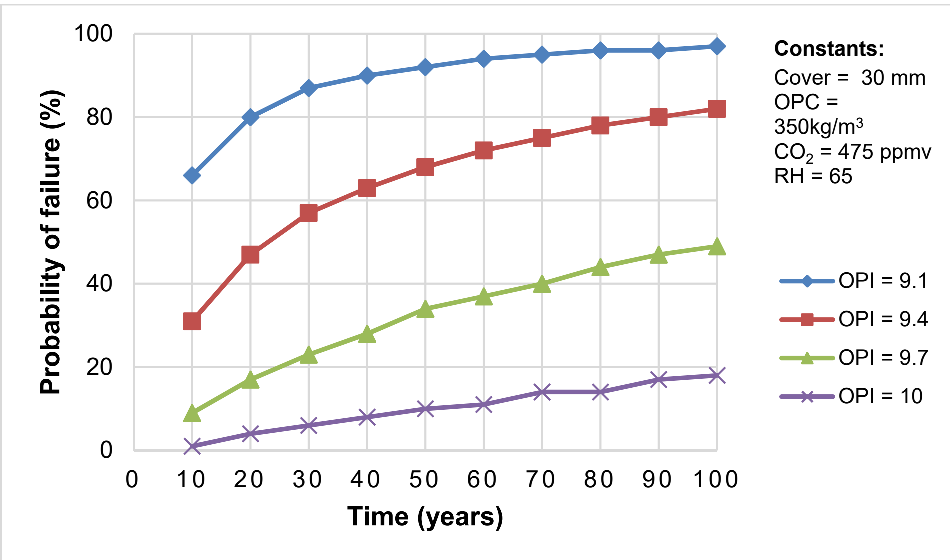

Figure 4. 17: Change in failure probability with time for different OPI values

4.4.3 Deterministic versus probabilistic analysis

The effect of the service life calculation is considerable when comparing deterministic with probabilistic analysis. This is illustrated in Figure 4.18, when the result of the durability performance analysis is presented for a 30 mm cover concrete and OPI of 9.4. The calculation is based on data presented in Table 4.5.

Table 4. 5: Parameter values for simulation

Parameter	Distribution	Mean	Standard deviation
Cover depth (mm)	Log - Normal	30	6
OPI ($\times 10^{-10} m/s$)	Normal	9.4	0.25
Binder content (kg/m^3)	Deterministic	350	-
CO_2 conc. (ppmv)	Deterministic	475	-
RH (%)	Deterministic	65	-

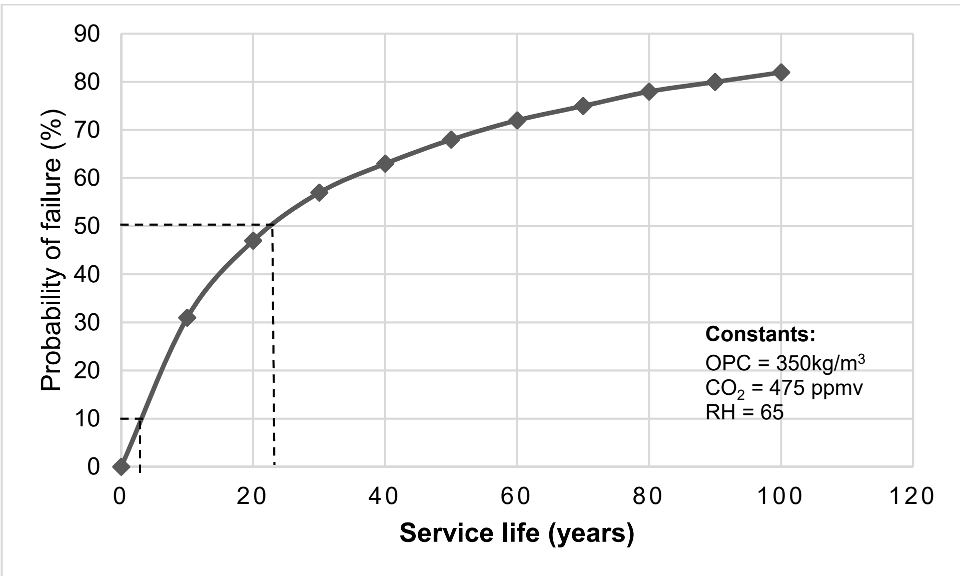

Figure 4. 18: Deterministic versus probabilistic analysis

Based on the data in Table 4.5, a probabilistic calculation of the service life was performed. The result of the calculation is shown in Figure 4.18. At 50% probability for initiation of corrosion (depassivation), the service life is 25-years. The 50% probability corresponds to the deterministic result. The International Federation for Structural Concrete (fib, 2006) prescribes a limit-state failure probability for serviceability equal to 10% for structures with service life greater than or equals to 50 years. For a serviceability limit state of 10% (probability of corrosion initiation), the computed service life duration is only 5 years. The probabilistic approach helps to put the results into perspective. For the same set of input parameters, the results show a 25-year service life with 50% chances of failure compared to a 5-year service life with a 10% chance of failure.

4.5 Summary

As presented in this chapter, the MS Excel program developed can equally help inform the designer and engineer on the sensitivity of the input parameters on the model results. The analysis of the sensitivity of the service life to each of the five model parameters informed on the sensitivity of the developed model. The sensitivity of the service life (model output) to the different parameters under investigation was performed by analysis of the effect of the variation on the outcome of the simulation. As a summary of the sensitivity analysis presented in this chapter, Table 4.6 shows to what extent the different parameters influence the performance of the RC structure. The effect of an increase of a parameter on the performance of the structure is presented.

Table 4. 6: Effect of increase of a parameter on the performance of the structure

Parameter	Service life
Concrete cover depth	Improves greatly
OPI	Improves greatly
Binder content	Improves
CO_2 concentration	Reduces
Relative humidity	Reduces up to an average RH of about 55% RH, then improves the performance with increase in RH

The results of a comparative analysis of the deterministic versus a probabilistic analysis showed that: the deterministic approach provides one single number with a 50% probability of failure, while a probabilistic analysis provides several values for service life with different failure probabilities. From a practical point of view, the probabilistic approach helps to put into perspective the computed or chosen value for the service life.

CHAPTER 5

5. Model application in practice

5.1 Introduction

The proposed model has been developed for the durability analysis of carbonating RC structures. Such durability analysis can either be applied for estimating the long-term performance of new structures or for conformity assessment of newly built structures. For the design of new structures, the designer is required to make certain assumptions concerning the parameters to be used for the simulation. Values for the OPI, cover depth, exposure parameters and concrete mix must be defined. For the conformity assessment of newly built structures, the concrete quality, cover depth and environmental loading can be measured directly on the structure with appropriate testing procedures.

To demonstrate how the proposed probabilistic model can be used for the conformity assessment of newly built structures, real data from an existing project is needed. The Gauteng Freeway Improvement project (GFIP) was used as a case study. To give an overview of the as-built performance (reliability) of the structural elements in the GFIP, data from two sub-projects undertaken between 2009 - 2010 located in Gauteng province from the study reported in Nganga et al. (2013) was analysed. The test results from the two sub-projects represent samples from both in-situ and precast elements. This consists of results from concrete cubes, test panels and structural elements in the GFIP.

