UNIVERSITY OF CAPE TOWN

DURABILITY SPECIFICATIONS FOR STRUCTURAL CONCRETE – AN INTERNATIONAL COMPARISON

By

Justine G. Kessy

Thesis Supervisor: Prof. Mark G. Alexander

Co-Supervisor: Assoc. Prof. Hans Beushausen

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DECLARATION

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

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ABSTRACT

Premature deterioration of reinforced concrete (RC) structures has become an issue of global concern. As a result, many upgrades and improvements have recently been made in design standards and specifications, to include requirements that account for durable RC structures. This dissertation examines and compares such durability requirements in design standards and specifications developed in the United States of America, Australia, Canada, Europe, India, and South Africa. It discusses issues relating to exposure conditions, limiting values of material compositions and proportions, and cover depth to the reinforcing steel. Both prescriptive and performance requirements for concrete durability are described. In general terms, this dissertation concludes that most design standards are based on prescriptive requirements with a few having some elements of performance requirements for durability design.

The prescriptive approach that outlines requirements for material compositions and proportions, procedures, and test methods, is commonly used in most design standards and specifications for durability purposes. Though such approaches may encompass requirements for, inter alia, minimum compressive strength, maximum water-to-cementitious material (w/cm) ratio and cover depth, the desired concrete performance is not generally described. Material and construction variability are not taken into account, and even if intensive construction supervision is carried out, it is difficult to ensure all specified parameters are achieved. Moreover, requirements such as maximum w/cm and minimum water content are impractical or costly to measure or verify in practice. Generally, it should be acknowledged that this approach has limited applications and often stifles innovations.

In an attempt to move away from the prescriptive approach, research has focused on performance approaches, which measure relevant properties of the concrete, in particular transport-related properties that account for durability. Performance approaches impose few or no restrictions on the concrete composition, proportioning, or construction methods, but rather promote innovations. Worldwide there is a consensus that in order to extend the service life of RC structure, performance approaches are imperative.

This dissertation gives an overview of the international efforts in the implementation of performance approaches, either in design standards or in project specifications. Performance

test methods are also discussed with respect to their strengths and weaknesses in terms of evaluating concrete performance. Case studies, both hypothetical and practical, that demonstrate application of performance requirements are included. Based on the experience from international efforts, it is concluded that performance requirements are a viable alternative to the current requirements in South Africa, which are mainly prescriptive.

This dissertation proposes changes to South African Standards, particularly SANS 10100-2, in order to follow the international trends. Both prescriptive and performance requirements for concrete durability are considered in a possible revised SANS 10100-2. EN 206-1 though still prescriptive, is considered in SANS 10100-2 in terms of exposure classifications and limiting values for material compositions and proportions. The requirements presented widen the prescriptive specifications while allowing the use of hybrid specifications prior to fully adopting performance-based specifications.

This dissertation also proposes the Durability Index (DI) values recently developed in South Africa to be incorporated into SANS 10100-2 under hybrid specifications. Challenges related to an integrated approach that incorporate refined DI values and service life models applied under performance-based specifications are discussed.

Guidelines for durability specifications that include prescriptive, hybrid and performance-based specifications suitable for South African construction industry are presented. The guidelines present roles and responsibilities, and indicate the risk of each stakeholder involved in the construction industry. Recommendations in moving closer to the ultimate goal of developing durability specifications analogous to SANS 10100-2 are presented.

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LIST OF DEFINITIONS

Concrete cover Thickness between the surface of the reinforcement steel and the nearest concrete surface that provide protection against the ingress of deleterious substances.

Corrosion A state of deterioration of reinforcement steel due to either carbonation or chloride attack.

Design Service Life (DSL) Assumed period for which a structure or a part of it is to be used for its intended purpose.

Deterioration Worsening of condition with time, or a progressive reduction in the ability of a structure or its components to perform according to their intended functional requirements.

Durability The capability of structures, materials or products of continuing to be useful after an extended period of time and usage.

Hybrid specifications Specifications that rely only on durability indicators through Performance Test Methods (PTMs) where Design Service Life (DSL) is not given explicit. It is a combination of prescriptive and performance specifications.

Performance The behaviour of a structure or structural element as a consequence of actions to which it is subjected or which it generates.

Performance criteria Quantitative limits, associated to a performance indicator, defining the border between desired and adverse behaviour.

Performance specifications Set of clear, measurable, and enforceable instructions that outline the functional requirements for hardened concrete depending on the application. Both hybrid and performance-based specifications belongs to this category.

Performance-based specifications Specifications that rely on durability indicators through Performance Test Methods (PTMs) and measurements linked to Service Life Models (SLMs) for estimating Design Service Life (DSL).

Prescriptive specifications Set of instructions that outline material requirements, procedures and test methods, commonly specified in the design standards in order to achieve the intended functions.

Serviceability Ability of a structure or structural element to perform adequately for normal use under all (combinations of) actions expected during service life.

Service Life The period in which the required performance of a structure or structural element is achieved, when it is used for its intended purpose and under expected condition of use.

Specifications An explicit set of instructions related either to material, product, or service, given by the specifier to enable the producer and/or contractor to carry out the work in accordance with the intent of the design.

Standards Documents that govern and guide the various aspects involved in the construction industry. They describe technical requirements for materials, procedures, equipments or test methods required to achieve the intended functions.

LIST OF ACRONYMS

AAR	→ Alkali-Aggregate Reaction
ACI	→ American Concrete Institute
ACRR	→ Alkali-Carbonate Rock Reaction
AS	→ Australian Standard
ASR	→ Alkali-Silica Reaction
ASTM	→ American Society for Testing Materials
BRE	→ Building Research Establishment
CCI	→ Chloride Conductivity Index
CSA	→ Canadian Standards Association
CSF	→ Condensed Silica Fume
DEF	→ Delayed Ettringite Formation
DI	→ Durability Index
DSL	→ Design Service Life
EN	→ European Standard
FA	→ Fly Ash
GGBS	→ Ground Granulated Blastfurnace Slag
HPC	→ High Performance Concrete
IS	→ Indian Standard
PBS	→ Performance-Based Specifications
OPI	→ Oxygen Permeability Index
PTMs	→ Performance Test Methods
RC	→ Reinforced Concrete
RCPT	→ Rapid Chloride Permeability Test
SANS	→ South African National Standards
SANRAL	→ South African National Road Agency Limited
SCMs	→ Supplementary Cementitious Materials
SLMs	→ Service Life Models
W/B	→ Water-to- binder ratio
W/CM	→ Water-to-cementitious material ratio
WSI	→ Water Sorptivity Index

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Concrete is versatile, economical, strong and durable, thus making it an ideal construction material for most infrastructure. Unlike timber and ordinary steel, concrete dominates most types of infrastructure including buildings, long-span bridges, bored tunnels, and concrete pressure vessels. Its performance is vital for underpinning the nation's essential services and economic activities. Despite the remarkable work of concrete in terms of performance, it is now facing additional challenges with respect to durability.

The term 'durability' has multiple definitions, but they all mean much the same. According to SANS 10100 (2009), durability is the ability of a structure or material to withstand the environmental conditions without significant deterioration. Although in majority of cases RC structures satisfy durability requirements, there are few cases where they fail, as a result of premature deterioration. Recently, the world has experience severe durability problems mainly due to premature deterioration in much important infrastructure that requires extensive repair and maintenance. For instance, in the USA, annual costs of repair and replacement of bridges of up to about US$ 8.3 billion has been estimated (Yunovich et al., 2001). In Western Europe alone, annual costs of US$ 5 billion for repair of RC structures were estimated as long ago as – 1998 (Knudsen et al., 1998). In the Arabian Gulf, repair and replacement costs of about US$ 798 million due to extensive deterioration resulting from corrosion of reinforcing steel have been reported (Al-Bahar et al., 2003). Similarly, extensive costs of repair and maintenance of RC structures related to premature deterioration have been reported in several publications (Song and Shayan, 1998; Gjørv, 2009), that may illustrate the seriousness of the durability problems.

The means and methods for preventing premature deterioration are well-established in the numerous literature. Nevertheless, there are cases where some RC structures still do not correspond with the required durability because of several reasons. As identified by Neville (1987), poor understanding of material properties that results in inadequate materials selection, environmental actions, inadequate acceptance criteria of *in situ* concrete and inappropriate construction methods, are the main reasons. Many of these reasons are partly related with the requirements specified in the current design standards and specifications for durability purposes.

The prescriptive approach that outlines requirements for material compositions and proportions, procedures, and test methods, are commonly used in most design standards and specifications for durability purposes. Though such approach may encompass requirements for, inter alia, minimum compressive strength, maximum water-to-cementitious material (w/cm) ratio and cover depth to the reinforcing steel, the desired concrete performance is not necessarily described (Lobo et al., 2005; Bickley et al., 2006a; Bickley et al., 2006b). Material and construction variability are not taken into account, and even if intensive supervision is carried out, it is difficult to ensure all specified parameters are achieved (Day, 2005). In addition, while several requirements such as maximum w/cm and minimum water content may conflict with the intended performance, they are impractical or costly to measure or verify in practice (Torrent, 2006; Alexander et al., 2010). Generally, it should be acknowledged that this approach is based on lab data and past experience (Clifton, 1993; Folic, 2009), which cannot tell how long a structure will perform in service under selected exposure condition. Thus, the design service life (DSL) of the structure is unknown and the safety is also uncertain. To counter for uncertainty and ensure adequate service life, a more rational approach based on performance is imperative.

In an attempt to move away from the prescriptive specification approach for durability, research has focused on the approaches aimed at performance. As defined by the United States National Ready Mixed Concrete Association (NRMCA), performance specification is *'a set of clear, measurable and enforceable instructions that outlines the functional requirements for hardened concrete depending on the application'*. Performance approach measures relevant properties of the concrete, in particular transport-related properties that account for durability (Beushausen and Alexander, 2006a; Alexander et al., 2010). This approach can be categorised into performance-based approach and an intermediate approach termed 'hybrid'. Performance-based is an integrated approach that links durability specifications, including durability indicators from relevant test methods, and durability design through Service Life Models (SLMs) in order to estimate service life of the structure (Beushausen and Alexander, 2006a; Alexander et al., 2010; Alexander and Santhanam, 2012). In contrast, the 'hybrid' approach relies on durability indicators chosen based on technical recommendations without explicitly given Design Service Life (DSL). In this approach, the client and/or specifier could decide on the desired level of performance in a certain exposure condition and propose relevant durability indicators (Taylor, 2004). The producer and/or contractor could develop the desired concrete that satisfies the durability indicator limits set forth by the client and/or specifier. Generally, both approaches are aimed at achieving relevant durability indicators that demonstrate the suitability of the concrete and its compositions with respect to the exposure conditions. Performance approaches impose few or no restrictions on the concrete compositions, proportioning, or construction methods, but rather promote innovations (Bickley et al., 2006a; Carino et al., 2010; Chrzanowski, 2011).

Worldwide there is a consensus that in order to extend the service life of RC structure, performance approaches are imperative. However, performance approaches have been implemented only in few design standards because of several existing challenges as identified by Carino et al. (2010). The challenges include (a) lack of reliable, consistent and standardized test procedures which can evaluate concrete performance in time or routinely, (b) lack of adequate SLMs which can capture the main aspects involved in deterioration including environmental-specific factors, and (c) lack of experience in developing sufficient performance requirements with appropriate acceptance criteria.

1.1.1 A Critique on the Prescriptive Specifications in the Design Standards

In most design standards, emphasis is given on safety and serviceability requirements as the primary concerns while less attention is given to durability. Parameters such as maximum water-to-cementitious material (w/cm) ratio, minimum compressive strength and minimum cover depth with respect to exposure conditions are commonly specified. Though such parameters are likely to be sufficient for common structures, it is different for special structures requiring longer service life, particularly in aggressive environments. Design standards such as ACI 318 (2008) and AS 3600 (2001) acknowledge that the durability requirements presented are insufficient to ensure durable RC structures. They recommend a more stringent approach to be applied when longer design life is required in a specific environment. There are several shortcomings with the current requirements in most reviewed design standards as shortly described below.

Firstly, the requirements do not define the material limit states or deterioration mechanisms under specific exposure conditions throughout the service life of the structure. The limit state in this context is the border line that separates the desired states from the undesired states to which a structure is exposed during its service life (Edvardsen, 1999). Limit states considered are the Ultimate Limit State (ULS) that concerns the safety of the structure, and Serviceability Limit State (SLS) that concerns the functionality or aesthetics of the structure.

Secondly, strength is considered as an indicator of durability postulating that "a stronger concrete is more durable" and therefore specifying higher strength results to durable structures (Kwan and Wong, 2005). However, it is well-noted that strength does not account for materials and construction variability that greatly influence penetrability of deleterious substances (Ballim et al., 2009). In addition, while w/cm may indicate a good relation to penetrability and hence durability, there is no fixed relationship when different SCMs are used, in particular GGBS and FA (Obla et al., 2005). It is therefore less meaningful to rely on specified w/cm and strength requirements for evaluating concrete performance.

Thirdly, there is no quick and reliable method of verifying specified maximum w/cm or minimum cement content in practice (Torrent, 2006). This is interesting and may raise multiple questions with limited answers. For instance, *to what extent will the concrete producer ensure durability under this situation?*, the answer may be simple; the concrete producer will have to use own judgement based on field observation and past experience or simply rely on lab specimens, which bear little resemblance to the actual structure though made of the same material (Bungey et al., 2006).

Broadly, specifiers and/or designers rely on the specified parameters given in the design standards for safety, serviceability, and durability without adequate knowledge on how concrete interact with its environment. However, it is argued that there are better ways of specifying concrete for performance (Obla et al., 2005), e.g. by specifying penetrability limits in lieu of w/cm or strength. The questions remains, *how can these properties be specified and incorporated into the design standards?*; *what should be the acceptance criteria?*; *What will be the consequences of non-conformity?*. Certain design standards such as CSA A23.1/23.2 (2009) and AS 1379 (2007) may provide answers to some of these questions, where several properties reflecting performance have been incorporated.

1.1.2 International Effort on Performance Specifications for Durability

The demand for durable concrete which should incorporate several composite materials is greater and may not be achieved through compliance with the strictly prescriptive specifications. Scarcity of raw materials and increasing environmental concerns are the driving forces that influence movement from prescriptive to performance specifications.

Recently, performance specifications have attracted interest in Australia, Canada, USA, South Africa and many countries in Europe as shortly described below.

In 2002, the NRMCA who represents the ready-mixed concrete industry in USA developed an initiative called P2P (*i.e.* Prescription to Performance) that promote performance specifications. One of its goals has been to obtain technical data that demonstrate the benefits of performance specifications that could be used to support changes in the design standards. Bickley et al. (2006b) and (2008) produced two reports, i.e. "Preparation of performance-based specification for cast-in-place concrete" and "Guide to specifying concrete performance" respectively. These reports give both an overview of performance-based criteria for concrete and recommends alternative performance specifications in ACI 318. Carino et al. (2010) published a report named ACI ITG-8R-10 "Report on performance-based requirements for concrete", focused on the means to develop and implement performance-based alternatives in the design standards. In the report, essential elements of performance specifications including quality characteristics, test methods, and acceptance criteria are discussed. Moreover, alternative performance requirements are proposed for the prescriptive durability requirements in ACI 318.

In Europe, the Performance Based Building Network (PeBBu) project which involved more than 70 international organisations has been developed (Becker and Foliente, 2005). It is a thematic network funded under the European Commission's "EU" 5th framework – Competitive and Sustainable Growth. The PeBBU has been facilitating and enhancing existing performance based building (PBB) research activities by networking with the main European stakeholders and other international stakeholders. The PBB concepts have been adopted in some regulatory frameworks among some CEN members such as the European New Approach and the accompanying Construction Product Directive, and the Building Code of Australia and New-Zealand. A brief detail of this concept is described in the subsequent chapters.

In South Africa, Durability Index (DI) tests that characterize the potential durability of concrete depending on the relevant transport mechanism through the cover depth have been developed (Alexander et al., 1999). These tests include oxygen permeability for permeation, water sorptivity for absorption, and chloride conductivity for diffusion. Developments in DI tests may form an integral approach that link DI values and SLMs and therefore reflecting advances in PBS for concrete durability in South African concrete industry.

1.2 RESEARCH SIGNIFICANCE

Recent research indicated that inadequate attention to durability specifications is the critical cause of premature deterioration in RC structures. Though sufficient knowledge with respect to most deterioration mechanisms is currently available, some RC structures still do not correspond to the required durability. There are several reasons but the mains ones are partly related to the current design standards that focus on prescriptive rather than performance. Parameters such as w/cm and strength commonly specified only provide indirect indication to penetrability, which is significant to durability. Therefore measurements beyond these parameters that have a direct influence on in-place performance of concrete should be employed. Such measurements e.g. penetrability limits are now gained acceptance in the construction industry that influence their incorporation into the design standards so as to minimize potential conflicts currently exist.

In recent years, many upgrades and improvements have been made in several design standards to include performance alternatives for concrete durability. Despite these advances, many design standards including SANS which provides the basic principle for concrete production and construction, are still behind in many respect related to durability. In this context, fundamental changes that incorporate performance specifications are imperative in order to bring SANS up-to-date following international trends. Inclusion of performance specifications into SANS is an important step for solving many durability problems that threaten the performance of RC structures in South Africa and the world in general.

1.3 RESEARCH OBJECTIVES

The primary objective of the study is to examine and compare various approaches to performance specifications for concrete durability developed worldwide. Part of the study will also look into various performance test methods and the acceptance criteria for durability parameters developed. The aim is to look at a holistic framework in order to develop performance specifications for local application following international trends. Guidelines to develop performance specifications including suggestions to include performance alternatives in SANS 10100-2 will be presented. In order to achieve the objectives stated, the following aims were identified:

1. Perform an in-depth review of various design standards to identify where prescriptive requirements are incorporated and how they reflect durability specifications. The

focus is given to the following design standards, ACI 318 (2008), AS 3600 (2001) and AS 1379 (2007), CSA A23.1/23.2 (2009), EN 206-1 (2000), IS 456 (2000), and SANS 10100-2 (2009).

2. Perform a review on the efforts made to include performance alternatives in the design standards or in project specifications. The focus is limited to efforts in Australia, Canada, USA, South Africa, India and the European Union (EU) countries. The study also undertakes a survey of the performance test methods developed in these countries that can be used to evaluate performance. The study aim at proposing appropriate test method that is linked to penetrability requirements as a measure of performance.

3. Propose the way forward for South African specifications, in particular SANS 10100-2, to replace predominantly prescriptive provisions for durability with performance alternatives. The focus is limited to the durability requirements that relate to penetrability. The study also aimed at proposing future recommendations in moving closer to the ultimate goal of developing a durability standard that would be analogous to SANS 10100-2.

1.4 SCOPE AND LIMITATION

This dissertation focused on performance specifications for concrete durability of RC structures. Since most performance requirements are based on the transport mode relevant to the specific exposure condition, various transport mechanisms and accompanied durability problems are discussed. A brief description of the relevant Performance Test Methods (PTMs), their strengths and limitations, and acceptance criteria for evaluating concrete performance are included. Service Life Models (SLMs) are also discussed with respect to their strengths and limitations in estimating Design Service Life (DSL) of RC structure under specific exposure conditions.

Material-related durability problems such as Alkali-Silica Reaction (ASR), acid attack, sulfate attack, freeze/thaw attack, and chemical attack, are also considered in this dissertation. Although a brief discussion with respect to their mechanisms and the preventive measures are provided, no further details presented regarding their performance requirements, except for freeze/thaw and sulfate attacks. Many of these attacks may be dealt with effectively by controlling material compositions and proportions using prescriptive requirements. Material-related durability problems such as shrinkage, creep, abrasion, and fire resistance are beyond the scope of this dissertation and therefore not discussed.

This dissertation focused on materials and construction methods requirements for achieving durability requirements. It does not cover protective measures including non-reactive materials e.g. stainless steel or non-metallic reinforcement, special surface treatment e.g. coatings to the reinforcement or concrete, cathodic prevention, etc. that required for avoidance of deterioration.

1.5 DISSERTATION LAYOUT

Chapter 1: Gives an overview of the various approaches to durability specifications for RC structures. It stress that prescriptive specifications currently used in most design standards including SANS 10100-2, have limited application with respect to long-term performance. Performance specifications tough considered as an alternative is also described with respect to challenges facing its full application. The dissertation objective is then stated as the need to examine and compare various approaches to performance specifications for concrete durability developed worldwide. The dissertation significance is also clarified as the need to support changes in SANS 10100-2 to include performance alternatives following the international trends.

Chapter 2: Describes various transport mechanisms that influence durability of RC structures. An in-depth review of various mechanisms of deterioration and the recommended preventive measures as presented in the design standards are discussed. The consequences of concrete deterioration to the performance of RC structures are described.

Chapter 3: Conducts an in-depth review on the durability provisions in AS 3600 (2001) and AS 1379 (2007), ACI 318 (2008), CSA A23.1/23.2 (2009), EN 206-1 (2000), IS 456 (2000), and SANS 10100-2 (2009). The focus is limited to the environmental actions, limiting values for concrete properties and proportions, and cover depth requirements. General comments related to these parameters from each design standard are presented. A critique comparison of relevant material properties and proportions from each design standard that are provided to resist corrosion of reinforcing steel are presented based on exposure classes defined in EN 206-1.

Chapter 4: Describes the state-of-the-art of performance specifications for concrete durability. It describes efforts made in Australia, Canada, USA, Europe, India, and South

Africa towards implementing performance specifications either in the design standards or in project specifications.

Chapter 5**:** Conducts an in-depth review on PTMs incorporated in the design standards or in project specifications. Their strengths and limitations in evaluating concrete performance are discussed. SLMs for estimating DSL of RC structures are discussed. Challenges confronting PBS, i.e. an integrated approach that links durability specifications, including durability indicators from relevant PTMs, and durability design through SLMs are also described.

Chapter 6**:** Presents both hypothetical and practical case studies that demonstrate application of performance specifications. It describes the application of ASTM C1202 and the South African DI test methods in practical cases.

Chapter 7**:** Describes action plan required to improve SANS 10100-2 following international trends. The way forwards for South Africa towards performance specifications, particularly with the implementation of DI test methods in SANS 10100-2 is described. Guideline for developing durability specifications in the South African concrete industry are presented, with respect to roles and responsibilities of each stakeholder.

Chapter 8**:** Concludes the dissertation and suggests future recommendations in moving closer to the ultimate goal of developing a durability standard in South Africa.

CHAPTER 2: CONCRETE DETERIORATION AND DURABILITY

2.1 Introduction

Generally, concrete is a very versatile construction material with its own special properties including strength and durability. Apart from being cost-effective construction material, it offers a wide range of opportunities in terms of structural type and form and surface aesthetics. Careful considerations with respect to materials selection, exposure conditions, design and construction specifications, are required in order to achieve the intended properties. Failure to achieve the required properties may result in aesthetic, functional, and/or structural problems.

Concrete construction is the largest consumer of natural resources such as water, sand and gravel (Gjørv and Sakai, 2000). The ninth edition Global Cement Report published in 2011 found that 3.294 billion tonnes of cement were consumed in 2010 and the consumption is expected to reach 3.859 billion tonnes by the end of 2012. In terms of natural aggregates, Langer et al. (2004) estimated that about 16.5 billion tonnes are produced annually worldwide. It is clear that there is extensive exploitation of natural resources accompanied by environmental pollution, which is partly related to concrete deterioration.

Currently, an understanding on the concrete microstructure and the process of deteriorations has greatly improved (Sarja and Vesikari, 1996). Sufficient knowledge with respect to most deterioration mechanisms identified the following parameters that frequently affect durability of RC structures.

(i) Durability specifications: Material properties and proportions and cover depth.

(ii) Construction specifications: Mixing, transporting, compacting, finishing, and curing conditions.

(iii) Environmental actions: AAR, freeze/thaw attack, sulfate attack, carbonation and chloride ingress.

(iv) Quality control: Compressive strength and penetrability.

Several of these parameters are commonly specified in the design standards for durability purposes. However, many of them, particularly those related to transport properties, are difficult and may not always be achievable (Torrent, 2006; Alexander et al., 2010). With the exception of mechanical damage that influence cracks and allow easily penetrability, most types of deterioration in RC structures are related to the transport mechanisms through the cover concrete (Kropp and Alexander, 2007). In the following section, various transport mechanisms that contribute to the deterioration of RC structures are discussed.

2.2 Transport Mechanisms in Concrete

Concrete durability is primarily dependant on the penetrability of deleterious substances through the cover concrete and results in degradation of either concrete or reinforcing steel (Ballim et al., 2009; Oslakovic et al., 2010). Thus, quality of cover concrete determines the degree of penetrability, where high degree results from poor concrete quality or insufficient cover depth (Alexander et al., 2010). The degree of penetrability may also be affected by premature loading, overloads or settlement of the structure that may cause cracks and allow penetration.

Transport mechanisms in concrete are of several types such as permeation, diffusion, absorption and migration. Permeation and diffusion are the major transport mechanisms that influence the durability of the concrete (Kwan and Wong, 2005). Though transport mechanisms may occur in combinations, it is imperative to understand the principles behind individual transport mechanism. This may give proper guidelines when determining the appropriate methods for quantifying the resistance of concrete. In the following sections, the individual transport mechanisms are discussed.

2.2.1 Permeability

Permeability is defined as the ease with which a fluid under pressure can flow through a porous concrete (Mehta and Monteiro, 2006). In concrete, the mechanism depends on several factors including the concrete microstructure, the moisture condition of the materials, characteristics of the permeating fluid and the thickness of the concrete member. For a steady-state flow, the rate of transport, commonly referred as coefficient of permeability (K), is determined from Darcy's expression as shown in equation 2.1.

$$\frac{dq}{dt} = K\left(\frac{\Delta HA}{L\mu}\right) \tag{2.1}$$

Where $\frac{dq}{dt}$ = rate of fluid flow

K = coefficient of permeability (determined directly by tests)

μ = viscosity of fluid

ΔH = pressure difference

A = surface area

L = thickness of the solid

Permeability is influenced by material compositions and proportions and construction methods including compaction and curing methods. This mechanism is common for structures in contact with liquid under a pressure head such as water-retaining structures.

2.2.2 Diffusion

Diffusion is the process by which liquids, gases or ions penetrate into a porous medium under the influence of a concentration gradient (Ballim et al., 2009). As in the case of permeability, diffusion depends on the size, porosity and the connectivity of the pores. Chloride transport into concrete is mainly governed by diffusion mechanisms. Therefore mathematical simulation for chloride ingress is normally based on Fick's laws of diffusion that determine the diffusion rate into concrete. Fick's first law shown in equation 2.2 is used to determine the rate of chloride ingress by means of ionic flow (J). The negative prefix denotes that the flux occurs along a negative concentration gradient. The effective diffusion coefficient, surface chloride concentration and chloride penetration depth are determined experimentally.

$$J = -D_{eff}\frac{dC}{dx} \qquad [2.2]$$

Where

J = mass transport rate (g/m^2s)

D_{eff} = effective diffusion coefficient (m^2/s)

$\frac{dC}{dx}$ = concentration gradient ($g/m^3/m$)

c = chloride concentration (g/m^3)

x = distance (m)

Fick's first law is not used so often since it does not account for time variation. Fick's second law shown in equation 2.3 is commonly employed as it accounts for time variation. It is based on the validity of several assumptions such as the concrete is homogenous, inert and that diffusion properties do not change with time or with concentration of diffusing ion (Tang, 1996). Nevertheless, concrete is heterogeneous, both physical and chemical reactions occur, and the diffusion properties change with time and with the concentration of the diffusing fluid (Song and Shayan, 2000; Stephen et al., 2010).

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial^2 x} \qquad [2.3]$$

Using Fick's second law, time-dependent chloride profile can be predicted when the diffusion coefficient and the chloride surface concentration are known. Diffusion and permeability are inter-related as they are both influenced by similar geometric factors (Kwan and Wong, 2005). However, diffusion is very sensitive to the relative humidity or the degree of saturation of the pores when compared to permeability.

2.2.3 Migration

Migration is the process by which ions in electrolytes move under the action of an electrical field (Ballim et al., 2009). Migration is determined by laboratory-accelerated test methods, and is performed on the basis of Nernst-Planck equation given below (Kropp and Alexander, 2007).

$$q_i = -D_i\left(\frac{\partial c_i}{\partial x} + \frac{z_i F}{RT} c_i \frac{\partial \Phi}{\partial x}\right) + cv \qquad [2.4]$$

Where
q = mass flux (g/m^2s)
D = diffusion coefficient (m^2/s)
c = concentration (g/m^3)
v = velocity of capillary flow (m/s)
z = electrical charge
F = Faraday's constant (9.6548×10^4 J/Vmol)
T = absolute temperature (K)
Φ = electrical potential (V)
x = distance variable
R = gas constant (8.314 J/mol K)

The Nernst-Planck equation describes the mass flow due to diffusion, migration and convection. Thus, the total flux is the sum of diffusion, migration and convection. Migration effect is very significant for concrete structures that are subjected to stray current interference, galvanic corrosion effect, or under cathodic protection in the field.

2.2.4 Absorption

Absorption is the process by which liquids move through the porous concrete matrix under capillary suction (Bamforth, 1990). The capillary suction is dependant on the pore geometry i.e. interconnectivity, and the degree of saturation of concrete. The highest concentration of liquids occurs within the outer cover concrete and decreases with the depth of the cover concrete (Ballim et al., 2009). The rate at which liquids enters the pores is termed as sorptivity, which can be used to characterize the ability of a concrete to absorb and transmit water by capillary suction. According to Soshiroda and Voraputhaporn (1999), sorptivity can be calculated as shown in the following equation.

$$S = \frac{\Delta M_t}{\sqrt{t}}\left[\frac{1}{AZ}\right] \qquad [2.5]$$

$$Z = M_{sat} - M_0/AL$$

Where
- S = sorptivity (mm/$\sqrt{\text{min}}$)
- Z = effective porosity
- A = specimen cross sectional area
- M_{sat} = mass of the specimen at saturation
- M_o = dry mass of the specimen
- $\frac{\Delta M_t}{\sqrt{t}}$ = slope of straight line produced when mass of water absorbed is plotted against the square root of time

Absorption is an important transport mechanism with respect to the ingress of chlorides into semi-saturated concrete. Chlorides penetrate into the concrete much faster by capillary forces than by diffusion alone. For RC structures exposed to sea water or in dry conditions, the ingress of chloride is progressive through the capillary pores.

2.2.5 Wick action

Wick action is the transport of water and ions from saturated face of a concrete element to a drying face (Buenfeld et al., 1997). It can be considered as a combination of water absorption and water vapour diffusion and is therefore characterized by sorptivity and vapour diffusion coefficients respectively. In this situation, species dissolved in the water are transported to the dry zone of a concrete element and may result in salt crystallization as schematically shown in Figure 2.1.

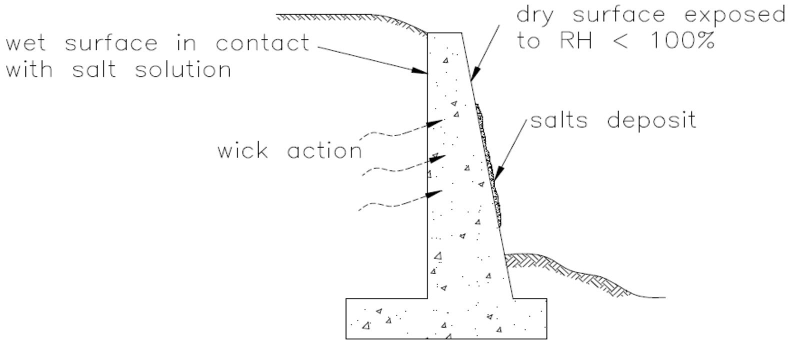

Figure 2.1: A schematic diagram of wick action in concrete (Puyate and Lawrence, 1999)

An often cited example is a sea wall shown in Figure 2.2, which may illustrate various transport mechanisms in concrete. The zone below tidal mark, where concrete is fully submerged chloride-ions diffuses. In the tidal zone, where alternate wetting and drying occurs as a result of changes in water levels, chloride ingress occurs due to combinations of diffusion and capillary absorption. Above the tidal zone, where waves irregularly splash, absorption due to capillary suction predominates but also carbonation may occur. And behind the sea wall, brackish groundwater may permeate the concrete driven by hydrostatic pressure.

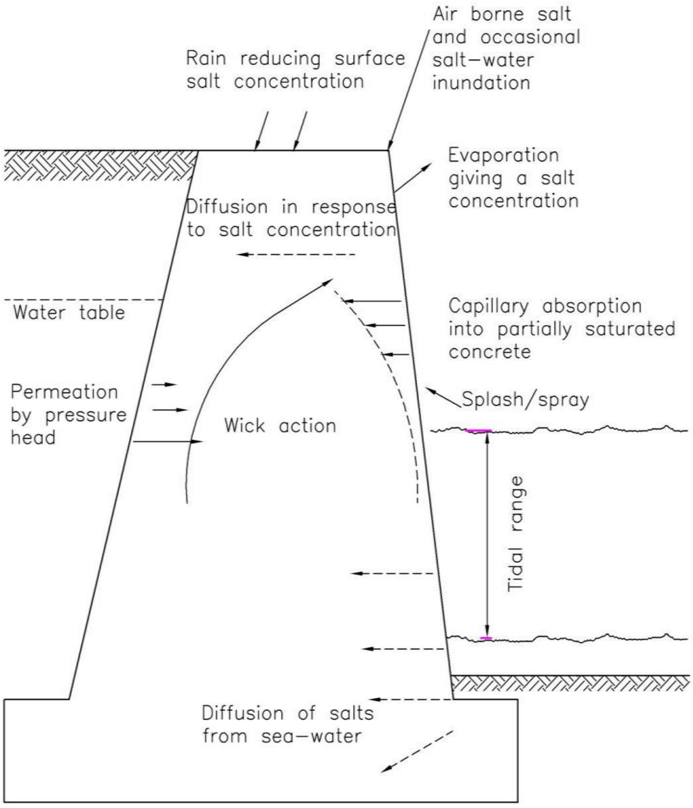

Figure 2.2: Various transport mechanisms in concrete sea wall (*adopted from* CS 109, 1996)

2.3 Modes of Concrete Deterioration

In practice, deterioration of RC structures is rarely due to a single cause (Mehta and Monteiro, 2006). It is a combination of actions (*i.e.* internal and external) determined by concrete as a system and its environment. Internal actions are those associated with the concrete system, which determines its ability to resist deterioration. They mostly depend on the quality of concrete (*i.e.* material compositions, proportioning, and construction methods). On the other hand, external actions are largely grouped as physical or chemical attacks, which influence the degree of deleterious substances that the concrete has to withstand.

Deterioration mechanisms can occur in several forms with different significance. This can be as shown in Figure 2.3, where a holistic model of concrete deterioration from commonly encountered environmental actions is presented.