5.2 Defining model parameters

The service life of the RC structures in the GFIP was defined as being 100 years (Nganga, 2011). However, no information was given in the project 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 absence of a proper definition and taking into account the characteristics of the proposed model, the serviceability limit state of depassivation of the reinforcement was chosen.

Information about the GFIP is presented in Appendix A. The data for OPI and concrete cover depths for both in-situ and precast elements were obtained from a study by Nganga et al. (2013). Concrete mix proportions were obtained from data provided by the ready-mix concrete producers in the GFIP project as reported by Nganga (2011). Information on the location and land use were extracted from the project brief as reported by Nganga (2011).

According to the European standard EN 1990 (2002), the probability of failure for a serviceability limit state should not exceed 7%, for a service life greater than 50 years. The International Federation for Structural Concrete (fib, 2006) on the other hand prescribes a limit-state failure probability for serviceability equal to 10% for structures with service life greater than 50 years. For this analysis, the value of 10% probability of failure for serviceability limit state prescribed by fib (2006) was chosen. This is because the fib Model code provides recommendations that serve as the basis for future codes for structures, with the work produced in fib (2006) and later published in 2010 being the most recent and comprehensive code on structures, which takes into account their complete life cycle (fib, 2010).

5.3 Measured as-built parameters

5.3.1 In-situ structural elements

Different tests were conducted on four bridges at different locations and measurements of compressive strength, DI values and concrete cover depth were taken (Nganga, 2011).

i. Mix-design proportions

Ready-mixed concrete was used for the in-situ structures. A summary of the concrete mix properties: w/b ratio, binder content and binder type as reported by Nganga (2011) is given in Table 5.1.

Table 5. 1: Summary of mix proportions of concrete used in production of in-situ structures (Nganga, 2011)

Mix constituents	**Proportions (kg/m^3)**
OPC	393
Fly ash	70
Total binder	463
Water content (l/m^3)	196
w/b ratio	0.44

ii. Concrete cover

The DI based specifications in the GFIP required measurement of cover depths of the finished structures. A summary of the cover depth results in terms of mean value, standard deviation value and coefficient of variation for the four bridges is summarized in Table 5.2 (Nganga, 2011).

Table 5. 2: Summary statistics of cover depth reading for in-situ structures (Nganga, 2011)

Location	Cover depth		
	Mean (mm)	**SD**	**CoV (%)**
A	46.6	8.9	19.0
B	50.7	7.5	14.7
C	58.0	17.1	29.4
D	55.3	12.1	21.9
Average	53	11.4	21.3

iii. OPI

A summary of the OPI measurements for the in-situ structures as reported by Nganga (2011) is presented in Table 5.3.

Table 5. 3: Summary statistics of OPI readings for in-situ structures (Nganga, 2011)

Project	Mean	Max	Min	SD	CoV (%)
1	9.91	10.42	9.37	0.22	2.24

5.3.2 Precast concrete elements

Project 2 consisted of precast concrete elements, where measurements were conducted on concrete median barriers (both double sided and single sided barriers) used on the freeway.

i. Mix design

A summary of the concrete mix proportions for the concrete used in the production of the precast elements, w/b ratio, binder content and binder type as reported by Nganga (2011) is presented in Table 5.4.

Table 5. 4: Summary of mix proportions of concrete used in production of the precast elements (Nganga, 2011)

Mix constituents	Proportions (kg/m^3)
OPC	410
Fly ash	176
Total binder	586
Water content (l/m^3)	220
w/b ratio	0.38

ii. Cover depth

The summary statistics for the cover depth readings from pre-cast elements in the project are presented in Table 5.5 (Nganga, 2011). The mean value is lower in comparison to that of the in-situ structures, as majority of the values complied with the limiting value of 40 mm.

Table 5. 5: Summary statistics of cover depth reading for precast concrete elements (Nganga, 2011)

Project	Cover depth (mm)		
	Mean	SD	CoV (%)
2	45.1	2.8	6.3

iii. OPI

The summary of the OPI measurements for the precast elements is presented in Table 5.6 (Nganga, 2011).

Table 5. 6: Summary statistics of OPI readings for precast concrete elements (Nganga, 2011)

Project	Mean	Max	Min	SD	CoV (%)
2	10.25	10.70	9.85	0.18	1.75

5.4 Results of conformity assessment

For conformity assessment of the structural elements in the GFIP, an analysis was performed to evaluate the as-built failure probability. The conformity assessment helps to quantify the difference that exist between the design and the as-built performance and to determine whether the as-built structure meets or exceeds the design performance. The assessment takes into account the variability observed in the OPI and the concrete cover depth values to compute the performance of the structure.

In the analysis, the values measured and reported by Nganga (2011) were chosen to be the as-built parameters. The statistical information (standard deviation and distribution function) for the as-built OPI and cover depth was chosen from measurements reported by Nganga (2011) and described in Section 5.3. Table 5.7 presents a summary of the input parameters used for the analysis of the different projects presented above. The deterministic parameters (binder type, binder content, location and land use) are described using a single value. Since the entire project was conducted in the Gauteng province of South Africa, the project location for both projects was set as Johannesburg. City/urban area was selected as the project land use.

The symbol N(…) indicates that the data has a normal distribution. The values between the bracket are the mean value and the standard deviation of the normal distribution. LN(…) indicates a log-normal distribution. D(…) symbolises that the data is deterministic, hence only a single value is given.

Table 5. 7: Input parameters for probabilistic analysis

Parameter	**Measured values (In-situ structures)**		**Measured values (Precast structures)**	
Binder type	OPC + FA		OPC + FA	
Binder content (kg/m^3)	OPC	D(393)	OPC	D(410)
	FA	D(70)	FA	D(176)
Location	Johannesburg		Johannesburg	
OPI	N(9.91, 0.22)		N(10.25, 0.18)	
Cover depth (mm)	LN (53, 11.4)		LN(45.1, 2.8)	
Land use	City		City	