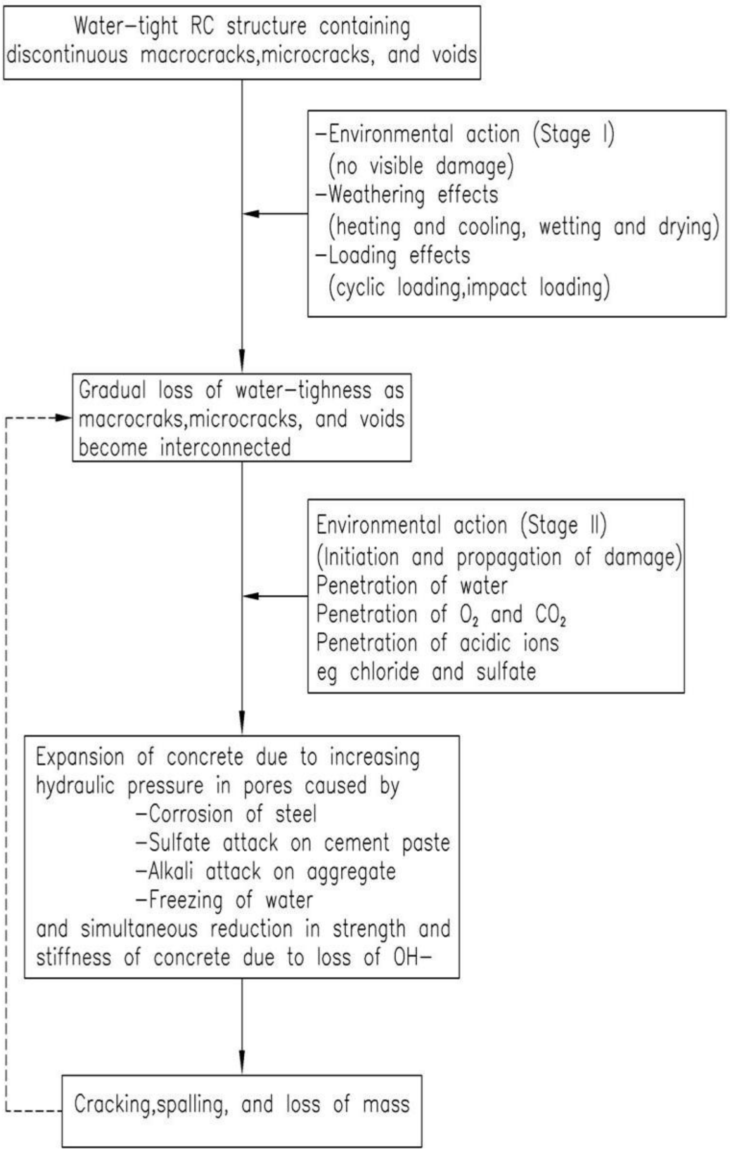

Figure 2.3: A holistic model of deterioration of RC (Mehta and Monteiro, 2006)

According to Figure 2.3, concrete is initially water-tight and it may remain that way as long as there is no interconnectivity between the pores or formation of microcracks reaching the surface of concrete. However, this is not often the case and therefore concrete loses its water-tightness as a result of environmental actions or structural loading effects which in turn allow penetrability. This marks the beginning of Stage II, where deterioration of concrete takes place and leads to visual effects such as cracking, spalling of the cover and loss of mass.

A flow chart is presented in Figure 2.4 illustrating how various modes of deterioration will be reviewed, and significant factors affecting durability of RC structures.

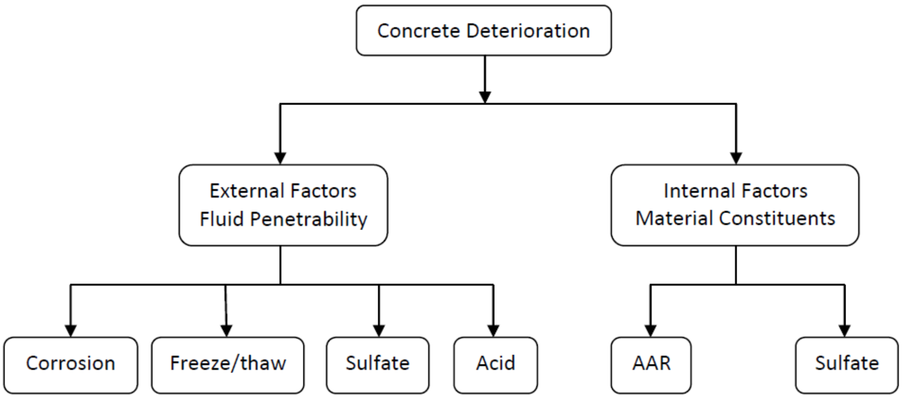

Figure 2.4: Flow chart illustrating review mode of various factors affecting durability

2.3.1 External Factors

Generally, aggressive external factors are related to environmental actions which influence concrete durability. Various external factors such as oxygen, moisture, carbonation, chloride ions, and sulfates are described in the following sections in relation to their deterioration mechanisms.

2.3.1.1 Corrosion of reinforcing steel

Reinforcement corrosion is the major cause of deterioration in RC structures and is probably the main durability problem threatening its performance at present (Gjørv, 2009). Research which focused mainly on concrete penetrability have shown two main causes that results in corrosion of reinforcing steel (Mackechnie, 1996; Glass and Buenfeld, 1997; Schiessl, 1998). These are carbonation and chloride ions, where their effects are initiated after a carbonation front or chlorides penetrate the cover concrete and reach the first layer of reinforcing steel.

Corrosion may result in different forms of deterioration such as cracking, spalling and reduction of the steel cross-section, referred as general corrosion caused by carbonation. Corrosion may also result in pitting commonly referred as localized corrosion caused by chloride ions (Folic and Zenunovic, 2010). Heckroodt (2002) presented three stages whereby corrosion of reinforcing steel develops leading to deterioration of RC structures. These are illustrated in Figure 2.5 indicating different dominating processes i.e. transportation of chloride ion or carbon dioxide; corrosion under relatively active conditions in un-cracked concrete; and corrosion of reinforcement in cracked concrete respectively.

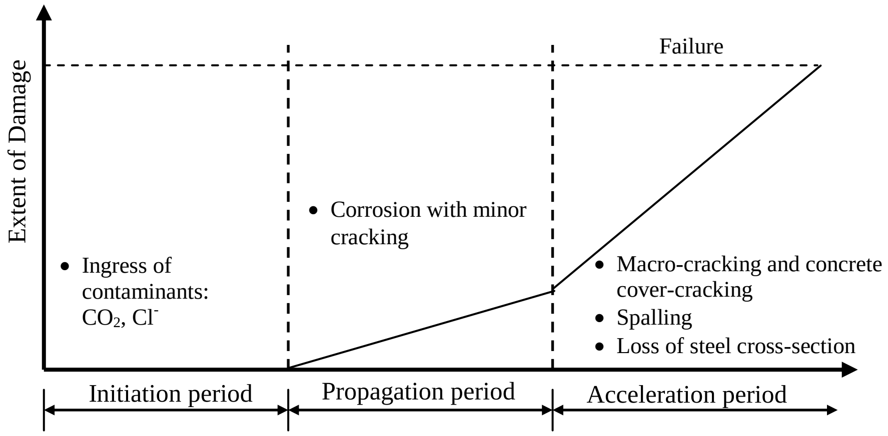

Figure 2.5: Three-stage model of corrosion damage (Heckroodt, 2002)

(i) *Initiation period*: The rate of penetrability of chloride ion and carbon dioxide is very low and hence low rate of corrosion of reinforcing steel. In this stage little deterioration occurs.

(ii) *Propagation period*: This may be referred as the end of initiation period where corrosion of reinforcing steel starts, which generates expansive corrosion products causing cracking, delamination and spalling of the cover concrete.

(iii) *Acceleration period*: Corrosion rate increases due to easily ingress of chloride ion and carbon dioxide as a result of further cracking and spalling.

2.3.1.1.1 Mechanisms of corrosion of steel

Concrete with pH ranging from 12 to 13 at the concrete-steel interface promotes formation of a thin protective layer on the surface of steel known as a depassivating layer (Richardson, 2002; Rostam, 2006). The cover concrete which prevents the ingress of deleterious substances is not always perfect due to presence of pores and microcracks in the concrete matrix that allow penetrability. As a result, pH value of concrete may drop to below 10.5 resulting in the breakdown of the depassivating layer and lead to subsequent corrosion of reinforcing steel.

Corrosion is generally considered as an electro-chemical process, which occurs when two dissimilar metals come into contact under the influence of moisture and oxygen. Similarly, corrosion may occur due to the differences in electro-chemical potential on the surface of the steel alone, which forms anodic and cathodic areas connected by electrolyte in form of the salt solution in the hydrated cement (Liu, 1996; Ballim et al., 2009). The process of reinforcement corrosion can be described according to Figure 2.6 below.

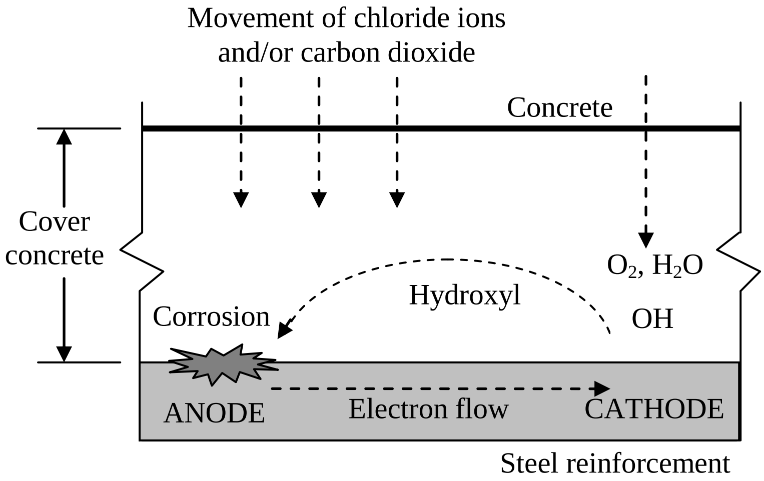

Figure 2.6: Corrosion process of reinforcing steel in concrete (Mackechnie, 2001)

For reinforcing steel to corrode, the following pre-conditions are necessary:

(i) Depassivation of reinforcing steel must take place creating a location for an anodic process to develop.

(ii) Depassivated area must be in metallic contact with neighbouring or nearby area of the reinforcing steel which is passivated in order for the cathodic process to take place.

(iii) The anodic and cathodic area must also be connected by an electrolyte in order for ions to move between the two areas.

(iv) There must be sufficient moisture in the concrete surrounding the reinforcing steel in order for ions to migrate between the cathode and anode.

(v) There must be a presence of oxygen, in particular at the cathode, where a corrosion electrical circuit forms as a result of electrical current flowing from the anode to cathode through the steel. In this case, hydroxyl ions will form as a product of corrosion at the cathode.

2.3.1.1.2 Initiation of chloride-induced corrosion

Chloride ingress into concrete is the most critical environmental action for RC structures in marine and de-icing salts environments. Chloride may penetrate into concrete either through diffusion or capillary suction. Chloride penetration within the surface of the outer cover depth is initially governed by capillary suction while chloride penetration to a deeper depth is mainly governed by long-term diffusion (Bamforth, 1990).

Chloride ingress is a very complicated process and not yet fully understood as it is governed by combinations of several mechanisms (Gjørv, 2009). The depassivation mechanism is different when compared to carbonation. For instance, chloride attack on the passive layer does not result in pH drop. Chloride acts as a catalyst to corrosion when there is sufficient concentration at the steel surface. Though chlorides are not consumed in the process, they help to break down the passive layer and allow corrosion process to proceed quickly in the form of localized pitting (Broomfield, 1997). This is schematically illustrated in Figure 2.7.

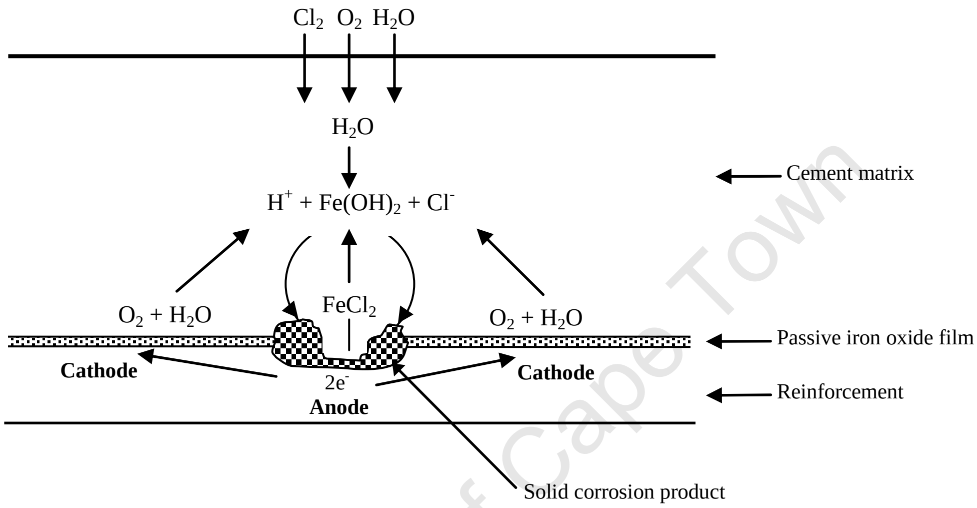

Figure 2.7: Corrosion of reinforcement in concrete due to chloride ions (Broomfield, 1997)

Thus, because of the acidic environment in the pit once it has formed, the pit remains active and increases in depth.

As suggested by Stanbury and Buchaman (2000), the following cathodic reactions occur in the presence of chlorides.

$$Fe^{2+} + 2Cl^- \rightarrow FeCl_2 \text{ (ferrous chloride)} \quad [2.1]$$

$$FeCl_2 + 2H_2O \rightarrow Fe(OH)_2 + 2HCl \quad [2.2]$$

Depending on the quality of concrete and the cover depth, chloride penetration may take years before reaching the reinforcing steel. However, when chlorides reach the reinforcing steel and the corrosion process starts, it may take only few years for the visual damage in the form of cracks and rust staining to be observed.

Both physical and chemical protection determines the degree of chloride-induced corrosion. Physical protection is generally afforded by cover depth which acts as a barrier to the ingress of chlorides. Chemical protection is provided by high alkaline pore solution of the concrete due to presence of sodium hydroxide, potassium hydroxide and calcium hydroxide produced in the hydration reactions of cement components (Broomfield, 1997). Generally, quality concrete of appropriate mixture compositions and proportions, adequate cover depth and proper curing practices provides an excellent protective environment against chloride-induced corrosion.

2.3.1.1.3 Initiation of carbonation-induced corrosion

Though carbonation is considered less aggressive than chloride, it is also severe as it reduces the ductility of the concrete depending on the degree of the ingress (Chang and Chen, 2005). Corrosion of reinforcing steel often increases when carbonation couple with the chloride ion since carbonation accelerates the rate of chloride ingress (Roziere et al., 2009). As described by Papadakis et al. (1991a and 1991b) in the following reactions, carbonation results from a chemical reaction between carbon dioxide and concrete hydrates.

Firstly, the atmospheric carbon dioxide (CO_2) reacts with water (H_2O) to form carbonic acid;

$$CO_2(\text{pore solution}) + H_2O \rightarrow H_2CO_3 \qquad [2.3]$$

Secondly, carbonic acid (H_2CO_3) reacts with calcium hydroxide ($Ca(OH)_2$) as follows;

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O \qquad [2.4]$$

Carbonic acid reduces the pH of pore solution in the concrete which was initially between 12-13 to about 8-9, the level at which the passive film is unstable and therefore corrosion occurs (Broomfield, 1997). At a relative humidity between 50-70% carbonation is mostly likely to occur (Neville, 1995), and the carbonation depth increases in proportional to the square root of time as shown in equation 2.5 below.

$$d_c = C\sqrt{t} \qquad [2.5]$$

where d_c= depth of carbonation, C = carbonation coefficient, and t = time of exposure

Carbonation occurs as a result of poor concrete quality which is a function of material compositions and proportions and construction methods. Moreover, carbon dioxide concentration and moisture content of the concrete may also contribute to the rate of carbonation (Newman and Choo, 2003). In this way, quality concrete incorporating SCMs and proper construction methods are necessary measures required to encounter carbonation effects.

2.3.1.1.4 Corrosion of steel in cracked concrete

Cracks developed in concrete structures resulting from load-induced stresses may allow easily penetrability when compared to un-cracked concrete. The transport mechanisms of either carbonation or chloride are also different when compared to un-cracked concrete. It is reasonable to assume that as crack width increases penetrability of deleterious substances also increases and subsequently increases the rate of corrosion (Rostam, 2006; Gjørv, 2009). Although it has been difficult to establish the relationship between crack width and the rate of corrosion, research indicated that crack orientation and exposure conditions enhance the rate of corrosion. Depending on exposure conditions it appears that the rate of corrosion is reduced over time by clogging of the crack caused by the precipitation of calcium carbonate which prevents further penetrations (Folic and Zenunovic, 2010). It is therefore, important to consider the means of reducing or eliminating cracks in the concrete especially when the structure has to face aggressive environments.

2.3.1.2 Freezing and thawing damage

Physical deterioration in RC structures resulting from freeze/thaw cycles is common in regions with cold climates. The severity of damage is dictated by successive cycles of freeze/thaw, and the concrete moisture content from precipitation or due to direct contact with water (Neville, 1995). Moreover, the degree of damage is influenced to a large extent by the characteristics of concrete including pore structure, type of aggregates and penetrability. The most common forms of damage are scaling resulting from paste failure and pop-outs, and D-cracking which both occur due to failure of aggregate.

There are three basic theories that may demonstrate the mechanism of freeze/thaw damage (Mehta and Monteiro, 2006). First is the hydraulic pressure theory which illustrates freeze/thaw damage caused by hydraulic pressure created by expansion of freezing water. The principle behind this theory is that if the hydraulic pressure exceeds the tensile strength of concrete (*i.e.* pores or voids in concrete with more than 91% full of water will create insufficient space that can accommodate expansion), then cracking and deterioration occurs. Second is the desorption theory which illustrates migration mechanism of water from small pores to large cavities. In this theory, water in large cavities begins to freeze and therefore results in vapour pressure differences, which cause migration of water. Thus, if migration is restrained due to high moisture content or rapid cooling, water pressure may exceed the tensile strength of concrete and lead to cracking. And last is the osmotic pressure theory which illustrates partial freezing of solutions in capillaries. When pore water begins to freeze, the concentration of solutions such as alkalies and chlorides in the unfrozen water in the pore increases and creates pressure causing the paste to fail.

Environmental conditions such as rate of cooling and frequent freeze/thaw cycles are beyond the control of the specifier against freeze/thaw damage. However, the capacity of concrete to accommodate expansion stresses may be improved through the use of frost resistant aggregates and air entraining admixtures (Neville, 1995). Adequate curing practices and proper design details which allow positive drainage of runoff may enhance resistance to freeze/thaw damage (Li et al., 2008). Methods for improving freeze/thaw resistance are summarized in Table 2.1.