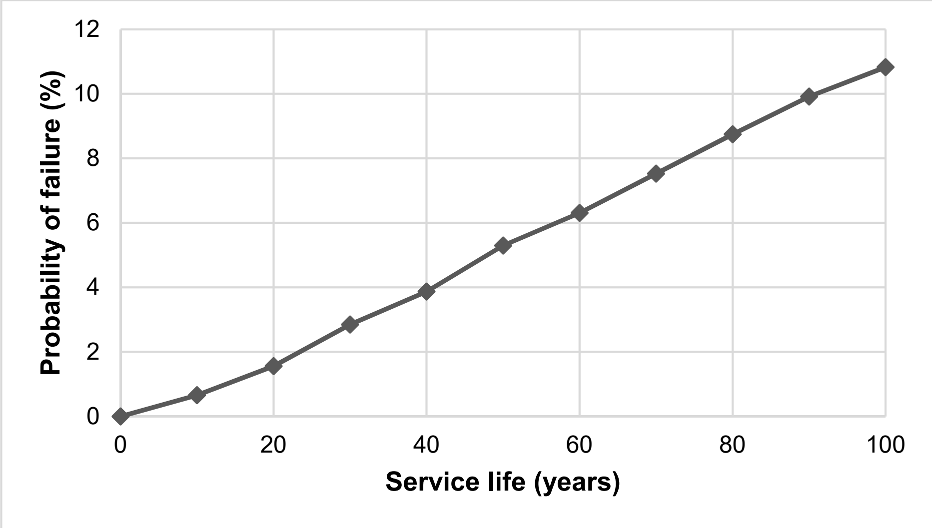

Figure 5. 1: Probability of failure versus time for in-situ RC structures

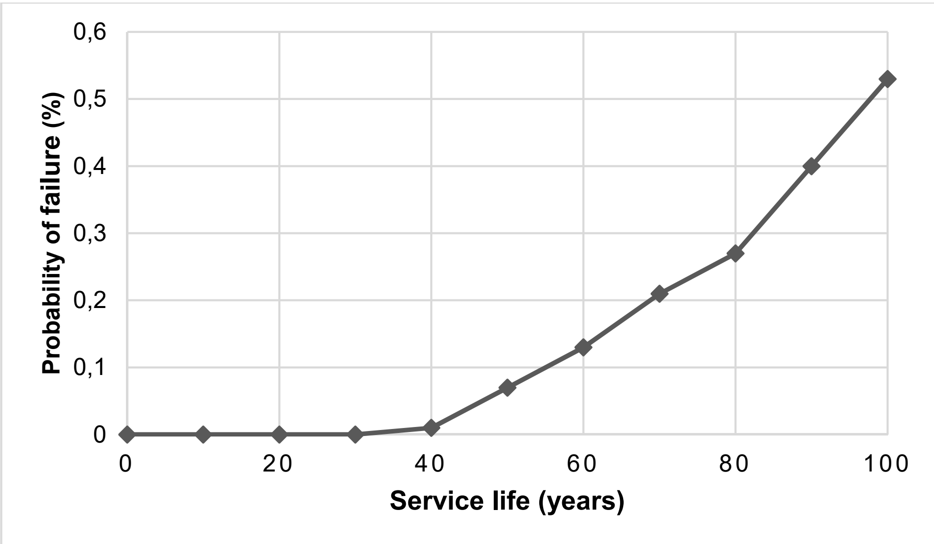

Figure 5. 2: Probability of failure versus time for precast RC structures

Figure 5.1 and 5.2 presents results of the as-built failure probability for in-situ and precast RC structures respectively. For both in-situ and precast structures, the requirement of 100-year service life is attained. The probability of failure is significantly higher for the in-situ structures compared to the precast concrete structures.

From the results, the probability of failure increases as the service life increases. For example, we have a failure probability of 5.3% and 0.07% for a service life of 50 years compared to 11% and 0.53% failure probability for a service life of 100 years for in-situ and pre-cast structures respectively. If we assume a 10% acceptable probability of failure as prescribed by the

International Federation for Structural Concrete (fib, 2006), we have a service life of 90 years for in-situ structures.

5.5 Summary

The result of the analysis of the durability performance of the RC structures in the GFIP demonstrates how the developed model can be exploited by designers and engineers in the field of durability design. The program offers users the ability to perform durability design of new structures and to perform conformity assessment of newly built structures. Data from conformity assessment of existing and newly built structures is used to predict the as-built performance of the structure.

From the quality control of both in-situ and precast structures in the GFIP, the requirement of 100-year service life is attained. However, the probability of failure is significantly higher for the in-situ structures compared to the precast concrete structures. This can be due to: materials used in production of concrete; quality of workmanship and execution of construction practices on site; and quality of testing in laboratories, which is influenced by apparatus used and skill of operators.

So far, durability design has been done using deterministic studies, with a single value for service life given which considers average conditions with 50% chances of failure. The above analysis help to provides a quality control of the as-built structure using data measured directly on the existing project. From a practical point of view, the probabilistic approach helps to put into perspective the computed or chosen value for the service life.

CHAPTER 6

6. Conclusions and Recommendations

6.1 Summary

This research study proposed a computer-based model for service life design of carbonating RC structures that can be used by engineers in practice. The model is based on sound physical and chemical principles, simple and user friendly, and includes probabilistic considerations in estimating the service life of RC structures. The developed model was based on a critical review of literature to select an appropriate model relevant to the South African Durability Index approach, determine the relevant limit state function parameters for the model, defining statistical information (distribution function, mean and standard deviation values) for the limit-state function parameters and modelling of carbonation initiation in terms of probability of exceeding the limit state in MS Excel. The conclusions from the study pertain to three distinct aspects.

i. The improvement of the existing deterministic carbonation model to a simple and user-friendly probabilistic carbonation model.
ii. The trend and sensitivity analysis of the input parameters to the model output (service life).
iii. The role and contribution of the developed model as a performance design and conformity assessment tool.

6.2 The improvement of the existing deterministic carbonation model to a simple and user-friendly probabilistic carbonation model

As Discussed in Section 1.4, the existing South African carbonation models, used to predict the service life of RC structures, are deterministic in nature, with each parameter of the model characterized by a mean value. Furthermore, common carbonation prediction models are generally very detailed and require detailed knowledge on the carbonation process and the cement chemistry. The carbonation model developed by Salvoldi (2010) was used as the base for the model developed in this research. The thus updated model is comprehensive, scientifically rigorous and accounts for important aspects of concrete carbonation on a fundamental level. The parameters in the model were grouped into concrete mix parameters, exposure parameters, a geometry parameter, and a parameter describing the concrete micro-structure (material parameters).

i. Concrete mix parameter: The concrete mix parameters describe the amount of carbonatable material in the concrete matrix. Standard compositions of binder types used in South Africa are already integrated in the program and can easily be identified for standard cement replacements adopted in industry (100% PC, 10% SF, 30% FA, and 50% GGBS).
ii. Exposure parameters: The variables which define the exposure conditions in the model are the relative humidity and the ambient carbon dioxide concentration. These parameters are already integrated in the program and can easily be identified for prevailing land use and location of the structure in South Africa.
iii. Geometric parameter: The geometric parameter is characterized by the concrete cover depth. Because of the random nature of the concrete cover depth for a single structure, the concrete cover in the model is represented as a random variable (distribution function, mean and standard deviation).
iv. Concrete micro-structure parameter: This parameter is characterized by the OPI, which is strongly influenced by the concrete composition (w/b ratio and binder composition), as well as construction-related parameters such as compaction and curing. Because of the somewhat random nature of the physical properties of the concrete cover, the OPI value in the model is represented as a random variable (distribution function, mean and standard deviation).

The model parameters were effectively represented in a limit-state function analogous to structural design theory. Monte Carlo simulation is used to compute values for carbonation depth development in the concrete, which can be related to the expected service life duration. The failure probability is computed, which corresponds to the confidence level or risk of failure. With the probabilistic approach, a reliability value can be associated with the computed service life value. From a practical point of view this helps to put the computed results into perspective.

6.3 Trend and sensitivity analysis of the input parameters to the model output

The sensitivity of the service life to the five model parameters – binder content and type, location, land use, OPI and concrete cover depth was analysed. The results revealed that the model is sensitive to the different input parameters. The analysis of the trends between the input parameters and the model output (service life) corroborated the information in literature; consequently, leading to the expected observations listed in the following. Notably, the model allows to quantify the influence that each of these parameters, or any combination of these parameters, has on the expected service life duration.

i. Binder content: An increase in binder content improves the service life result, as it is associated with an increase in the amount of carbonatable material and hence slower carbonation progression. It is important to note that these results are based on a constant OPI. In practice however, an increase in binder content could result in a lower (or higher) OPI.

ii. Carbon dioxide concentration: A decrease in CO_2 concentration improves the service life results.

iii. Relative humidity: The effect of relative humidity on the service life revealed that lower service life results are obtained at medium range relative humidity environment.

iv. Concrete cover: An increase in concrete cover depth greatly improves the service life results. The result of the sensitivity of the model output to the randomness of the concrete cover revealed that an increase in the variability of the concrete cover for a single structure greatly influences the failure probability and consequently the structural performance.

v. OPI: Similar to the concrete cover depth, an increase in the OPI value greatly improves the service life results. An increase in the variability of the OPI for a single structure greatly influences the failure probability and as result the structural performance.

6.4 The role and contribution of the developed model as a performance design and conformity assessment tool

Performance-based durability design and specification for RC structures have been adopted by the South African National Road Agency Limited (SANRAL). The new standard specifications for national roads and bridges, issued by the Committee of Transport Officials (COTO) adopted the performance-based approach and introduced a section for 'Durability Concrete' (Nganga, et al., 2017). The performance-based approach requires durability considerations to be introduced in the design of the concrete mixture and provides a quantification of the expected service life duration. The contribution of the developed MS Excel program as tool for performance design pertains to two distinct aspects, as discussed in the following.

i. The design of new RC structure

The developed service life design program can help engineers in practice at the design phase for estimating long-term performance of new RC structures. Depending on the exposure conditions, the binder type and content, the OPI and cover depth, the developed MS Excel program can be used to compute the expected service life duration. From the result of the analysis, a suitable combination of limiting values for oxygen permeability index (OPI) and cover depth can be extracted for use in performance specification.

ii. Conformity assessment of newly built RC structures.

The developed service life design program can equally be used for conformity assessment of newly built RC structures. For the conformity assessment of newly built structures, the as-built values of the parameters are measured directly on the structure. That is; both the concrete quality (represented by the OPI value) and cover depth are measured directly on the structure with the appropriate testing procedures. The MS Excel program can then be used to compute the probability of failure for the different structural components under investigation. This helps to measure the ability of the structure to fulfil the specified function during the design service life.

From the above discussion, the developed MS Excel program provides interesting features as a decision-making tool for the durability optimization of RC structures. The MS Excel program offers the users the ability to perform durability design of new RC structures with information about the binder characteristics and environmental conditions integrated in the program. The program can be used as a tool for conformity assessment of newly built RC structures. It equally takes into account probabilistic considerations in order to make the relevant prediction. All the above-mentioned benefits of the developed program can contribute to enhancing the newly adopted performance-based approach in durability design.

6.5 Recommendations for future research

The objective of this research was to develop a probabilistic, computer-based model for the prediction of carbonation in concrete structures, which can be used by engineers in practice. The computer model developed in this research is based on sound scientific principles and allows for continuous improvement as new information and knowledge are made available with time. Based on the findings of this research, the following recommendations for future research are given:

i. Further work should be done to improve the user-friendliness of the software program. This involves improving simultaneous assessment of the program and graphical display facilities of the program.

ii. Further work is needed in updating the software program to include newer and updated models and to consider the corrosion propagation stage with other limit-state criteria such as rebar corrosion-related cover cracking.

iii. Further research is also needed to improve the current knowledge on the variability of the parameters used in the service life model. For example, further measurements of OPI and cover depth should be conducted on various projects to build up a solid data base and hence provide more representative results of the statistical nature of these parameters.

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8. Appendices

APPENDIX A – Gauteng Freeway Improvement Project (GFIP)

8.1 Gauteng Freeway Improvement Project (GFIP)

8.1.1 Project description

The South African National Road Agency Limited (SANRAL) is responsible for the development and maintenance of the national road network in South Africa. In 2007, SANRAL initiated a 561km road network project in the Gauteng province of South Africa. A project that crosses the Johannesburg, Ekurhuleni and Tshwane metropolitan boundaries. The Gauteng Freeway Improvement Project (GFIP) is aimed at alleviating the traffic congestion problems experienced in the Gauteng province of South Africa. The project's scope was to upgrade the road network through expansion, by adding extra lanes to existing freeways, constructing and improving of interchanges, and constructing of concrete median barriers on the freeways (SANRAL, 2010). A plan of the area covered by the GFIP is illustrated in Figure 8.1 below.