Table 2.1: Freeze/thaw damage preventive measures (ACI 201.2R, 2008)

Mechanism of protection	Preventive measures
1.) Provide appropriate air void system	i.) use frost-resistant aggregate and sufficient air-entraining admixtures
2.) Limiting moisture from entering concrete	i.) reduce penetrability by: -low w/c ratio -use mineral admixtures, *e.g.* FA, CSF, GGBS -proper compaction to reduce voids and adequate curing practices
3.) Concrete surface treatments	i.) use waterproof membranes , *e.g.* sealers and surface overlays ii.) adequate drainage to prevent the ponding of water

2.3.1.3 External sulfate attack

Naturally occurring sulfates of sodium, potassium, calcium, or magnesium that may attack concrete are often found in soils or in groundwater. Sulfates may also occur from other sources such as sea water, in industrial effluents, from chemical and mining industries. Sulfate attack on concrete occurs either in form of chemical or physical damage mechanisms as shortly described below.

Chemical attack occurs through a complex mechanism that involves several secondary processes (Ballim et al., 2009), and is initiated by reaction between sulfate ions and calcium hydroxide in the hardened cement paste as follows:

$$\mathrm{Ca(OH)_2 + SO_4^{2-}(aq) \rightarrow CaSO_4.2H_2O\ (gypsum) + 2OH^-(aq)} \qquad [2.6]$$

Formation of gypsum at this step does not result in deterioration. However, formation of enttringite during the second step may result to an increase in solid volume leading to expansion and cracking (Hooton, 2007). This can be shown in the following reaction:

$$3CaSO_4.2H_2O + C_3A + 25H_2O \rightarrow 3CaSO_4.C_3A.31H_2O \quad [2.7]$$

Physical attack on the other hand does not involve any form of reaction with the cement. It occurs when water containing sulfates in concrete evaporates and produces an increase of solid volume that may cause cracking of concrete (Thaulow and Sahu, 2004). This form of attack is common for structures located in sea water within the line of the water level or in the tidal zone where alternating saturation and drying of the concrete dominates.

Sulfate attack destroys the integrity of concrete and therefore affects the mechanical properties of concrete (*i.e.* strength, elastic modulus) (Mehta and Monteiro, 2006). The severity of deterioration is related to the amount of sulfates and the availability of reactive substance (*i.e.* aluminates within the cement), and the rate of transportation. Cracks developed due to sulfates increases the rate of penetrability thus accelerating the process of deterioration. Sulfate attack is influenced by several factors such as the type of cement, the permeability of concrete, and exposure conditions.

Concrete may be protected from sulfate attack by using high quality concrete with suitable constituents that retard the ingress of sulfate. Neville and Brooks (2010) suggested two different ways that improve the quality of concrete. First is by reducing the C_3A content using sulfate resistant cements, where reactions between C_3A and $CaSO_4$ can be avoided. Second is by reducing the quantities of $Ca(OH)_2$ in the cement paste using mineral admixtures such as FA, CSF and GGBS. Moreover, lowering w/cm, sometimes by incorporating air-entraining admixture, is necessary in order to control penetrability since sulfate resistant cements solely slow the rate of sulfate reactions (Bickley et al., 2006b). Additionally, proper placement, compaction, finishing and curing of concrete are essential to minimize the penetrability. Preventive measures against sulfates attack are summarized in Table 2.2.

Table 2.2: Preventive measures against sulfate attack (ACI 201.2R, 2008)

Mechanism of protection	Preventive measures
1.) Limit reactive substances, *i.e.* C_3A and $Ca(OH)_2$	i.) use sulfate resistant cements with low C_3A content ii.) use mineral admixtures, *e.g.* FA, CSF and GGBS
2.) Limiting sulfates from entering concrete	i.) reduce penetrability by: -low w/cm ratio -use mineral admixtures, *e.g.* FA, CSF and GGBS -use chemical admixtures, *e.g.* air entraining -proper placement, compaction, finishing and curing of concrete

2.3.1.4 Acid attack

Generally, concrete is not stable under acidic environments formed in the sewage or wastewater treatment plants, water draining from some mines, or in some industrial waters. In sewers, acid attack occurs as a result of sulphuric acid formed when hydrogen sulfide (H_2S) react with aerobic bacteria located on the inner crown surface of the sewer or in a specific location within a plant. This reaction can be described in the following equation:

$$H_2S + 2O_2 \rightarrow H_2SO_4 \qquad [2.8]$$

Sulphuric acid produced dissolves the calcium silicate hydrate (C-S-H) matrix of concrete and result in deterioration (Kosmatka et al., 2003). The rate of deterioration depends upon the concentration of sulphuric acid solution and the rate of flow of fluid.

Several mechanisms may be applied to reduce the risk of acid attack in concrete structures. A dense concrete with a low w/cm provides a degree of protection against mild acid attack. In high acid concentration (*i.e.* pH of 3 or lower), hydraulic cement solely may not provide adequate protection. Thus, it is recommended to incorporate mineral admixtures such as FA, CSF and GGBS to increase resistance (ACI 201.2R, 2008). Suitable resin coatings may also be required to provide protection for high acid concentration. This method however is only effective when there are neither pinholes nor defects due to coating operations.

2.3.2 Internal Factors

Some of the aggressive factors do not attack concrete directly, but their presence in the concrete mixture can result in chemical reaction and cause deterioration at a later stage. These factors include cement composition, impurities in water and aggregates, and some types of chemical admixtures. Most of these factors get into the concrete during the time of mixing. The most common type of internal factor causing deterioration is discussed in the following sections.

2.3.2.1 Alkali-Aggregate Reaction (AAR)

The mechanism of AAR is a form of chemical attack on concrete. It is a chemical reaction between alkalis and certain types of aggregates in the presence of moisture, which form an expansive gel resulting in concrete cracking (Mehta and Monteiro, 2006). This reaction is very slow and its effects become noticeable after several years of service. The consequences of AAR are cracking in the concrete in the form of a map or spider web pattern, which rarely affects structural integrity. Cracks formation affects the appearance of the structure and provides a means of ingress of moisture and other deleterious substances that result in durability problems. Depending on the types of aggregates, AAR can be classified as either alkali-silica reaction or alkali-carbonate rock reaction as shortly described below.

2.3.2.1.1 Alkali-Silica Reaction (ASR)

Alkali-silica reaction (ASR) is the most common form of AAR. It occurs when alkaline pore solution of concrete react with metastable or highly disordered silica phases (*e.g.* opal, trydimite, cristobalite and volcanic glasses) found in particular silicate aggregates (*e.g.* quartzite, greywacke, granite, etc.) to form expansive alkali-silica gel (Obla, 2005; Oberholster, 2009). The main contributors to the reaction are higher alkalinity in the pore solution, sufficient amount of reactive minerals in the aggregate and conductive environmental conditions of temperature and moisture. Signs of this type of deterioration are mapping cracking, leaking of the gel from cracks and joints, and joint closure due to the gel swelling.

Locally available cements in combination with FA, CSF or GGBS may be used to minimize the risk of ASR. Low-alkali cement (*i.e.* < 0.60% of Na_2O-equivalent) has shown satisfactory performance when used with alkali-reactive aggregates (Farny and Kosmatka, 1997). Certain chemical admixtures that contain lithium compounds when used in sufficient quantity can also reduce ASR expansion in concrete.

2.3.2.1.2 Alkali-Carbonate-Rock Reaction (ACRR)

Alkali-carbonate-rock reaction (ACRR), like ASR, is a chemical reaction that can induce physical damage resulting from expansion and cracking of concrete. It occurs when alkaline pore solutions react with certain carbonate rocks (*e.g.* argillaceous dolomitic limestone) without the formation of alkali-silica gel (Oberholster, 2009). The reaction is believed to be caused by reactive rocks which contain larger crystals of dolomite scattered in and surrounded by a fine-grained matrix of calcite and clay. This reaction is not very common and well-understood compared to ASR. Concrete may contain a certain percentage of carbonate reactive aggregates without experiencing detrimental expansion. Importantly, aggregates susceptible to this kind of reaction are usually unsuitable for concrete production for various reasons such as strength requirements, etc. (Farny and Kosmatka, 1997).

It is difficult to control ACRR once expansion has begun and therefore preventive measures should be taken when aggregates are susceptible to the reaction. One of the preventive measures is to limit the nominal maximum size or the amount of reactive aggregate in the concrete so as to minimize detrimental expansion (Swenson and Gillot, 1960). Compared to ASR, low-alkali cement (*i.e.* < 0.6% of Na_2O-equivalent) and mineral admixtures, even at higher levels, are not very effective at reducing ACRR to acceptable limits (Tang et al., 1994). For instance, larger amounts of pozzolans may possibly reduce ACRR effects, but these amounts are generally too high for practical application. Moreover, alkali content of concrete is more important than alkali cement level, which should be kept as low as possible when using alkali-carbonate reactive aggregate.

Mechanisms of AAR are very complicated and so their determination and precautionary measures (Mehta and Monteiro, 2006). For example, there are no simple ways to determine deleterious or non-deleterious alkali-reactive aggregates or between safe and unsafe alkali concentration in concrete. Moreover, quantitative field data are not always available to prescribe precaution measures against AAR. However, several preventive measures based on laboratory data have been proposed (Farny and Kosmatka, 1997; Oberholster, 2009) as summarized in Table 2.3. By reducing concrete penetrability using low w/cm, movement of alkalis within the concrete can be limited. In addition to limiting the presence of moisture, the use of non-reactive aggregates is of particular importance. Furthermore, concrete containing alkali-reactive aggregates and high alkali content can be protected by improving drainage or by using waterproof membranes so as to limit the access of water.

Table 2.3: Preventive mechanisms against alkali-aggregate reaction (ACI 201.2R, 2008)

Mechanism of protection	Preventive measures
1.) Internal sources (*i.e.* cement, admixtures or aggregates) Limiting alkali content of concrete	i.) use low-alkali cement ii.) use mineral admixtures, *e.g.* FA, CSF and GGBS iii.) use chemical admixtures, *e.g.* lithium salts or alkyl alkoxy silane iv.) use non-reactive aggregate
2.) External sources Limiting alkalis from entering concrete	i.) reduce penetrability by: -low w/c ratio -use mineral admixtures, *e.g.* FA, CSF and GGBS -proper compaction to reduce voids and adequate curing practices
3.) External sources Concrete surface treatments	i.) use waterproof membranes or suitable coatings, *e.g.* silane

2.3.2.2 Internal sulfate attack

Chemical sulfate attack within the concrete may lead to delayed ettringite formation (DEF). It occurs when either gypsum-contaminated aggregate or cement containing high sulfate content is used in the production of concrete (Mehta and Monteiro, 2006). Although the mechanism of concrete deterioration due to DEF is still not clear, some favouring conditions required to induce DEF have been identified (Fu, 1996). These include high curing temperature between 65°C and 70°C, use of Portland cement having a high $S0_3/Al_20_3$, and exposure to a moist environment for a number of years. Moreover, a combination of these conditions is essential to initiate and extend DEF.

The DEF mechanism is common for massive concrete elements that are in contact with water or exposed to wet environments, and for the elements for which the heat of hydration is not released. Although DEF-induced damage is not a common phenomenon in concrete, it may increase the risk of secondary forms of deterioration such as freeze/thaw attack or corrosion of reinforcing steel.

It is generally accepted that to effectively prevent DEF, the internal concrete temperature should be limited to about 70°C during very early life of the concrete. This may be prevented either by direct specification, or indirectly by limiting the cement content, or specifying the use of very low heat cement (BRE, 2001). However, when concrete mixture is exposed to temperature in excess of 70°C, incorporation of mineral admixtures such as FA and GGBS may effectively prevent DEF-induced expansion (Fu, 1996). Several other measures including low w/cm and proper construction practices such as adequate curing may prevent the occurrence of DEF.

2.4 Consequences of Concrete Deterioration

Concrete structures normally require large investments and therefore they are expected to be durable and long-lasting. Concrete durability is influenced by several factors including material compositions and proportions and construction methods. Therefore careful consideration with respect to design, materials, and construction methods are imperative in order to ensure durability.

Structural failure and consequently collapse as a result of concrete deterioration, in particularly corrosion of reinforcing steel, are rare. Some of the examples of the structures collapsed as a result of corrosion of reinforcing steel include the Berlin Congress Hall in 1980 and a post-tensioned concrete bridge in Wales USA in 1984 (Song and Shayan, 1998). Structural failure does not necessarily mean collapse, but also loss of serviceability which is characterized by excessive deflection of structural members, cracking and spalling of the concrete. The loss of serviceability has several detrimental effects including threaten of the people's safety as a result of either falling concrete pieces. A man was killed in New York City while a car was badly damaged in Michigan as a result of concrete slab spilled off a bridge sub-structure (Broomfield, 1997). As a result of falling concrete, metal canopies have been built around the lower floors of high rise buildings to prevent damage.

In addition to structural and serviceability problems, concrete deterioration costs large sum of money for repair and rehabilitation. The higher costs involved to restore durability threaten the private and public sector budgets. Deterioration not only threatens the economic growth but also has a great impact to natural resources and human safety in general.

Inspection of RC structures for deterioration, particularly corrosion damage using non-destructive tests or coring procedures, are quite costly as they require experts to conduct the tests and interpret the results. However, it might be the best solution at this stage before visible sign of damage such as rust, stains and/or cracks appear where simple and cost-effective measures that can restore durability are very high. In other words, if not treated at this stage they may develop into safety problem that is more difficult to control.

As a basis for the durability, efforts should be made preliminarily to obtain the best possible measures that may control concrete deterioration. Unless proper measures with respect to material selection, design and proper construction methods are taken, concrete deterioration will continue to occur and possibly at higher rates. Principally, if deterioration mechanisms are clearly understood and undertake proper preventive measures prior construction, then costs for repair and rehabilitation could be avoided, and potentially serious accidents could be avoided.

2.5 Concluding Remarks

Similar to other construction materials such as timber and ordinary steel, concrete have limited durability in the long-term as a result of the interaction with its environment causing deterioration. Though the mechanisms of deterioration are complex and often varied, the mitigation measures and the means for ensuring durable concrete are widely available and accessible. Research indicated numerous reasons for concrete deterioration where the main ones are related to inadequate attention to durability specifications. The issues concerning durability specifications are partly related to the design standards currently used that does not take into account the mechanisms of deterioration.

Most relevant properties of the concrete are associated with its composition in terms of cement type and content, w/cm and cover depth as far as durability concerns. Many of these requirements are commonly specified in the design standards for durability purposes. Compliance with such requirements has often resulted into insufficient durability, especially for special structures. For instance, compliance with strength requirements does not ensure sufficient penetrability requirements which significantly accounts for durability.

It is well-agreed that by measuring relevant transport properties, the quality of concrete against penetrability can be evaluated and hence determines its durability. However, this is a requirement where most design standards do not provide. In this context, pertinent sections in the design standards should undergo fundamental changes to include specifications that have a direct influence on the in-place performance of the concrete.

CHAPTER 3: PRESENT STATUS OF SELECTED DURABILITY SPECIFICATIONS

3.1 Introduction

The term "service life and/or working life" is introduced in many durability definitions (IS 456, 2000; AS 3600, 2001; EN 1992-1-1, 2004), and it is often erroneously interchanged with the term 'durability' (ACI Committee 365, 2000). The term 'service life' is well-defined in EN 1990 (2002). It states "A structure shall be designed and executed in such a way that it will, during its intended life, with appropriate degrees of reliability and in an economical way;

- sustain all actions and influences likely to occur during execution and use, and
- remain fit for the use for which it is required".

In Table 2.1 of EN 1990 further description of the term 'service life' is given by providing typical examples for different categories of civil structures and their design service life (DSL). These are given in Table 3.1 where the most relevant categories are 4 and 5 for most RC structures, which may be of long-term concern.

Table 3.1: Recommended design service life for different structure types (EN 1990, 2002)

Design Service Life Category	Indicative Design Service Life (years)	Examples of Structures
1	10	Temporary[1]
2	10 to 25	Replaceable Structural Parts
3	15 to 30	Agricultural and Similar Structures
4	50	Building and Other Common Structures
5	100	Monumental Building Structures, Bridges and other Civil Engineering Structures

Notes:

1. Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.

The purpose of design is to provide a structure that has adequate strength, desired serviceability, durability, and that satisfies other relevant requirements such as constructability and budget (AS 3600, 2001). Thus to fulfil this purpose, several elements should be undertaken ranging from a structural point of view to a durability perspective. In case of structural design, the key elements are based on the limit states that identify all ways in which the structure might fail to meet its intended purpose. In this way, the loads to be resisted are specified. As these loads normally vary, partial factors of safety are applied to account for uncertainties (Alexander et al., 2010). In case of durability design, both the structure and the environmental actions are considered. In this way, elements such as DSL, cover depth and material compositions and proportions are determined based on the actual environmental action. Sarja (2000) proposed a design procedure which includes structural design and durability design. The proposed design procedure is modified as presented in a flow chart given in Figure 3.1 below.

As described by Rostam (2003), there are two approaches for ensuring durable RC structures. These include (1) avoidance of the deterioration mechanisms threatening the structure depending on the environmental action, (2) and material compositions and proportions optimisation. The former rely on the preventive measures that include the use of non-reactive materials e.g. stainless steel or non-metallic reinforcement, or use of surface treatment for concrete or steel, which is beyond the scope of this dissertation. The later is of particular importance as it allows different types of interventions and it is mostly specified in the design standards, which is the focus of the dissertation. In the following sections, a brief description of the approaches related to material compositions and proportions for durability specifications are presented.

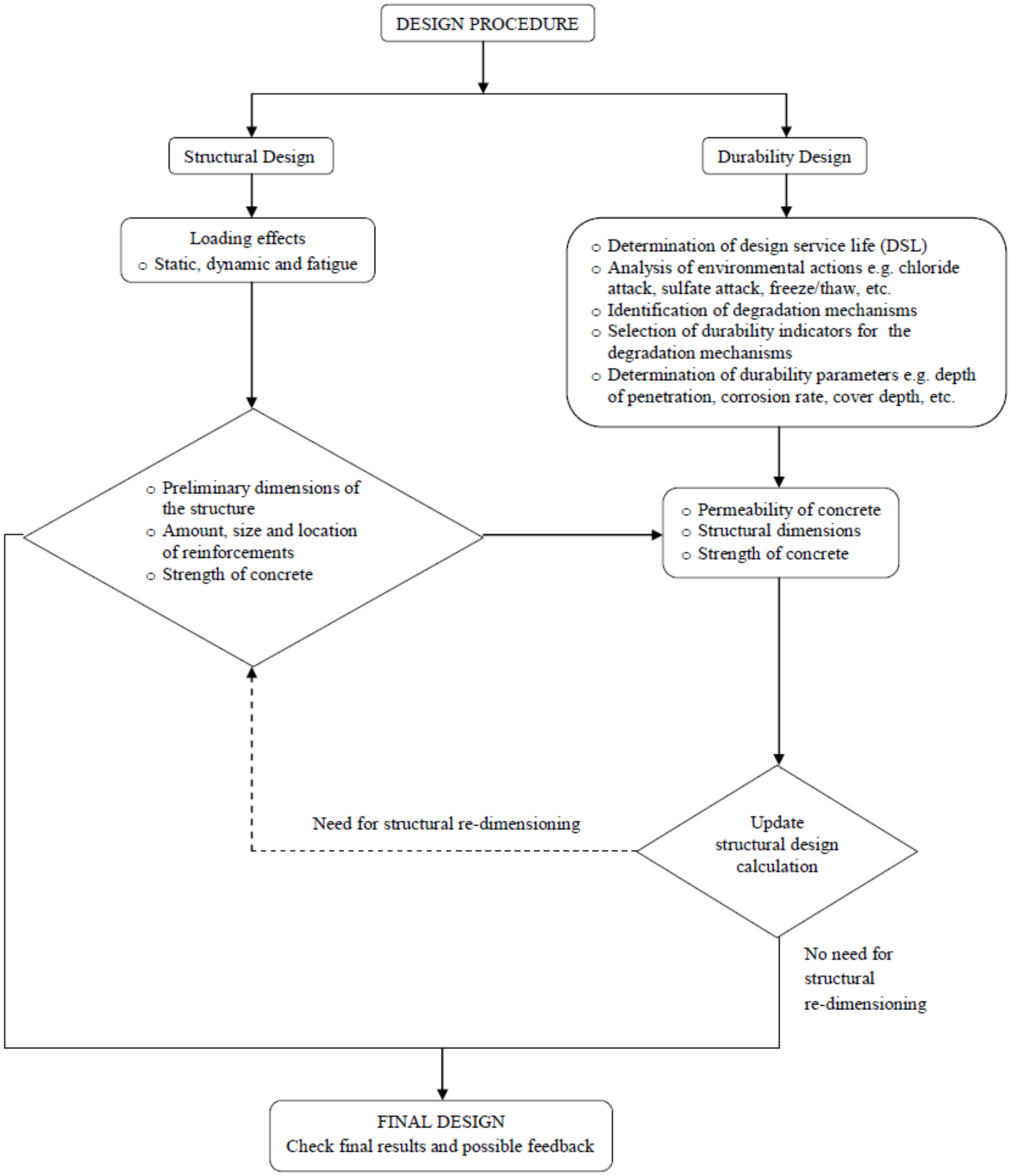

Figure 3.1: Flow chart of design procedure (*Adapted from* Sarja, 2000)

3.2 Current Overview of Durability Provisions in Design Standards

When considering alternative approaches to specifying concrete, it is imperative to understand the current state of practice in the design standards. In this way, durability provisions in the design standards developed in Australia, Canada, USA, Europe, India, and South Africa are reviewed, as an overview of the present state-of-the-art of durability specifications. The basis of the review is not to provide a comprehensive comparison of all provisions related to durability but to limit the effort to the environmental actions, limiting values for concrete compositions and proportions, and cover depth to the reinforcing steel. Figure 3.2 illustrates the mode of review undertaken.

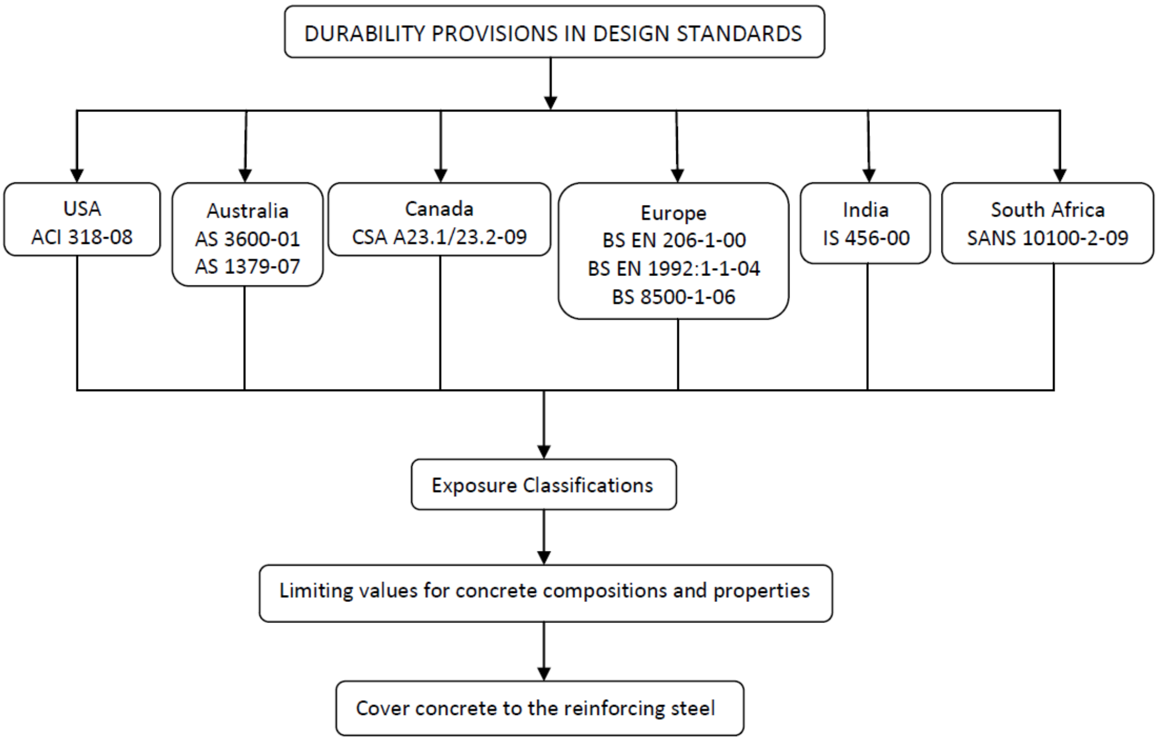

Figure 3.2: Durability provisions review mode in different design standards

3.2.1 American Concrete Institute, ACI 318 (2008)

ACI 318 (2008) *"Building Code Requirements for Structural Concrete and Commentary"* provides minimum requirements for materials, design and construction practices. It also covers strength evaluation of existing concrete structures. It does not cover detailed design and construction of special structures e.g. silos, arches, bins, chimneys, or composite structures, but the basic principles with respect to concrete quality should comply with the standard requirements.

3.2.1.1 Environmental exposure conditions in ACI 318

Chapter 4 of ACI 318, Table 4.2.1, defines exposure categories and classes, excluding carbonation, based on the degree of severity of exposure condition. The following exposure categories are defined:

- Class 'F' for concrete exposed to freezing and thawing,

- Class 'S' for concrete exposed to sulfates,
- Class 'P' for concrete requiring low permeability, and
- Class 'C' for concrete subjected to corrosion of reinforcement.

These exposure categories are further sub-divided into sub-classes as shown in Table 3.2. Following this standard, the specifier and/or designer is required to select relevant exposure classes for each structural member when more than one exposure class is involved.

Table 3.2: Exposure categories and classes in ACI 318

Category	Severity	Class	Condition	
F Freezing and thawing	Not applicable	F0	Concrete not exposed to freezing and thawing cycles	
	Moderate	F1	Concrete exposed to freezing and thawing cycles and occasional exposure to moisture	
	Severe	F2	Concrete exposed to freezing and thawing cycles and in continuous contact with moisture	
	Very severe	F3	Concrete exposed to freezing and thawing and in continuous contact with moisture and exposed to de-icing chemicals	
S Sulfate			**Water-soluble sulfate (SO_4) in soil, % by weight**	**Dissolved sulfate (SO_4) in water, mg/*l***
	Not applicable	S0	$SO_4 < 0.10$	$SO_4 < 150$
	Moderate	S1	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 < 1500$ Seawater
	Severe	S2	$0.20 \leq SO_4 \leq 2.00$	$1500 \leq SO_4 \leq 10{,}000$
	Very severe	S3	$SO_4 > 2.00$	$SO_4 > 10{,}000$
P Requiring low permeability	Not applicable	P0	In contact with water where low permeability is not required	
	Required	P1	In contact with water where low permeability is required	
C Corrosion protection of reinforcement	Not applicable	C0	Concrete dry or protected from moisture	
	Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides	
	Severe	C2	Concrete exposed to moisture and an external source of chlorides from de-icing chemicals, salt, brackish water, seawater, or spray from these sources	

3.2.1.2 Limiting values for concrete composition and properties in ACI 318

Chapter 4 of ACI 318 contains tables of durability requirements in terms of maximum w/cm, minimum compressive strength and additional requirements depending on the type of exposure category. Additional requirements include limits on air content and SCMs for freeze/thaw exposure, limits on chloride content for chloride exposure, and requirements for cement types for sulfate resistance. These requirements are summarised and linked together as shown in Table 3.3.

While the requirements contained in Table 3.3 are clearly prescriptive, except for sulfate requirements, the standard permits some flexibility for materials selection and proportioning. It allows the use of alternative combinations of cementitious materials to be combined to provide resistance against fluid penetration.

Table 3.3: Requirements for concrete composition and properties in ACI 318

Exposure Class	**Max w/cm**	**Min f_c (MPa)**	**Additional requirements**
F0	-	17	-
F1	0.45	31	Lower entrained air
F2	0.45	31	Higher entrained air
F3	0.45	31	Higher entrained air and limits on SCMs
S0	-	17	-
S1	0.50	28	Cement types : ASTM II, IP(MS), IS(<70)(MS) No restriction to calcium chloride admixture Maximum expansion (ASTM C 1012) : 0.10% at 6 months
S2	0.45	31	Cement types : ASTM V, IP(HS), IS(<70)(HS) Calcium chloride admixture is not permitted Maximum expansion (ASTM C1012) : 0.05% at 6 months or 0.10% at 12 months
S3	0.45	31	Cement types : ASTM V + pozzolan or slag, IP(HS) + pozzolan or slag or IS(<70)(HS) + pozzolan or slag Calcium chloride admixture is not permitted Maximum expansion (ASTM C1012) : 0.10% at 18 months
P0	-	17	-
P1	0.50	28	
C0	-	17	Chloride ion limit (water soluble chloride by % wt. of cement) of 1.00 in Reinforced Concrete
C1	-	17	Chloride ion limit (water soluble chloride by % wt. of cement) of 0.30 in Reinforced Concrete
C2	0.40	35	Chloride ion limit (water soluble chloride by % wt. of cement) of 0.15 in Reinforced Concrete Adequate cover

3.2.1.3 Minimum cover depth to the reinforcing steel in ACI 318

Section 7.7 of ACI 318 presents provisions for minimum cover depth to the reinforcement based on concrete quality as indicated in Table 3.4. Nevertheless, the specified cover depth is not linked directly to the exposure classes. For concrete in corrosive environments or other severe exposure conditions, the standard recommends the amount of concrete protection to be suitably increased.

Table 3.4: Minimum cover depth to the reinforcing steel in ACI 318

Exposure conditions	Concrete cover (mm)
Concrete cast against and permanently exposed to earth	75
Concrete exposed to earth or weather	50
Concrete not exposed to weather or in contact with ground (Beam, Column)	40

3.2.1.4 General comments on durability provisions in ACI 318

ACI 318 is a prescriptive standard with respect to durability specifications. It imposes limits to w/cm, entrained air, maximum SCMs content, maximum chloride content, and specifies types of cement to be used in certain exposure categories. It defines exposure conditions in an expanded form based on the anticipated severity of exposure. ACI 318 promotes improvement of properties related to penetrability by limiting w/cm and by the use of SCMs. Though ACI 318 provides cover depth to the reinforcing steel based on the concrete quality, it does not link with the exposure classes. As it can be observed, ACI 318 does not consider relevant factors with respect to exposure classes and concrete quality, which has a direct influence on the durability.

3.2.2 Australian Standards: AS 3600 (2001) and AS 1379 (2007)

AS 3600 (2001) *"Concrete Structures"* provides minimum requirements for the design and construction of plain and reinforced concrete structures. It does not cover the design of mass concrete structures.

AS 1379 (2007), *"Specification and Supply of Concrete"* provides minimum requirements for (a) materials, plant and equipment used in the supply of concrete, (b) production, if applicable, the delivery of concrete in the plastic state, (c) specifying, sampling, testing and compliance with specified properties of plastic and hardened concrete, and (d) uniformity of mixing. Though it covers the supply of all concretes, it does not intend to be applied for mortars or grouts.

3.2.2.1 Environmental exposure conditions in AS 3600

Clause 4.3 of AS 3600 categorises exposure conditions for a surface of a member in the following main groups:

- In contact with the ground,
- In interior environments,
- In above-ground exterior environments,

- In water, and
- In other environments.

Table 4.3 of AS 3600 classifies the main groups into 17 sub-classes as shown in Table 3.5. In the table, class A represents the most benign condition while class C represents the most severe condition. Class ‘U’ refers a condition for which the degree of exposure severity is not fully known and therefore needs proper assessment prior to specifying concrete.

Table 3.5: Exposure classifications in AS 3600

Surface and exposure environments	Exposure classification	
	Reinforced or pre-stressed concrete members (Note 1)	**Plain concrete members (Note 1)**
1. Surface of members in contact with the ground		
(a) Members protected by a damp-proof membrane	A1	A1
(b) Residential footings in a non-aggressive soils	A1	A1
(c) Other members in non-aggressive soils	A2	A1
(d) Members in aggressive soils (Note 2)	U	U
2. Surfaces of members in interior environments		
(a) Fully enclosed within a building except for a brief period of weather exposure during construction	A1	A1
(b) In industrial buildings, the member being subject to repeated wetting and drying	B1	A1
3. Surfaces of members in above-ground exterior environments in areas that are:		
(a) Inland (>50 km from coastline) environment being -		
(i) non-industrial and arid climatic zone (Notes 3 and 4)	A1	A1
(ii) non-industrial and temperate climatic zone	A2	A1
(iii) non-industrial and tropical climatic zone	B1	A1
(iv) industrial and any climatic zone	B1	A1
(b) Near-coastal (1 km to 50 km from coastline) any climatic zone	B1	A1
(c) Coastal (up to 1 km from coastline but excluding tidal and splash zones) (Note 5), any climatic zone	B2	A1
4. Surfaces of members in water		
(a) In fresh water	B1	A1
(b) In sea water-		
(i) permanently submerged	B2	U
(ii) in tidal or splash zones	C	U
(c) In soft or running water	U	U
5. Surfaces of members in other environments Any exposure environment not otherwise described in items 1 to 4	U	U

Notes:

1. In this context, reinforced concrete includes any concrete containing metals that rely on the concrete for protection against environmental degradation. Plain concrete members containing reinforcement or other metallic embedment should, therefore, be treated as reinforced members, when considering durability.
2. Permeable soils with a pH < 4.0, or with ground water containing more than 1 g per litre of sulphate ions, would be considered aggressive. Salt-rich soils in arid areas should be considered as exposure classification C.

3. The climatic zones referred to are those given in Fig 3.3
4. Industrial refers to areas that are within 3 km of industries that discharge atmospheric pollutants.
5. For the propose of this table, the coastal zone includes locations within 1 km of large expanses of salt water (e.g. Port Phillips Bay, Sydney Harbour east of Spit and Harbour Bridges, Swan River west of the Narrows Bridge). Where there are strong prevailing winds or vigorous surf, the distance should be increased beyond 1 km and higher levels of protection should be considered. Proximity to small salt water bays, estuaries and rivers may be disregarded.

Since exposure conditions vary significantly according to geographical locations, Figure 4.3 of AS 3600 offers a map of the Australian continent indicating different zones such as tropical, arid and temperate zone. The map is given in Figure 3.3 and should be read in conjunction with Table 3.5.