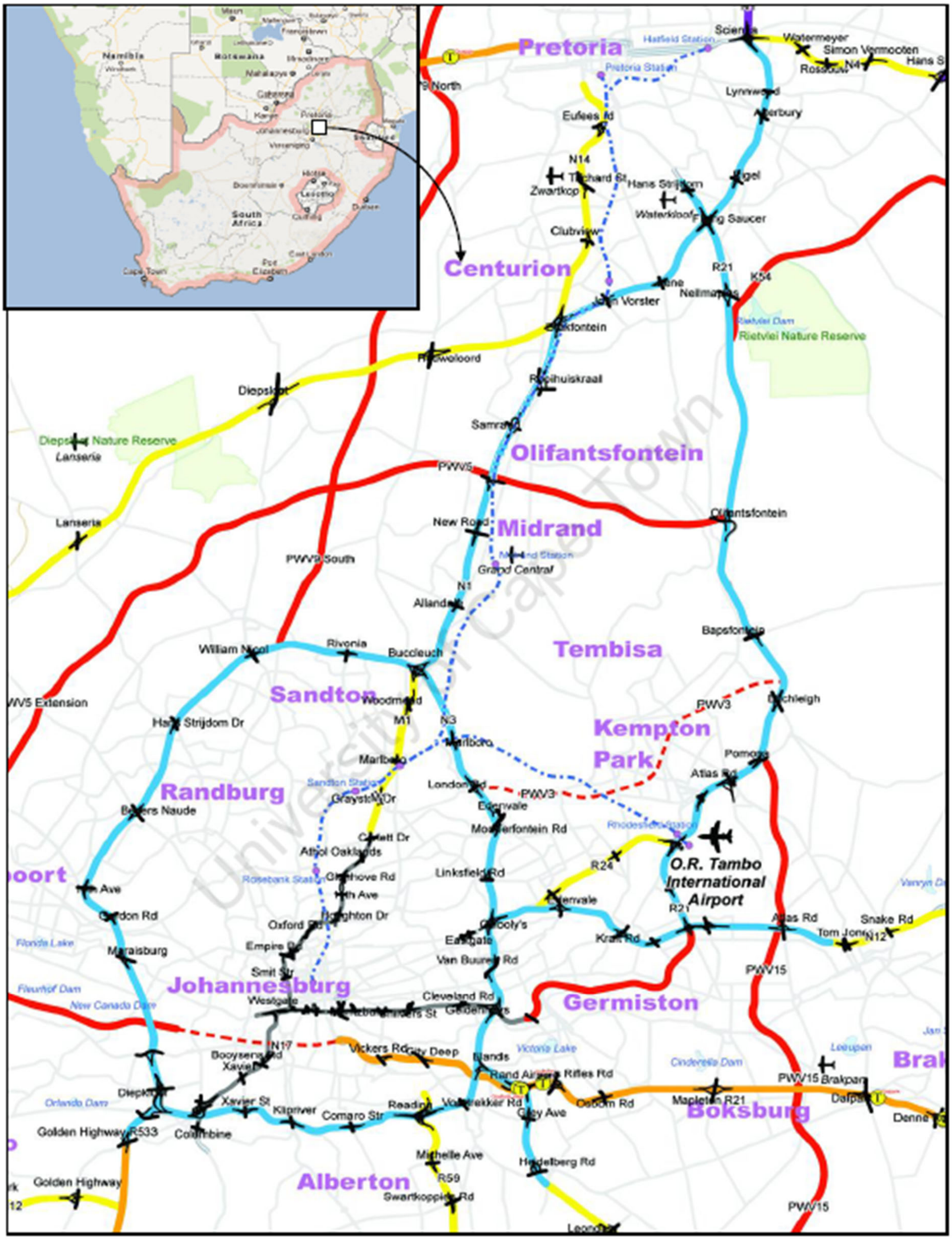


Figure 8. 1: Schematic representation of the area covered in the first phase of the Gauteng Freeway Improvement Project (GFIP) - highlighted in blue (SANRAL, 2010).

8.1.2 Project data

The durability index performance-based specifications have been applied in the GFIP. The aim was to ensure the construction of durable RC structures with an expected service life of 100 years. In these specifications, concrete structures required measures on strength, DI values and cover depth (Nganga, et al., 2013).

i. Cover depth values

The project specifications in the GFIP required the measurement of concrete cover depth of all RC structural elements (designated as 'Class W' for durable concrete). The recommended minimum value for cover depth in this project was 40 mm for in situ and precast elements. Table 8.1 summarizes the acceptable ranges for cover depth and the allowed tolerance values.

Table 8. 1: Acceptance range for concrete cover adapted from (SANRAL, 2010).

Specified cover	**Minimum**		**Maximum**	
	Individual bar	overall	Individual bar	overall
40mm	70% of specified cover	85% of specified cover	Specified cover +25 mm	Specified cover +15 mm
Acceptable values	28mm	30mm	65mm	55mm

ii. Concrete mix proportions

The concrete for the GFIP was obtained from ready mix concrete producers in Gauteng. A summary of the ranges of mix properties is given in Table 8.2 and Table 2.3.

Table 8. 2: Summary of mix proportions for ready mix concrete, adapted from (SANRAL, 2010).

RMC Plant	Concrete mix code	Cement	GGBS	Fly Ash	Total binder	w/b ratio
	Kg/m³					
Plant: W	W140E4FA	383	0	68	451	0.44
	W140E4FC	396	0	70	466	0.44
	W140E4DA	398	0	70	468	0.44
Plant: X	W140E4EA	383	0	68	451	0.44
	W140E4EC	402	0	71	473	0.44
Plant: Y	W140E4FA	383	0	68	451	0.41
	W140E4EC	403	0	71	474	0.41
Plant: Z	W130E4FA	360	90	0	450	0.45
	W130E4FC	373	93	0	466	0.44
	W130E4DA	369	92	0	461	0.45

Table 8. 3: Summary of mix proportions of concrete used in production of precast elements, adapted from (SANRAL, 2010).

Mix constituents	Proportions (kg/m³)
Cement	410
Fly ash	176
Total binder content	586
Water content (l/m³)	220
Water/binder ratio (w/b)	0.38

iii. OPI values

The project specification in the GFIP required the measurement of OPI values of hardened concrete produced on site. The project exposure classes, according to EN 206 is XC3, i.e. concrete exposed to moderate humidity, such as inside buildings with moderate or high air humidity, or external concrete sheltered from the rain. The recommended minimum OPI values for these exposure classes in the project specifications are 9.4 and 9.0 (on a log scale) respectively (Nganga, et al., 2013). The values are linked to the durability prediction model for carbonation and consider the expected service life of the structure (100 years). Table 8.4 summarizes the proposed limiting values for OPI for the different structures in the GFIP.

Table 8. 4: Durability specifications for different carbonation exposure class for the GFIP – adapted from (Nganga, et al., 2013).

<table>
<tr><th>Durability indicator</th><th colspan="5">Environmental classification (concrete cover 40mm)</th></tr>
<tr><td rowspan="3">OPI</td><td>XC1</td><td>XC2</td><td colspan="2">XC3</td><td>XC4</td></tr>
<tr><td rowspan="2">-</td><td rowspan="2">-</td><td>Inside building with moderate/high humidity</td><td>< 9.4</td><td rowspan="2">-</td></tr>
<tr><td>External concrete sheltered from rain</td><td>< 9.0</td></tr>
</table>

APPENDIX B – User Guide

8.2. General

This Chapter presents a brief explanation of the developed software program. The software was implemented in MS Excel. The software represents the implementation of the probabilistic model proposed in this research. The software helps to facilitate the probability-based approach to durability analysis. The model can be used for durability design of new RC structures as well as a basis for condition assessment of existing RC structures. This section presents the user manual with simple step-by step explanation of how the program works, the input parameters required and the various options available to the user.

8.2.1 Starting the service life model

The service life design model is an MS Excel program. To start the service life model, simply select and double click on the program icon on your computer. When the file opens, the screen should look similar to Figure 8.2.

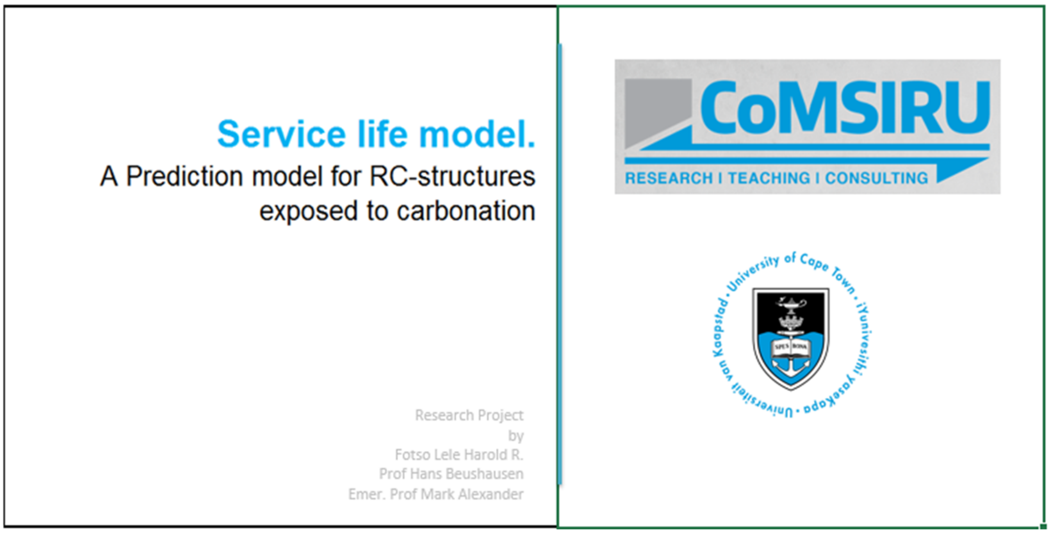


Figure 8.2: Starting sheet of the program

At the bottom of the screen, we have 4 worksheets (Figure 8.3).