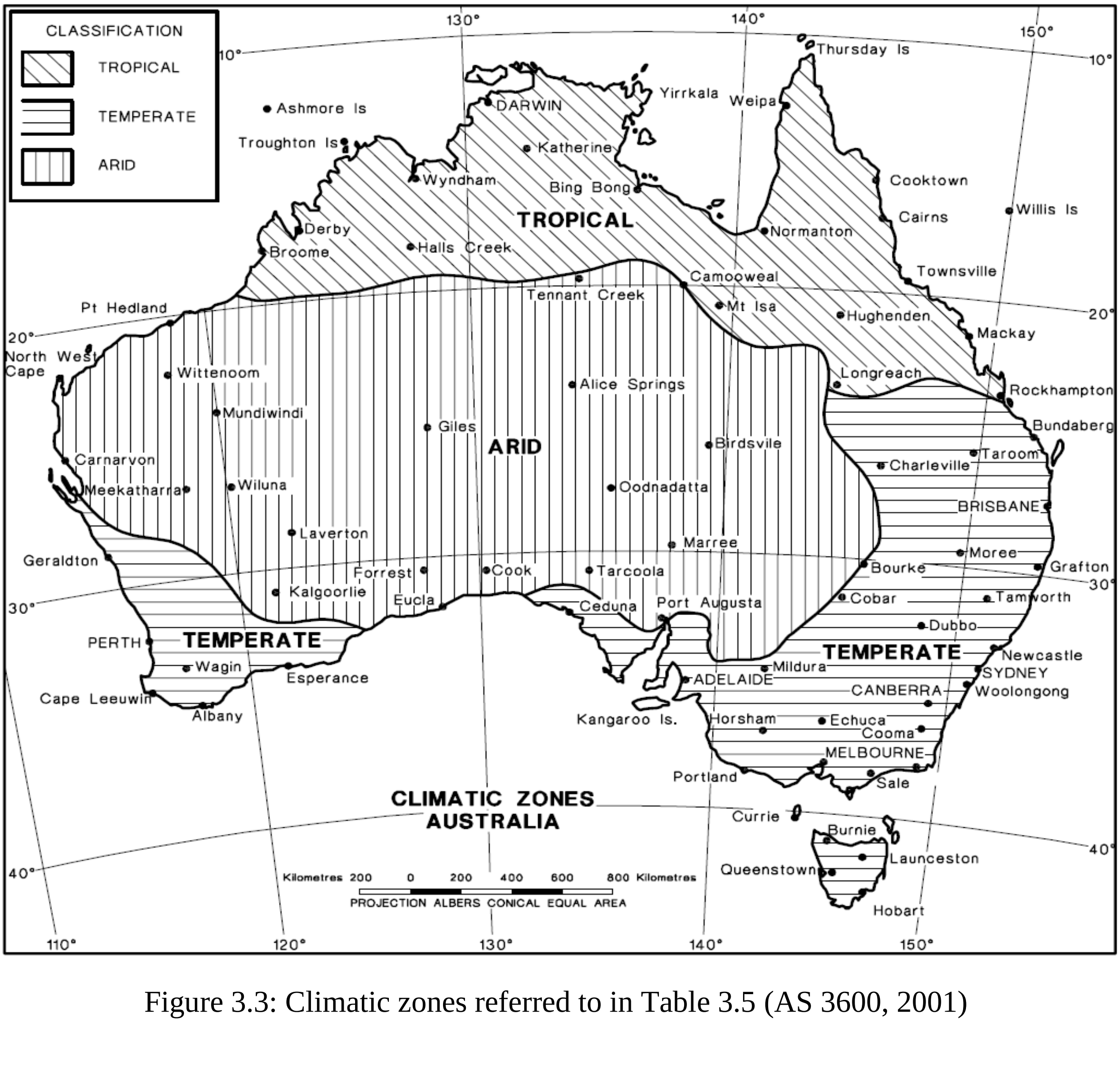

Figure 3.3: Climatic zones referred to in Table 3.5 (AS 3600, 2001)

3.2.2.2 Limiting values for concrete composition and properties in AS 3600

In Clause 4.4 to 4.6 of AS 3600, present the requirements for concrete in terms of minimum compressive strength and types of curing for ‘Normal Class’ concrete. Clause 4.8 of AS 3600 provides additional requirements for concrete exposed to freeze/thaw attack in terms of minimum compressive strength and entrained air content. These values are reproduced in Table 3.6.

Table 3.6: Requirements for concrete composition and properties in AS 3600

Exposure Class	f_c (MPa)	Curing requirement	
		Initial continuous curing (days)*	**Average compressive strength at completion of curing (MPa)**
A1	Not less than 20	3	Not less than 15
A2	Not less than 25	3	Not less than 15
B1	Not less than 32	7	Not less than 20
B2	Not less than 40	7	Not less than 25
C	Not less than 50	7	Not less than 32
U	Concrete shall be specified to ensure durability under the particular exposure environment		
Additional requirements for freezing and thawing	Not less than 32 MPa for occasional exposure Not less than 40 MPa for frequent exposure Air content between 8% to 4% for 10 – 20 mm nominal size aggregate Air content between 6% to 3% for 40 mm nominal size aggregate		

Note:

1. *Provision will not apply for concrete cured by accelerated method. However, average compressive strength requirement at the completion of accelerated curing will govern.
2. Where the strength requirement of class C cannot be satisfied due to inadequate aggregate strength, concrete with f_c not < 40 MPa may be used provided that cement content of the mix is not < 470 kg/m^3 and cover required by Clause 4.10.3 (Cover for corrosion protection) increased by 10 mm.

AS 3600 does not provide requirements for material compositions and proportions. These are given in AS 1379 for production of ‘Normal Class’ and ‘Special Class’ concrete. Since ‘Special Class’ concrete in AS 1379 is specified either in prescriptive or performance basis, specific details are discussed in chapter 4. The requirements for ‘Normal Class’ concrete in AS 1379 that should correspond with the requirements presented in AS 3600, are discussed below in this section.

In AS 1379 ‘Normal Class’ concrete refers to concrete that can be produced by plants throughout Australia, using materials and practices meeting relevant Australian Standards. It is primarily specified by compressive strength and other parameters such as density, acid-soluble chloride content and sulfate content. Several basic parameters such as maximum nominal size of aggregate, slump at the point of acceptance, method of placement, air content, and strength grade should be specified by the customer when ordering this type of concrete. These parameters are described in clause 1.5.3 of AS 1379 and are summarised in Table 3.7 below.

Table 3.7: Requirements for 'Normal Class' concrete in AS 1379 (2007)

Specified parameter	**Requirements for 'Normal Class' concrete**
Basic ordering parameter	
Maximum nominal aggregate size, mm	10, 14, or 20
Slump at the point of acceptance, mm	20 to 120 in 10 mm intervals
Maximum air content, %	≤ 5.0
Minimum compressive strength, MPa	20, 25, 32, 40 and 50
Additional ordering parameter	
Cement type	Any cement complying with AS 3972 alone or in combination with one or more SCMs.
Density, kg/m^3	2100 to 2800
Acid-soluble chloride content, kg/m^3	≤ 0.8
Sulfate content expressed as $S0_3$, g/kg	≤ 50
Temperature at discharge, °C	5 < t < 35

3.2.2.3 Minimum cover depth to the reinforcing steel in AS 3600

AS 3600 considers types of formwork and compaction for cover depth provisions at different grades of concrete. Table 4.10.3.2 and Table 4.10.3.4 of AS 3600 provide minimum cover depth for standard and rigid formwork as well as normal and intense compaction. These values are reproduced in Table 3.8.

Table 3.8: Minimum cover depth to the reinforcing steel in AS 3600

Exposure Classification	**Required cover, mm**				
	Characteristic strength, f_c (MPa)				
	20	**25**	**32**	**40**	**≥ 50**
Standard formwork and compaction					
A1	20	20	20	20	20
A2	(50)	30	25	20	20
B1	-	(60)	40	30	25
B2	-	-	(65)	45	35
C	-	-	-	(70)	50
Rigid formwork and intense compaction					
A1	15	15	15	15	15
A2	(35)	20	15	15	15
B1	-	(45)	30	25	20
B2	-	-	(50)	35	25
C	-	-	-	(55)	40

Notes:

1. Bracketed figures are the appropriate covers when concession given in Clause 4.3.2 (concession for exterior exposure of a single surface), relating to the strength grade permitted for a particular exposure classification, is applied.

2. Increased values are required if Clause 4.10.3.3 (cast against ground) applies.

3.2.2.4 General comments on durability provisions in AS 3600 and AS 1379

AS 3600 is a prescriptive standard with respect to durability provisions. Exposure classifications in AS 3600 are not environment-specific but rather ranges from A to C regardless of the environment. For instance, where exposure conditions considered similar in terms of severity, they are grouped together in lieu of their environment. In addition to defined exposure classes, AS 3600 also include a map of the Australian continent showing different macroclimates that may impair durability of RC structures. Macroclimates are further classified based on their distance from the coastline, i.e. coastal "up to 1 km from the coast", near-coastal "1 km to 50 km from the coast", and inland "beyond 50 km towards the interior".

As shown in previous section, AS 3600 does not provide requirements for material compositions and proportions. These are covered in AS 1379 for 'Normal Class' concrete that should comply with other requirements presented in AS 3600, and for 'Special Class' concrete that is more of performance as described in chapter 4. AS 3600 specifies concrete strength for different exposure conditions with respect to the type of curing. It also relates cover depth to the reinforcing steel with exposure classes and the types of compaction for different types of formwork.

3.2.3 Canadian Standard, CSA A23.1/A23.2 (2009)

CSA A23.1/A23.2 (2009), *"Concrete materials and methods of concrete construction/Test methods and standard practices for concrete"* provides requirements for materials and methods of construction for (a) cast *in-situ* concrete and concrete precast in the field, and (b) residential concrete used in the construction of buildings conforming to Part 9 of the National Building Code of Canada (NBCC). It does not specify the following: (a) requirements for the design of concrete structures, which are provided in CSA A23.3 and CSA-S6, (b) designs of specialty concrete products, which are described in separate CSA Standards, (c) design provisions governing the fire resistance of RC structures, which are set out in NBCC, (d) requirements for the plant production of precast concrete, which are provided in CSA A23.4, and (e) use of proprietary materials or methods of construction; however, this may be permitted by the owner under a separate specification, provided that the quality of the resulting construction meets the minimum requirements of this Standard.

3.2.3.1 Environmental exposure conditions in CSA A23.1/A23.2

CSA A23.1/A23.2 presents the following five major exposure classes. For clarity, the major exposure classes are further sub-divided into 14 sub-classes dealing with different degrees of severity. Table 1 of CSA A23.1/A23, providing the definitions of exposure classes with typical examples, is reproduced in Table 3.9 below.

- Class ‘C’ for concrete exposed to chloride exposure,
- Class ‘F’ for concrete exposed to freezing and thawing without chlorides,
- Class ‘N’ for concrete exposed to neither chlorides nor freezing and thawing,
- Class ‘A’ for concrete exposed to chemical attack, and
- Class ‘S’ for concrete exposed to sulfate attack.

3.2.3.2 Limiting values for concrete composition and properties in CSA A23.1/A23.2

CSA A23.1/23.2 specifies requirements for durability based on both prescriptive and performance limiting values. In terms of prescriptive requirements, maximum w/cm, minimum compressive strength and the age at test, air content, cement restrictions, and minimum period and type of curing are specified. In terms of performance requirements, maximum chloride permeability values are given for the critical exposure classes. Both prescriptive and performance limiting values are given in Table 3.10 below.

Table 3.9: Environmental exposure classes in CSA A23.1/23.2

Subclass	Definition
	Chloride exposures
C-XL	Structurally reinforced concrete exposed to chlorides or other severe environments with or without freezing and thawing conditions, with higher durability performance expectations than the C-1, A-1, or S-1 classes.
C-1	Structurally reinforced concrete exposed to chlorides with or without freezing and thawing conditions. Examples: bridge decks, parking decks and ramps, portions of marine structures located within the tidal and splash zones, concrete exposed to seawater spray, and salt water pools.
C-2	Non-structurally reinforced (i.e., plain) concrete exposed to chlorides and freezing and thawing. Examples: garage floors, porches, steps, pavements, sidewalks, curbs, and gutters.
C-3	Continuously submerged concrete exposed to chlorides, but not to freezing and thawing. Examples: underwater portions of marine structures.
C-4	Non-structurally reinforced concrete exposed to chlorides, but not to freezing and thawing. Examples: underground parking slabs on grade.
	Freezing- and-thawing exposures
F-1	Concrete exposed to freezing and thawing in a saturated condition, but not to chlorides. Examples: pool decks, patios, tennis courts, freshwater pools, and freshwater control structures.
F-2	Concrete in an unsaturated condition exposed to freezing and thawing, but not to chlorides. Examples: exterior walls and columns.
	Not exposed to exterior influences
N	Concrete not exposed to chlorides, nor to freezing and thawing. Examples: footings and interior slabs, walls, and columns.
	Exposed to chemical attack
A-1	Structurally reinforced concrete exposed to severe manure and/or silage gases, with or without freeze-thaw exposure. Concrete exposed to the vapour above municipal sewage or industrial effluent, where hydrogen sulphide gas might be generated. Examples: reinforced beams, slabs, and columns over manure pits and silos, canals, and pig slats; and access holes, enclosed chambers, and pipes that are partially filled with effluents.
A-2	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure. Examples: reinforced walls in exterior manure tanks, silos and feed bunkers, and exterior slabs.
A-3	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure in a continuously submerged condition. Concrete continuously submerged in municipal or industrial effluents. Examples: interior gutter walls, beams, slabs, and columns; sewage pipes that are continuously full (e.g., force mains); and submerged portions of sewage treatment structures.
A-4	Non-structurally reinforced concrete exposed to moderate manure and/or silage gases and liquids, without freeze-thaw exposure. Examples: interior slabs on grade.
	Exposed to sulfate attack
S-1	Concrete subjected to very severe sulfate exposures (Tables 2 and 3).
S-2	Concrete subjected to severe sulfate exposure (Tables 2 and 3).
S-3	Concrete subjected to moderate sulfate exposure (Tables 2 and 3).

Notes:

1. All classes of concrete exposed to sulfates shall comply with the minimum requirements of ‘S’ class noted in Table 2 and 3 of CSA A23.1/23.2 (Table 2: Requirements for C, F, N, A, and S classes of exposure and Table 3: Additional requirements for concrete subjected to sulfate attack). In particular, Classes A-1 to A-4 in municipal sewage elements could be subjected to sulfate exposure.

Table 3.10: Requirements for concrete composition and properties in CSA A23.1/23.2

Class of exposure	Maximum water-to-cementing materials ratio*	Minimum specified compressive strength, MPa and age (d) at test	Air content for 14-20 mm nominal aggregate size	Curing type Normal concrete (Not HVSCM)#	Cement restrictions	Chloride ion penetrability requirements and age (d) at test** (Coulombs)
C-XL	0.37	50 within 56d	4-7 or 5-8 % if exposed to freezing	Extended	-	< 1000 within 56 d
C-1 or A-1	0.40	35 at 28d	4-7 or 5-8 % if exposed to freezing	Additional	-	< 1500 within 56 d
C-2 or A-2	0.45	32 at 28d	5-8 %	Additional	-	-
C-3 or A-3	0.50	30 at 28d	4-7 %	Basic	-	-
C-4**** or A-4	0.55	25 at 28d	4-7 %	Basic	-	-
F-1	0.50	30 at 28d	5-8 %	Additional	-	-
F-2	0.55	25 at 28d	4-7 %	Basic	-	-
N***	For structural design	For structural design	None	Basic	-	-
S-1	0.40	35 at 56d	4-5 %	Additional	HS or HSb	-
S-2	0.45	32 at 56d	4-7 %	Basic	HS or HSb	-
S-3	0.50	30 at 56d	4-7 %	Basic	MS or MSb^{+}	-

Source: Bickley et al., 2006b (*Modified from* CSA A23.1/23.2-2009: Table 2)

Notes:

1. * The water-to-cementing materials (w/cm) ratio shall not be exceeded for a given class of exposure, regardless of exceeding the strength requirement.
2. **Where calcium nitrite corrosion inhibitor is to be used, the same concrete mixture, but without calcium nitrite, shall be pre-qualified to meet the requirements for the permeability index in this table.
3. ***To allow proper finishing and wear resistance, Type N, concrete intended for use in an industrial concrete floor with a troweled surface exposed to wear shall have a minimum cementing materials content of 265 kg/ m^3.
4. ****The requirement for air-entrainment should be waived when steel troweled finish is required. Interior ice rink slabs and freezer slabs with a steel troweled finish have been found to perform satisfactorily without entrained air.
5. $^{+}$Other types of cements meeting LH, HS, HSb are also allowed. Although LH cements are for low heat, they are allowed for moderate sulphate resistance based on C_3A content.
6. #*Basic curing* - 3 day at ≥ 10°C or for the time necessary to attain 40% of the specified strength; *Additional curing* – 7 day at ≥ 10°C and for the time necessary to attain 70% of the specified strength; *Extended wet curing* - A wet-curing period of 7 day at ≥ 10°C. The curing types allowed are ponding, continuous sprinkling, absorptive mat, or fabric kept continuously wet.
7. For HVSCM (high-volume SCMs) concretes, curing requirements are more rigorous for some severe exposures.
8. MS and HS cements refers to Moderate and Highly Sulphate resistant Portland cements while MSb and HSb refers to the use of blended cements or Portland-SCM.

Application of SCMs in combinations with Portland cement is allowed provided that they comply with the requirements given in CSA A3001 (Cementitious materials for use in concrete). For such combinations, concrete should be cured and tested for compressive strength at least for 56 day especially in extreme chloride exposure conditions in order to present better strength characteristics.

Concrete exposed to sulfates should comply with the minimum requirements of class 'S' given in Table 3.10 for all classes. Additional requirements are given in the Appendix (Table A.1) regarding severity of sulfates exposure, cementing materials to be used, and performance requirements of the binder.

3.2.3.3 Minimum cover depth to the reinforcing steel in CSA A23.1/A23.2

Table 17 of CSA A23.1/23.2 presents provision for minimum cover depth to the reinforcing steel based on the life expectancy of the structure, exposure conditions, protective systems, and consequences of corrosion. These values are reproduced in Table 3.11. Since it is difficult to achieve the specified cover depth in some cases, CSA A23.1/23.2 allows a tolerance of ±12 (however, the cover depth shall in no case be reduced by more than 1/3 of the specified cover depth).

Table 3.11: Minimum cover depth to the reinforcing steel in CSA 23.1/23.2

Exposure condition	**Exposure class**		
	N*	**F-1, F-2, S-1, S-2**	**C-XL, C-1, C-3, A-1, A-2, A-3**
Cast against and permanently exposed to earth	-	75 mm	75 mm
Beams, girders, columns, and piles	30 mm	40 mm	60 mm
Slabs, walls, joists, shells, and folded plates	20 mm	40 mm	60 mm

Notes:

1. *This refers only to concrete that will be continually dry within the conditioned space (i.e., members entirely within the vapour barrier of the building envelope)
2. Greater cover or protective coatings might be required for exposure to industrial chemicals, food processing, and other corrosive materials.

3.2.3.4 General comments on durability provisions in CSA A23.1/A23.2

CSA A23.1/23.2 defines exposure conditions with typical examples, which provides guidance when need to select appropriate exposure class. Though CSA A23.1/23.2 imposes limits to several durability requirements, it also specifies the curing type and penetrability limits for the critical exposure classes. Moreover, cover depth requirements are given with tolerance in order to ensure proper protection.

Several parameters which have a direct influence on in-place concrete performance are determined and accepted based on the limits given in CSA A23.1/23.2. For instance, chloride permeability for chloride exposure, air-void system for freeze/thaw resistance and expansion limits for sulfate resistance are determined using relevant ASTM Standards. Though CSA A23.1/23.2 is not strictly a prescriptive or performance-based standard, need to be followed closely by other standards which are typically prescriptive.

3.2.4 European Standard (EN 206-1: 2000) and British Standard (BS EN 1992-1-1:2004 and BS 8500-1: 2006)

EN 206-1(2000) *"Concrete – Part 1: Specification, Performance, Production and Conformity"* applies to concrete for structures cast *in situ*, precast structures, and structural precast products for buildings and civil engineering structures. It describes the requirements for classification, properties verification, design types, delivery, conformity control and criteria, and production control and conformity. The basic guidance for performance-based approach to specifications is given in Annex J of EN 206-1.

EN 1992-1-1 (2004) *"Design of concrete structures – Part 1-1: General rules and rules for buildings"* applies to the design of buildings and civil engineering works in plain, reinforced and pre-stressed concrete. It describes the principles and requirements for safety, serviceability and durability of concrete structures, together with specific provisions for buildings. The requirements for durability are given in section 4 of EN 1992-1-1, in particular for cover depth requirements for corrosion protection.

BS 8500-1 (2006) *"Concrete – Complementary British Standard to EN 206-1 – Part 1: Method of specifying and guidance for the specifier"* provides UK national provisions where required or permitted by EN 206-1. It covers materials, methods of testing and procedures that are out side the scope of EN 206-1, but within the national experience. The guidance for the specifier including concrete quality based on selected exposure classes, intended service life and nominal cover requirements to the reinforcing steel are given in Annex A of BS 8500-1. Guidance on the use of protective measures including stainless steel, non-metallic reinforcement or cover depth requirements for fire protection are beyond the scope of the standard.

3.2.4.1 Environmental exposure conditions in EN 206-1

Section 4.1 of EN 206-1 classifies the environmental actions into six major groups based on different degradation mechanisms as shown below.

- ‘X0’ No risk of corrosion or attack,
- ‘XC’ Corrosion induced by carbonation,
- ‘XD’ Corrosion induced by chlorides other than from sea water,
- ‘XS’ Corrosion induced by chlorides from sea water,
- ‘XF’ Freeze/thaw attack with or without de-icing salts, and
- ‘XA’ Chemical attack.

Table 1 of EN 206-1 further classifies each of the major group into sub-classes resulting in a total of 18 classes with typical examples as shown in Table 3.12. Since concrete may be subject to more than one degradation mechanisms, EN 206-1 allows combinations of exposure classes to account for environmental actions.

EN 1992-1-1 (2004) and BS 8500-1(2006) are among the standards in Europe which have adopted similar exposure classes developed in EN 206-1. In EN 1992-1-1, exposure classes are similar as in EN 206-1 while in BS 8500-1 there are slight changes especially in the informative examples for UK conditions. Moreover, BS 8500-1 allows the specifier to use judgement to cover for the specific exposure conditions that are not described in the standard. Exposure classes presented in BS 8500-1 for concrete exposed to chemical attack vary significantly from the ones given in EN 206-1. For specific details regarding exposure classes in chemical attack, reference should be made in Table 2 of EN 206-1 and Table A.2 of BS 8500-1.

Table 3.12: Exposure classes in EN 206-1

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack	
	For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity.
2 Corrosion induced by carbonation Where concrete containing reinforcement or other embedded metal is exposed to air and moisture. (*see* Note 1)		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity. Concrete permanently submerged in water.
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact. Many foundations.
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity. External concrete sheltered from rain.
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure Class XC2.
3 Corrosion induced by chlorides other than from sea water Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from sea water		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides.
XD2	Wet, rarely dry	Swimming pools. Concrete exposed to industrial waters containing chlorides.
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides. Pavements. Car park slabs.
4 Corrosion induced by chlorides from sea water Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5 Freeze/thaw attack with or without de-icing agents Where concrete is exposed to significant attack by freeze/thaw cycles whilst wet		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing.
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents.
XF3	High water saturation, without de-icing agent	Horizontal concrete surfaces exposed to rain and freezing.
XF4	High water saturation, with de-icing agent or sea water	Road and bridge decks exposed to de-icing agents. Concrete surfaces exposed to direct spray containing de-icing agents and freezing. Splash zones of marine structures exposed to freezing.
6 Chemical attack *see* Note 2		
XA1	Slightly aggressive chemical environment according to Table 2	
XA2	Moderately aggressive chemical environment according to Table 2	
XA3	Highly aggressive chemical environment according to Table 2	

Notes:

1. The moisture condition relates to that in the concrete cover to reinforcement or other embedded metal, but in many cases, conditions in the concrete cover can be taken as reflecting that in the surrounding environment. In these cases classification of the surrounding environment may be adequate. This may not be the case if there is a barrier between the concrete and its environment.
2. When concrete is exposed to chemical attack from natural soils and ground water as given in Table 2 of EN 206-1, the classification should be according to Table 3.12. The classification of sea water depends on the geographical location. Thus, a special study may be needed to establish the relevant exposure condition where there are situation outside the limits of Table 2 of EN 206-1, other aggressive chemicals, chemically polluted ground or water, or high water velocity in combination with the chemicals in Table 2 of EN 206-1.
3. Table 2 of EN 206-1 provides limiting values of SO_4, pH, CO_2, NH_4, Mg for ground water and SO_4 and acidity of natural soil for exposure classes, XA1, XA2 and XA3.

3.2.4.2 Limiting values for concrete composition and properties in EN 206-1

Clause 5.3.2 of EN 206-1 specifies several prescriptive requirements in order to satisfy various exposure classes. These include maximum w/c, minimum strength class, minimum cement content, and minimum air content. Table F.1 of EN 206-1, reproduced as Table 3.13 present the limiting values of concrete composition and properties based on 50 years working life assumption. It should be noted that the requirements presented in Table 3.13 are specifically for cement type CEM 1 and aggregate with maximum nominal size in the range of 20-32 mm.

EN 206-1 covers only CEM 1, and CSF and FA as additions while BS 8500-1 considers more cement types including CEM II, III and IV and additions such as FA, CSF and GGBS. The additions in BS 8500-1 are considered as part of blended cement rather than a replacement to the Portland cement. Moreover, several strength classes such as C6/8, C28/35 and C32/40 that are not in EN 206-1 are included in BS 8500-1. In terms of maximum w/c and minimum cement content, BS 8500-1 gives slight different values for the intended service life of 50 and 100 years when compared to EN 206-1. For specific details regarding concrete composition and proportions with respect to exposure classes for different degradation mechanisms reference should be made in section A.4.2 to A.4.4 of BS 8500-1.

Table 3.13: Recommended limiting values for composition and properties of concrete in EN 206-1

	Exposure classes																	
	No risk of corrosion or attack	**Carbonation -induced corrosion**				**Chloride-induced corrosion**						**Freeze/thaw attack**				**Aggressive chemical environments**		
						Sea water			**Chloride other than from Sea water**									
	X0	XC1	XC2	XC3	XC4	XS1	XS2	XS3	XD1	XD2	XD3	XF1	XF2	XF3	XF4	XA1	XA2	XA3
Maximum w/c	-	0.65	0.60	0.55	0.50	0.50	0.45	0.45	0.55	0.55	0.45	0.55	0.55	0.50	0.45	0.55	0.50	0.45
Minimum strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
Minimum cement content (kg/m^3)	-	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360
Minimum air content (%)	-	-	-	-	-	-	-	-	-	-	-	-	4.0[a]	4.0[a]	4.0[a]	-	-	-
Other requirements												Aggregate in accordance with EN 12620 with sufficient freeze/thaw resistance					Sulfate- resisting cement[b]	

Notes:

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

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

Though cover depth is not among the requirements in EN 206-1, BS 8500-1 and EN 1992-1-1 provides cover depth requirements based on similar exposure classes defined in the standard. BS 8500-1 provides cover depth requirements for different degradation mechanisms while EN 1992-1-1 gives cover depth for corrosion protection only. For other degradation mechanisms, EN 1992-1-1 emphasises on concrete compositions and proportions developed in section 6 of EN 206-1. EN 1992-1-1 further prescribes cover requirements in terms of nominal cover that include minimum cover plus any expected deviation (*i.e.* $c_{nom} = c_{min} + \Delta c_{dev}$). For guidance, cover depth requirements for corrosion protection given in Table 4.4N of EN 1992-1-1 are reproduced in the Appendix A i.e. Table A.2. In case of cover depth requirements developed in BS 8500-1, reference should be made in Table A.4 and Table A.5 of BS 8500-1.

3.2.4.3 General comments on durability provisions in EN 206-1

Generally, EN 206-1 should apply to all European Economic Community members (EEC). However, each country is free to produce an Annex to this standard to cover for specific issues related to national choice. Among the issues left open for national choice include various concrete compositions and cover depth requirements.

EN 206-1 considers corrosion of reinforcement as the main deterioration mechanism, no matter if it is the result of chloride ingress, freeze/thaw attack, or chemical attack. EN 206-1 allows modifications of the prescriptive requirements for minimum cement content and maximum w/c. For instance, section 5.2.5.2 of EN 206-1 discounts SCMs by a coefficient k i.e. w/(c+ k x addition), where k is the efficiency factor in range of 0.2 to 5, with the lower range being characteristic to FA and the upper range to CSF. In this regard, w/(c+ k x addition) should have less than the specified values while for cement plus addition, at least equal minimum cement content presented should be obtained for different exposure classes.

Though EN 206-1 provides prescriptive requirements for different exposure classes, it recommends the requirements to be specified on performance basis when absolute test methods are available. An important element in EN 206-1 is the exposure classes that are well-defined and opens a wide area for scientific investigation.

3.2.5 Indian Standard, IS 456 (2000)

IS 456 (2000) *"Plain and Reinforced Concrete"* presents the requirements for general use of structural concrete for plain and reinforced concrete. Special structures including shells, folded plates, arches, bridges, chimneys, hydraulic structures¸ liquid retaining structures¸ etc., are not covered. However, the standard shall be used in conjunction with the standards covering special structures.

3.2.5.1 Environmental exposure conditions in IS 456

Table 3 of IS 456 provides environmental exposure conditions into five categories based on the level of severity. These categories are shown in Table 3.14.

Table 3.14: Environmental exposure conditions in IS 456

Environment	Exposure conditions
i) Mild	- Concrete surfaces protected against weather or aggressive conditions, except those situated in coastal area
ii) Moderate	- Concrete surfaces sheltered from severe rain or freezing whilst wet - Concrete exposed to condensation and rain - Concrete continuously under water - Concrete in contact or buried under non-aggressive soil/ground water - Concrete surfaces sheltered from saturated salt air in coastal area
iii) Severe	- Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing whilst wet or severe condensation - Concrete completely immersed in sea water - Concrete exposed to coastal environment
iv) Very severe	- Concrete surfaces exposed to sea water spray, corrosive fumes or severe freezing conditions whilst wet - Concrete in contact with or buried under aggressive sub-soil/ground water
v) Extreme	- Surface of members in tidal zone - Members in direct contact with liquid/ solid aggressive chemicals

3.2.5.2 Limiting values for concrete composition and properties in IS 456

Table 5 and Table 16 of IS 456 presents prescriptive requirements for concrete in terms of maximum free w/c ratio, minimum cement content, minimum cover depth, and minimum grade of concrete for different exposure conditions as shown in Table 3.15 below.

Table 3.15: Requirements for concrete composition and properties in IS 456

Exposure	+Minimum cement content kg/m^3	Maximum free water/ cement ratio	Minimum grade of concrete	*Minimum cover depth, mm
i) Mild	300	0.55	M20	20**
ii) Moderate	300	0.50	M25	30
iii) Severe	320	0.45	M30	45***
iv) Very severe	340	0.45	M35	50***
v) Extreme	360	0.40	M35	75

Notes:

1. +Cement content prescribed in the table is irrespective of the grades of cement and it is inclusive of SCMs.
2. *For a longitudinal reinforcing bar in a column, nominal cover shall not be less than 40 mm, nor less than the diameter of such bar.
3. ** For reinforcement up to 12 mm diameter bar for mild exposure, the nominal cover may be reduced by 5 mm.
4. ***For severe and very severe exposure conditions, reduction of 5 mm may be made, where concrete grade is M35 and above. The actual cover concrete should not deviate from the required nominal cover by +10 mm.

IS 456 allows SCMs to be incorporated into concrete if their suitability is established and the maximum amount taken into account does not exceed the limits specified. Minimum curing period of 14 days is specified for concrete incorporating SCMs in dry and hot weather conditions.

An upper limit of cement content of 450 kg/m^3 is introduced for concretes exposed to all conditions. Maximum total acid-soluble chloride content of 0.6 kg/m^3 is given for concrete exposed to chlorides, while total soluble-sulfate content is limited to 4% (expressed as SO_3) by mass of cement in the mix for sulfate exposure. For concretes exposed to sea water or directly along the coastal line, a minimum of M30 concrete strength with inclusion of SCMs is recommended.

3.2.5.3 General comments on durability provisions in IS 456

Although IS 456 was recently revised to present some advances in concrete technology, it is still a prescriptive standard on durability basis for many requirements. IS 456 defines exposure classes in qualitative terms such as mild, moderate, etc., which seems to be restrictive. Several proposals are undertaken to bring the standard up-to-date, especially for exposure classifications following the international trend (Kulkarni, 2009). Although the standard imposes limits to several material compositions and proportions, it encourages the use of SCMs to enhance durability. Cover depth to the reinforcing steel is provided generally for corrosion protection with allowed flexibility depending on the type of the structural member and exposure conditions.

3.2.6 South African Standard, SANS 10100-2 (2009)

SANS 10100-2 (2009) "*The structural use of concrete Part-2: Materials and execution of works"* though currently unpublished, intend to replace SANS 10100-2 (1992). This standard covers materials and execution of work related to the structural use of concrete in buildings and structures for reinforced, pre-stressed and precast concrete. It does not cover the structural use of concrete made with high-alumina cement.

3.2.6.1 Environmental exposure conditions in SANS 10100-2

SANS 10100-2 defines conditions of exposure in an arbitrary form such as mild, moderate, etc. based on the environment to which the concrete structure is subjected. Table 1 of SANS 10100-2 gives detailed definitions of conditions of exposure as shown in Table 3.16 below.

Table 3.16: Exposure conditions in SANS 10100-2

Condition of exposure	Description of member or surface to which the cover applies
1. Moderate conditions	-Surfaces protected by the superstructure, such as the sides of beams and the undersides of slabs and other surfaces not likely to be moistened by condensation -Surfaces protected by a waterproof cover or permanent formwork not likely to be subjected to weathering or corrosion. -Enclosed surfaces -Structures or members permanently submerged -Transnet limited structures: (a) Surfaces of precast elements not in contact with soil (b) Surfaces protected by permanent formwork not likely to be subjected to weathering or corrosion (c) Surfaces in contact with ballast (d) All other surfaces
2. Severe conditions	-All exposed surfaces -Surfaces on which condensation takes place -Surfaces in contact with soil -Surfaces permanently under running water -Transnet Limited structures: (a) Surfaces of precast elements not in contact with soil (b) Surfaces protected by permanent formwork not likely to be subjected to weathering or corrosion (c) Surfaces in contact with ballast (d) All other surfaces -Cast in situ piles: (a) Wet cast against casing (b) Wet cast against soil (c) Dry cast against soil
3 .Very severe conditions	-All exposed surfaces of structures within 30 km from the sea -Surfaces in rivers polluted by industries -Cast in situ piles, wet cast against casings
4 .Extreme conditions	-Surfaces in contact with sea water of industrially polluted water -Surfaces in contact with marshy conditions

Notes:

1. Concrete exposed to mild conditions: Specified strength shall be determined by structural design considerations. If the concrete is to include embedded metal, the characteristic strength shall not be less than 20 MPa. There is no requirement for maximum water/cement ratio or for minimum cement content.

3.2.6.2 Limiting values for concrete composition and properties in SANS 10100-2

SANS 10100-2 provides an upper limit of cementitious binder content of 550 kg/m^3 for all exposure conditions, maximum w/b ratio of 0.53 for freeze/thaw attack and 0.5 for concrete requiring low permeability. A chloride ion limit less than 0.2% by mass of cementitious materials is given for additional chloride ingress from external environment, while 0.4% is given where no additional chloride ingress is expected. For sulfate resistance, total soluble-sulfate content is limited to 4% (expressed as SO_3) by mass of the cementitious binder in the mix. SANS 10100-2 does not restrict types of cements or SCMs to be used as long as they comply with the relevant South African standards.

3.2.6.3 Minimum cover depth to the reinforcing steel in SANS 10100-2

Table 3 of SANS 10100-2 provides minimum cover depth to the reinforcement based on the grade of concrete and the surface to which cover applies for different conditions of exposure. The minimum cover depth applies for normal-density and low-density concrete, however, no clear distinction is made regarding the values provided. These values are given in Table 3.17.