5. The About sheet, which presents an overview of the program and the individuals that contributed to designing the program.
6. The SL-Model sheet, which contains the actual design tool.
7. The Randomotor sheet, which provides simulation values for the MCS analysis.
8. The Portlandite sheet, which computes the amount of portlandite for the given parameters.

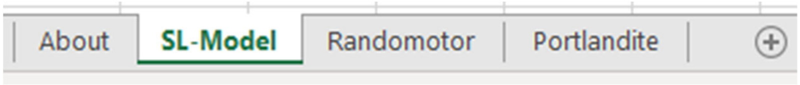


Figure 8. 3: Various accessible sheets

8.2.2 SL-Model sheet

To start a new project, select/click on the SL-model sheet. This sheet contains six main sections; a project description section, an input section, a key section, an output section, a summary graph and a summary statistics section (Figure 8.4). To conduct the analysis, each section needs to be completed with the necessary information. Each of the sections is discussed below, together with relevant illustrations.

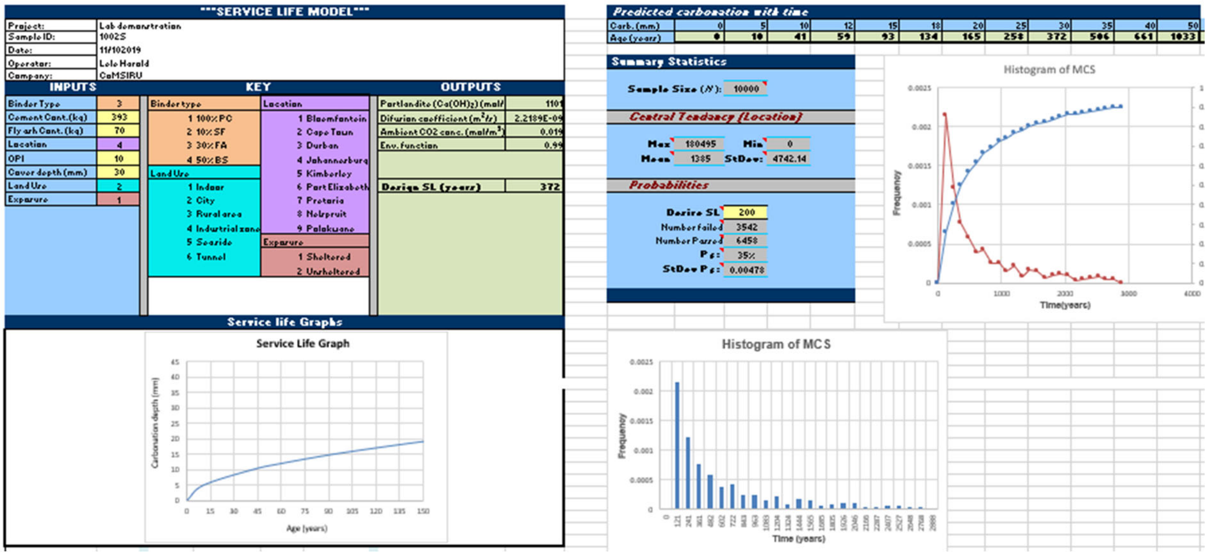


Figure 8. 4: Overview of SL-model sheet

a. Project description section

In the project description section, the information about the project is introduced. This include; the project title, sample ID, the operator’s identity and the company name. An example is given in Figure 8.5.

SERVICE LIFE MODEL	
Project:	Lab demonstration
Sample ID:	1002S
Date:	11/102019
Operator:	Lele Harold
Company:	CoMSIRU

Figure 8. 5: Project description section

b. Input section

Next is the input section. In this section, information about the carbonation parameters are introduced. This include; the binder type, binder content, the project location, OPI value, cover depth value, the land use and the project exposure condition. The design values are required here for structures at the design stage and measured values are used for an existing project. (Figure 8.6).

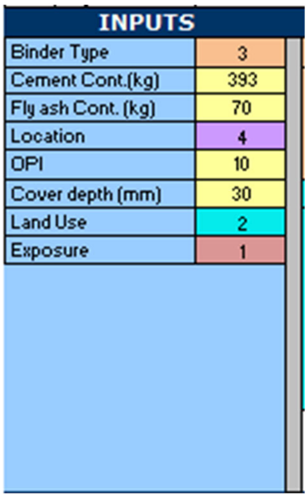


Figure 8.6: Definition of the carbonation parameters in the input section

i. Binder type

The relevant binder type for the project is selected from the key provided (Figure 8.6). Standard binder compositions available include; 100% PC, PC+10%SF, PC+30%FA, or PC+50% BS Figure 8.6.

ii. Binder content

Once the binder type is selected, the quantities in (kg) of the different binders are entered.

iii. Location

Various locations around South Africa are provided in the key section (Figure 8.6). The project location is selected amongst the different locations provided. This information will be used to estimate the average RH and the average temperature based on historical data collected in the different locations.

iv. OPI

Next, the design OPI value is entered.

v. Concrete cover

Information about the design concrete cover depth for the project or the specific element is entered.

vi. Land Use

The carbon dioxide concentration is a function of the environment around the structure (land use). Here, land use around the structure is chosen from a list of predefined land uses, as shown in Figure 8.7.

vii. Exposure

The final information required is the exposure environment. Either the sheltered or Unsheltered condition is selected as shown in Figure 8.7.

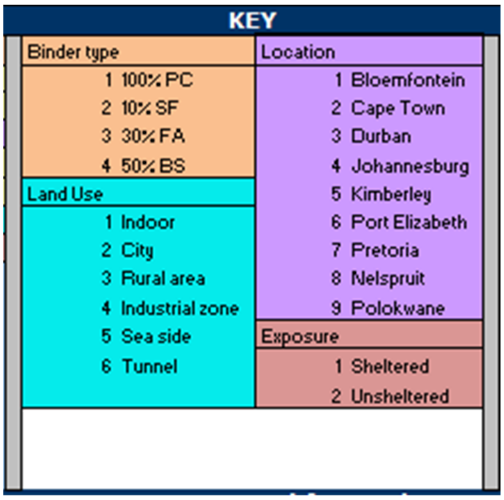


Figure 8. 7: Key section

c. Output section

After all the parameters in the input section have been entered, the simulation is initiated immediately. Once the simulation has ended, the results are presented in the output section as shown in Figure 8.8.

i. Portlandite

This shows the total amount of carbonatable material ($Ca(OH)_2$) present in the material sample (*mol/dm*^{3}).

ii. Diffusion coefficient

Here, the diffusion coefficient in (*m^2/s*) of the concrete sample is displayed.

iii. Ambient carbon dioxide concentration

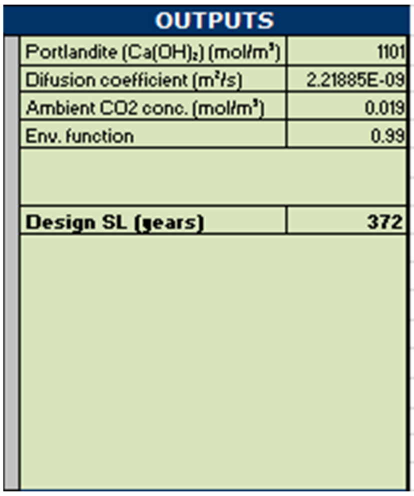
The carbon dioxide concentration in *mol/m^3* of the area surrounding the project location is computed here.

iv. The Environmental function

This parameter takes into account the influence of relative humidity, temperature around the project site.

v. Design Service life

An average value for the design service life in years is computed.



OUTPUTS	
Portlandite ($Ca(OH)_2$) (mol/m^3)	1101
Difusion coefficient (m^2/s)	2.21885E-09
Ambient CO2 conc. (mol/m^3)	0.019
Env. function	0.99
Design SL (years)	**372**

Figure 8. 8: Output section

d. Service life graph section

Once the simulation has ended, the user has an option to view the outcome graphically. A graph of the performance of the RC structure with time is displayed in the graph section (Figure 8.9).

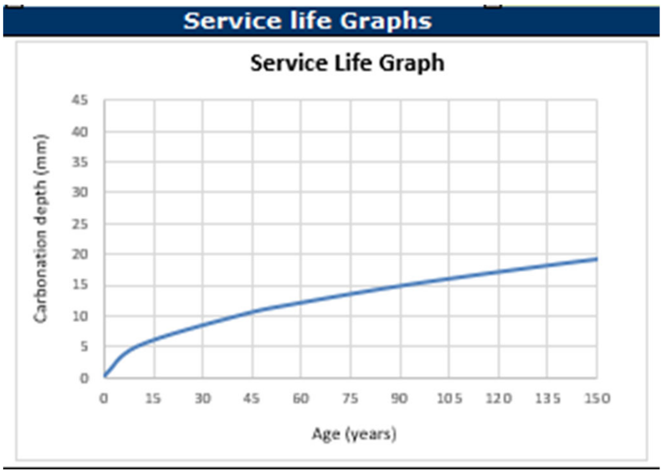


Figure 8. 9: Service life graph section

e. Summary statistics section

The summary statistics section provides a resume of the probabilistic analysis. This consist of the central tendencies, the probability values and the probability of failure vs time graph as shown in Figure 8.10.

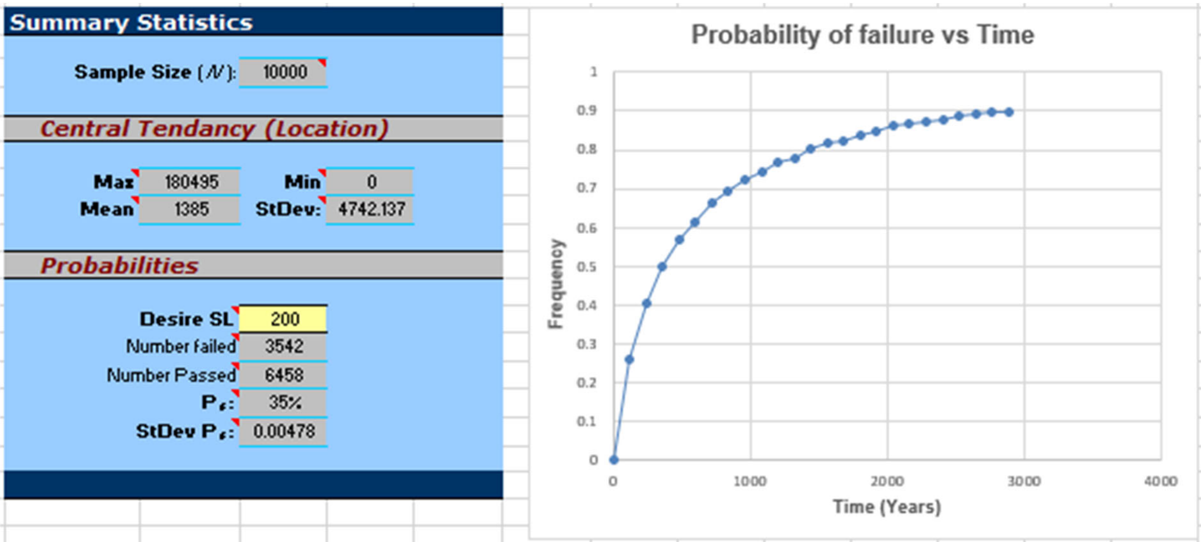


Figure 8. 10: Summary statistics and graphs

8.2.3 Randomotor sheet

The randomotor sheet (Figure 8.11) computes the service life for 10,000 different simulations for the different random parameters. The random variable considered here are OPI and concrete cover depth. The OPI value is approximated as a normal distribution, while the concrete cover is approximated as a log-normal distribution. Once the 10,000 simulations are completed, statistical information of the computed service life is extracted (refer to Section 3.4 for more details).

Randomoteor : Creates randomly sampled inputs of from different type of distribution

Uniform distribution		
Variable name	Mean	Stdev
K-value	1	7.16E-01
OPI	10	0.27

Log-normal distribution		
Variable	Mean	Stdev
Concrete cover	0.03	0.006

Min:	0
Max:	1613256
N:	5000

Vertical (Value) Axis Major Gridlines

Empty>>

Trial	Rand (Prob.)	OPI	Diff_coef	R_cover	Cover	SL
1	0.50960725	10.0065	2.147E-09	1.030603	30	574
2	0.846231764	10.2755	5.4955E-10	1.036783	36	3230
3	0.72380973	10.1604	9.8441E-10	1.034135	34	1608
4	0.445124879	9.96274	2.6797E-09	1.029602	29	430
5	0.165795303	9.73785	8.3724E-09	1.024469	24	94
6	0.565991389	10.0449	1.7677E-09	1.031482	31	745
7	0.090388159	9.63864	1.3839E-08	1.022213	22	48
8	0.351055238	9.89673	3.7438E-09	1.028093	28	287
9	0.545144178	10.0306	1.9001E-09	1.031156	31	693
10	0.206436582	9.77891	6.8002E-09	1.025404	25	126
11	0.462349833	9.97448	2.5251E-09	1.02987	29	456
12	0.1945275	9.76744	7.2071E-09	1.025143	25	119
13	0.375079837	9.91402	3.4298E-09	1.028488	28	313
14	0.89388412	10.3368	4.0285E-10	1.038196	37	4655
15	0.261796429	9.82779	5.3087E-09	1.026519	26	174
16	0.646302625	10.1013	1.3279E-09	1.032778	32	1056
17	0.482126483	9.9879	2.3591E-09	1.030177	30	523
18	0.838586849	10.2669	5.7393E-10	1.036585	36	3093
19	0.461061036	9.9736	2.5363E-09	1.02985	29	454
20	0.079936511	9.62052	1.517E-08	1.021801	22	44
21	0.278909757	9.84176	4.9461E-09	1.026837	26	187
22	0.466768601	9.97748	2.4869E-09	1.029939	29	463
23	0.776967242	10.2057	7.8254E-10	1.035177	35	2144
24	0.759181756	10.19	8.4751E-10	1.034814	34	1868
25	0.959421902	10.4709	2.0426E-10	1.041294	40	10729
26	0.347209517	9.89393	3.7974E-09	1.028028	28	283
27	0.271987461	9.83616	5.0883E-09	1.02671	26	182
28	0.774437135	10.2035	7.9164E-10	1.035124	35	2120
29	0.021588013	9.45406	3.5253E-08	1.018029	18	13
30	0.679667364	10.126	1.1718E-09	1.033344	33	1273
31	0.470681937	9.98014	2.4537E-09	1.03	30	502
32	0.9768125	10.5378	1.4551E-10	1.042844	42	16604
33	0.583212921	10.0567	1.6646E-09	1.031754	31	791
34	0.853548572	10.284	5.2647E-10	1.036978	36	3372
35	0.085841827	9.63096	1.4388E-08	1.022038	22	46
36	0.951450731	10.448	2.2942E-10	1.040763	40	9552
37	0.058180298	9.57604	1.9004E-08	1.020792	21	32
38	0.794888422	10.2223	7.194E-10	1.035559	35	2332

Bins	Count	Scaled	Total
0	1	6.19865E-08	0.0002
3227	4191	0.000259785	0.8384
6453	406	2.51665E-05	0.9196
9680	162	1.00418E-05	0.952
12906	67	4.15309E-06	0.9654
16133	35	2.16953E-06	0.9724
19359	31	1.92158E-06	0.9786
22586	20	1.23973E-06	0.9826
25812	13	8.05824E-07	0.9852
29039	13	8.05824E-07	0.9878
32265	11	6.81851E-07	0.99
35492	4	2.47946E-07	0.9908
38718	5	3.09932E-07	0.9918
41945	4	2.47946E-07	0.9926
45171	1	6.19865E-08	0.9928
48398	5	3.09932E-07	0.9938
51624	7	4.33905E-07	0.9952
54851	4	2.47946E-07	0.996
58077	0	0	0.996
61304	2	1.23973E-07	0.9964
64530	3	1.85959E-07	0.997
67757	3	1.85959E-07	0.9976
70983	1	6.19865E-08	0.9978
74210	3	1.85959E-07	0.9984
77436	0	0	0.9984
	8	4.95892E-07	1

About | SL Model | Randomotor | Portlandite Ca(OH)2 | Carbonation

Figure 8. 11: Randomotor sheet

8.2.4 Portlandite sheet

The portlandite sheet presents the background equations used to compute the amount of calcium hydroxide consumed during the carbonation process. It uses the hydration model proposed by Papadakis (1991-b) to compute the amount of portlandite (refer to Section 3.3 for more details).

INPUTS			
Cement Cont. (g)	425000		
Silica fume Cont.	42500		
Location	1		
Age(days)	28		
OUTPUTS			
RH (%)	53		
Amb Temp (K)	297.65		
Alpha RH	3.90184E-06		
Alpha T	1.000185423		
Alpha b_SF	0.97		
Hydration degree & Pozzolanic activity			
Due to Curing Effect & temp.		Normal hydration process	
f_{C3S}	3.55813E-06	f_{C3S}	0.911899893
f_{C2S}	2.62313E-06	f_{C2S}	0.672252499
f_{C2AF}	3.08035E-06	f_{C2AF}	0.789446129
f_{C3A}	3.75097E-06	f_{C3A}	0.96132739
p_{SF}	2.27545E-06	p_{SF}	0.583141994
p_{FA}	1.28445E-06	p_{FA}	0.32915637
p_{BS}	1.13595E-06	p_{BS}	0.29110185
C_3S	1251.168224	C_3S	1251.168224
C_2S	268.9873418	C_2S	268.9873418
C_3A	141.6666667	C_3A	141.6666667
C_4AF	105.0052503	C_4AF	105.0052503
CSH_2	148.1077062	CSH_2	148.1077062
Degree Pozzolanic activity			
Due to Curing Effect & temp.		Normal Pozzolanic process	
CaO	0	CaO	0
Al_2O_3	4.168301295	Al_2O_3	4.168301295
SiO_2	672.0206391	SiO_2	672.0206391
CH (mol/m^3)	148.1105799	CH (mol/m	884.6116368
		Total CH (mol/m3)	**1032.722**

Figure 8. 12: Portlandite sheet

APPENDIX C – Assessment of ethics form

Application for Approval of Ethics in Research (EiR) Projects
Faculty of Engineering and the Built Environment, University of Cape Town

APPLICATION FORM

Please Note:
Any person planning to undertake research in the Faculty of Engineering and the Built Environment (EBE) at the University of Cape Town is required to complete this form before collecting or analysing data. The objective of submitting this application *prior* to embarking on research is to ensure that the highest ethical standards in research, conducted under the auspices of the EBE Faculty, are met. Please ensure that you have read, and understood the EBE Ethics in Research Handbook (available from the UCT EBE, Research Ethics website) prior to completing this application form: http://www.ebe.uct.ac.za/ebe/research/ethics1

APPLICANT'S DETAILS		
Name of principal researcher, student or external applicant		FOTSO LELE HAROLD ROMUALD
Department		CIVIL ENGINEERING
Preferred email address of applicant:		HRLFOT001@MYUCT.AC.ZA
If Student	Your Degree: e.g., MSc, PhD, etc.	MSc
	Credit Value of Research: e.g., 60/120/180/360 etc.	120
	Name of Supervisor (if supervised):	Prof. HANS BEUSHAUSEN
If this is a researchcontract, indicate the source of funding/sponsorship		CoMSIRU
Project Title		Computer Based Probabilistic Model for Service life Design of Concrete structures in Carbonation

I hereby undertake to carry out my research in such a way that:

- there is no apparent legal objection to the nature or the method of research; and
- the research will not compromise staff or students or the other responsibilities of the University;
- the stated objective will be achieved, and the findings will have a high degree of validity;
- limitations and alternative interpretations will be considered;
- the findings could be subject to peer review and publicly available; and
- I will comply with the conventions of copyright and avoid any practice that would constitute plagiarism.

SIGNED BY	Full name	Signature	Date
Principal Researcher/ Student/External applicant	FOTSO LELE HAROLD ROMUALD		29 Jan 2019

APPLICATION APPROVED BY	Full name	Signature	Date
Supervisor (where applicable)	Prof. HANS BEUSHAUSEN		29 Jan 2019
HOD (or delegated nominee) Final authority for all applicants who have answered NO to all questions in Section1; and for all Undergraduateresearch (Including Honours).	DG Randall		25 Feb 2019
Chair : Faculty EIR Committee For applicants other than undergraduate students who have answered YES to any of the above questions.			

Page 1 of 2