Table 3.17: Minimum cover depth to the reinforcing steel in SANS 10100-2

Condition of exposure	Description of member/ surface to which the cover applies	Minimum cover, mm				
		Grade of concrete, MPa				
		20	25	30	40	50
Moderate	-Surfaces protected by the superstructure, such as the sides of beams and the undersides of slabs and other surfaces not likely to be moistened by condensation. -Surfaces protected by a waterproof cover or permanent formwork not likely to be subjected to weathering or corrosion. -Enclosed surfaces. -Structures or members permanently submerged in water	50	45	40	30	25
Severe	-All exposed surfaces. -Surfaces on which condensation takes place -Surfaces in contact with soil. -Surfaces permanently under running water	NA	50	45	40	35
	-Cast in situ piles: (a) Wet cast against casing (b) Wet cast against soil (c) Dry cast against soil	 50 75 75	 50 75 75	 50 75 75	 50 75 75	 50 75 75
Very severe	-All exposed surfaces of structures within 30 km from the sea -Surfaces in rivers polluted by industries -Cast in situ piles, wet cast against casings	NA NA NA	NA NA NA	NA NA NA	60 60 80	50 50 80
Extreme	-Surfaces in contact with sea water of industrially polluted water -Surfaces in contact with marshy conditions	NA	NA	NA	65	65

Notes:

1. The cover values are characteristic minimum cover values and not more than 5% of cover measurements should fall below these values. In addition, no single cover measurement should fall below 5 mm less than the relevant cover value indicated above.

3.2.6.4 General comments on durability provisions in SANS 10100-2

SANS 10100-2 (2009) though intends to replace the current SANS 10100-2 (1992), it is still lacking in many aspects in terms of durability requirements. It still defines conditions of exposure in qualitative terms such as mild, moderate, etc., which seems to be restrictive. Although it does not restrict the types of cements or SCMs to be used, it also gives no limits to w/cm for many exposure conditions, expect for freeze/thaw attack and for concrete requiring low permeability. SANS 10100-2 also specifies cover depth though in different form, where some of the cover depth reduces with increasing concrete strength, something that conflicts the durability requirements.

SANS 10100-2 is the main concern of this dissertation and therefore a critical analysis of relevant clauses related to durability requirements are reviewed in a subsequent chapter (*i.e.* chapter 7). In addition, several recommendations to bring the standard up-to-date following the international trends are presented.

3.3 Comparison of Relevant Durability Provisions in the Design Standards

3.3.1 Comparison of exposure classes, material compositions and proportions, and cover depth requirements

Comparison of environmental exposure conditions is not an easy task because of many factors including geographical locations and climatic conditions which vary significantly from region to region. In this way, regions subjected to chemical attack, sulfate attack, or ASR, which occurs naturally in soils, aggregates, ground waters, or due to industrial processes, are considered specific conditions and therefore are not discussed in this section. They can be dealt with effectively by controlling material compositions and proportions under a prescriptive approach, or by consulting expert literature relevant on the attack mechanism. Other exposure conditions such as carbonation, chloride and probably freeze/thaw attack which mainly depends on the penetrability of the concrete, appear common worldwide. Moreover, essential parameters related to penetrability may be measured and evaluated based on performance. In this context, a comparison is made with respect to essential parameters related to penetrability in various design standards. Table 3.18 presents various exposure classes and categories for corrosion of reinforcing steel resulting from either carbonation or chloride, and freeze/thaw attack. The approaches required to achieve resistance for such exposure conditions with respect to material compositions and proportions and cover depth is presented in Table 3.19.

From Table 3.18, it can be shown that many design standards classify exposure conditions based on either degree of severity of exposure or cover depth of a member. For chloride-induced corrosion, exposure condition is further classified depending on the distance of the structures from sea water. According to Table 3.18, freeze/thaw attack is well classified in ACI 318, CSA A23.1/23.2, and EN 206-1, where in SANS 10100-2, IS 456 and AS 3600 no particular classes are given.

From Table 3.19, it can be shown that various prescriptive requirements such as maximum w/cm, minimum compressive strength, maximum chloride content, and total air content are given to provide resistance to corrosion of reinforcing steel and freeze/thaw attack. In addition, EN 206-1 gives minimum cement content for different exposure classes while IS 456 and SANS 10100-2 gives maximum total cement content for all exposure categories.

Quality of cover depth is one of the important parameters for controlling penetrability. It is noted that achievement of denser, quality, and adequate cover depth, may prevent penetrability of deleterious substances into concrete. Design standards have different ways of specifying cover depth to meet durability requirements as shown in Table 3.19. For instance, EN 1992-1-1 and SANS 10100-2 specifies cover based on the exposure conditions and compressive strength or structural class, while AS 3600 specifies cover based on the types of formwork or methods of compaction. Other design standards e.g. ACI 318 does not link cover requirements with the exposure conditions, while CSA A23.1/23.2 and IS 456 allows tolerance in the specified cover depth.

Table 3.18: Comparison of environmental exposure classifications in various design standards

ACI 318: 2008	AS 3600: 2001	CSA A23.1/23.2: 2009	EN 206-1/EN 1992-1-1	IS 456: 2000	SANS 10100-2: 2009
Classification is based on exposure severity of structural members	Classification is based on surfaces of members and exposure environment	Classification is based on surfaces of members and exposure environment	Classification is based on the provisions valid in the place of use of concrete	Classification is based on exposure severity of concrete	Classified based on type of structure or surface to which cover applies
Exposure classification related to resistance to corrosion of reinforcement					
C0: No exposure to moisture C1: Exposure to moisture but not to external Chlorides C2: Exposure to moisture and external chlorides	B1: Any climatic zone between 1 km–50 km from coastline B2: Any climatic zone up to 1 km from coastline (no tidal and splash) and permanently submerged C: Surface in tidal or splash zones	C-XL: Exposure to chlorides or other severe conditions requiring higher durability performance C1: Exposure to chlorides in tidal, splash, and spray zones C3: Permanently submerged in chlorides conditions	XCI - XC4: Corrosion induced by carbonation XD1 - XD3: Corrosion induced by chlorides other than from sea water -Chlorides from sea water XS1: Exposed to airborne salt but not in direct contact with sea water XS2: Permanently submerged XS3: Tidal, splash and spray zones	Categorized as mild, moderate, severe, very severe and extreme -Concrete permanently submerged in sea water or along the coastline regarded as severe -Concrete in sea water spray or corrosive fumes regarded as very severe -Concrete in tidal zone regarded as extreme	Categorized as mild, moderate, severe, very severe, and extreme -Concrete surfaces within 30 km from the sea regarded as very severe -Concrete surfaces exposed to abrasive action of sea water regarded as extreme
Exposure classification related to freezing and thawing resistance					
F0: Not exposed at all F1: Freezing and thawing and occasional moisture F2: Freezing and thawing and continuous moisture F3: Freezing and thawing, continuous moisture and de-icing chemicals	No particular class assigned, it's treated as additional durability requirements	F1: Freezing and thawing and moisture condition F2: Freezing and thawing and no moisture condition	XF1: Moderate moisture, without de-icing agent XF2: Moderate moisture, with de-icing agent XF3: High moisture, without de-icing agent XF4: High moisture, with de-icing, or sea water	No category related to freezing and thawing, but concrete exposed to freezing whilst wet is regarded as moderate, severe and very severe	No category related to freezing and thawing exposure. The standard consider it as different exposure category

Table 3.19: Comparison of material compositions and properties and cover depth requirements in various design standards

ACI 318: 2008	AS 3600: 2001	CSA A23.1/23.2: 2009	EN 206-1/EN 1992-1-1	IS 456: 2000	SANS 10100-2: 2009
Resistance of concrete to corrosion of reinforcement					
Maximum w/cm ratio, minimum strength, and maximum chloride ion content are specified based on exposure class	Minimum strength and types of curing are specified based on exposure class	-Maximum w/cm ratio and minimum strength are specified based on exposure class -Performance requirements with respect to limits on chloride permeability is specified	-Maximum w/c ratio, minimum strength class, and minimum cement content are specified based on exposure class -Provisions are made for performance-related specifications with respect to exposure classes	-Maximum free w/c ratio, minimum cement content, and minimum grade of concrete are specified based on exposure conditions -Maximum cement content limited to 450 kg/m^3 for every exposure conditions	Maximum cementitious binder content is limited to 550 kg/m^3 for every exposure conditions
Resistance of concrete to freezing and thawing					
-Maximum w/cm ratio and minimum strength are specified based on exposure class -Total air content limits of concrete as delivered are specified based on exposure class	Minimum strength for occasional and frequent exposure, and minimum air content for different aggregate size are specified	-Maximum w/cm ratio and minimum strength are specified based on exposure class -Minimum air content limits of concrete based on exposure class are given -Performance requirements with respect to limits on air void system are given	-Maximum w/c ratio, minimum strength class, and minimum cement content are given based on exposure class -Minimum air content limits of concrete based on exposure class are given	Total air content limits of concrete as delivered are given	-Maximum w/b ratio is specified -Limits on total air content for normal-density and low-density concrete are specified
Cover requirements to resist both corrosion and freezing and thawing exposure conditions					
-Cover depth is not direct linked with the exposure classes -Recommends cover for corrosion only	Limiting values are based on types of formwork and compaction for classes of concrete and exposure	Limiting values are based on types of structural member and exposure class	-EN 206-1 gives no limits to cover requirements -EN 1992-1-1 provides cover requirements for corrosion only as $C_{nom} = C_{min} + \Delta C_{dev}$, with C_{min} based on structural class and exposure class	-Limiting values are based on exposure conditions	-Limiting values are based on grade of concrete and condition of exposure

Notes:

1. C_{nom} = Nominal cover; C_{min} = Minimum concrete cover; ΔC_{dev} = Addition to the minimum cover to allow for the deviation

3.3.2 Comparison of limiting values for material proportions and cover depth

An indirect control of penetrability based on specified w/cm and acceptance of concrete based on compressive strength is commonly used in most design standards to resist corrosion and freeze/thaw attack. For corrosion resistance in extreme chloride exposure classes, w/cm ranging from 0.4 to 0.5 is widely specified, where compressive strength for both cubes and cylinders ranges between 30-35 MPa when tested at 28-day. In terms of chloride limits, a threshold value in the range of 0.2 to 0.4% by mass of cement is considered acceptable (Song and Shayan, 2000), as can be shown in Table 3.20. Although each design standard complies with this requirement, the threshold value is never constant and varies due to moisture and oxygen content of the environment (Broomfield, 1997).

Table 3.20: Risk of corrosion initiation on total chloride content (Stipanovic, 2005)

Risk of Corrosion	**CL^- % by mass of cement**	**CL^- % by mass of concrete for 450 kg cement/m^3**
Certain	> 2.0	> 0.37
Probable	1.0-2.0	0.19-0.37
Possible	0.4-1.0	0.08-0.19
Negligible	< 0.4	< 0.08

For freeze/thaw exposure, w/cm ranges from 0.45 to 0.55 while minimum compressive strength is approximately 30 MPa when tested at 28-day. In most design standards, total air content is specified based on aggregate size and exposure classes. Thus, aggregate size ranges from 10 to 20 mm is compared and the total air content is found to range from 4% to 8%.

Most design standards specify cover depth to the reinforcing steel in different ways for corrosion protection. For instance, cover depth and strength in EN 1992-1-1 and IS 456 increases with increase in severity of exposure, while SANS 10100-2 cover decreases with increase in strength. Moreover, most design standards, except CSA A23/1/23.2, does not provide cover requirements for freeze/thaw attack. In this case, cover depth requirements for corrosion are considered suitable for freeze/thaw attack.

In terms of performance, only CSA A23/1/23.2 evaluates the quality of concrete based on performance using standardized test methods such as ASTM C1202 *"Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*, ASTM C457 *"Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete"* and ASTM C1012 *"Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution"*. Table 3.21 presents relevant durability parameters for resistance to corrosion and freeze/thaw attack given in the design standards.

In terms of construction methods, design standards emphasise good practices with respect to placing, compaction, and curing conditions. Table 3.22 illustrates recommended values for temperature at time of delivery, minimum duration and condition of curing in each design standard. It can be shown that recommended values are almost similar, particularly for minimum duration of curing and temperature at delivery.

3.3.3 Comparison of limiting values for resistance to corrosion in various design standards with respect to exposure classes defined in EN 206-1

A comparison of relevant material compositions and proportions developed in different design standards is made considering the exposure classes in EN 206-1 as a basis. It is believed that EN 206-1 covers to a large extent major exposure classes responsible for corrosion of reinforcing steel, ranging from marine exposure towards the main land where carbonation dominates.

Table 3.23a and 3.23b given below, illustrates these comparisons. It can be shown that many design standards describe exposure to chloride from sea water with respect to the distance from the sea without giving classes as presented in EN 2061. For critical chloride exposure i.e. XS2 and XS3, w/cm ranging from 0.4 to 0.45 is specified, with 0.4 being common for class XS3 in many design standards. Requirements for exposure to chloride other than sea water are not described in many design standards, except in ACI 318 and CSA A23.1/23.2. In ACI 318, similar requirements given for chloride exposure from the sea water are presented for exposure to chloride other than sea water.

In terms of compressive strength, values from 30 to 50 MPa are specified for chloride exposure, with 35 MPa being a typical value for class XS3. Cover depth ranging from 45 to 75 mm is provided in most design standards for chloride exposure from sea water, with cover greater than 50 mm being suitable for critical exposure class i.e. XS3.

Exposure to carbonation is not described explicitly or mentioned in most design standards. Requirements presented for chloride exposure are conservatively expected to provide similar protection for carbonation.

Table 3.21: Comparison of limiting values for material compositions and proportions and cover depth for RC structures in various standards

Resistance of concrete to corrosion of reinforcement

	ACI 318: 2008			AS 3600: 2001/ AS 1379: 2007			CSA A23.1/23.2: 2009			EN 206-1: 2000/ EN 1992-1-1: 2004			IS 456 :2000	SANS 10100-2:09
Exposure class	C0	C1	C2	B1	B2	C	C-XL[#]	C1	C3	XS1	XS2	XS3	No class	No class
Maximum w/cm ratio	-	-	0.40	-	-	-	0.37	0.40	0.50	0.50	0.45	0.45	0.40- 0.45	-
Maximum cement content (kg/m^3)	-	-	-	-	-	-	-	-	-	300	320	340	320-360	-
Minimum strength, f_c (MPa)	17	17	35	32	40	50	50	35	30	C30/37	C35/45	C35/45	M30-M35	40 - 50
Minimum cover (mm)	-	> 50	> 50	40	45	50	75	75	75	45	50	55	45 - 75	60 - 65
Maximum chloride ion (% by mass of cement)	1.00	0.30	0.15	0.8 kg/m^3	0.8 kg/m^3	0.8 kg/m^3	0.15	0.15	0.15	0.2	0.2	0.2	0.6 kg/m^3	0.2
Chloride ion penetrability limits (coulombs) and age at test (days) using ASTM C1202	-	-	-	-	-	-	<1000 in 56d	<1500 in 56d	-	-	-	-	-	-

Resistance of concrete to freezing and thawing

	ACI 318: 2008			AS 3600: 2001/ AS 1379: 2007	CSA A23.1/23.2: 2009		EN 206-1: 2000/ EN 1992-1-1: 2004				IS 456 :2000	SANS 10100-2:09
Exposure class	F1	F2	F3	No designated class	F1	F2	XF1	XF2	XF3	XF4	No class	No class
Maximum w/cm ratio	0.45	0.45	0.45	-	0.50	0.55	0.55	0.55	0.50	0.45	-	0.53
Maximum cement content (kg/m^3)	-	-	-	-	-	-	300	300	320	340	-	-
Minimum strength, f_c (MPa)	31	31	31	32 occasionally 40 for frequently	30	25	C30/ 37	C25/ 30	C30/ 37	C30/ 37	-	-
Minimum cover (mm)	-	-	-	-	40	40	-	-	-	-	-	-
Minimum air content (%) of plastic concrete	5- 6	6-7.5	6-7.5	4 - 8	5 - 8	4 - 7	-	4	4	4	5±1	4 - 5
Maximum average spacing factor (mm) using ASTM C457	-	-	-	-	0.23	-	-	-	-	-	-	-

Notes:

1. [#]Concrete compressive strength at 56- day.

Table 3.22: Minimum curing duration, strength development and temperature for RC structures in various design standards

	ACI 318: 2008	**AS 3600: 2001**	**CSA A23.1/A23.2:2009**	**EN 206-1/EN 1992-1-1**	**IS 456: 2000**	**SANS 10100-2: 2009**
Concrete strength development	Maintain in moist conditions	Maintain in moist conditions	3d or until 40% f_c 7d and until 70% f_c	$f_{cm}2/f_{cm}28 > 0.5$	Maintain in moist conditions	$f_{cm}3/f_{cm}28 > 0.5$
Minimum days	7	3 or 7 depending on exposure class	3 for basic curing 7 for additional curing	1	7 for OPC concretes 10 for SCMs concretes	7
Temperature at time of delivery	> 4°C < 35°C	> 5°C < 35°C	> 10°C < 25°C	> 5°C < 35°C	> 15°C < 40°C	> 5°C < 35°C

Notes: f_c - Specified strength, f_{cm} -Mean compressive strength

Table 3.23a: Comparison of limiting values for corrosion resistance in RC structures with reference to exposure classes in EN 206-1

Exposure classification with reference to EN 206-1: 2000	Limiting values for resistance to corrosion of reinforcement														
	EN 206-1: 2000/EN 1992-1-1: 2004					CSA A23.1/23.2: 2009					SANS 10100-2: 2009				
	Max. w/c ratio	Min. cement kg/m^3	Min. strength class (MPa)	Max. Cl^- ion % by mass of cement	Min. Cover (mm)	Max. w/cm ratio	Min. cement kg/m^3	Min. strength (MPa)	Max. Cl^- ion % by mass of cement	Min. Cover (mm)	Max. w/cm ratio	Min. cement kg/m^3	Min. strength (MPa)	Max. Cl^- ion % by mass of cement	Min. cover (mm)
Corrosion induced by chlorides from sea water (or air carrying salt originating from sea water)															
XS1: Exposed to airborne salt but not in direct contact with sea water	0.50	300	C30/37	0.2	45	-	-	-	-	-	-	-	40	0.2	60
XS2: Permanently submerged	0.45	320	C35/45	0.2	50	0.50	-	30	0.15	75	-	-	-	0.2	-
XS3: Tidal, splash and spray zones	0.45	340	C35/45	0.2	55	0.40	-	35	0.15	75	-	-	40	0.2	65
Corrosion induced by chlorides other than from sea water (i.e. contact with water containing chlorides or de-icing salts)															
XDI: Moderate humidity	0.55	300	C30/37	0.2	45	-	-	-	-	-	-	-	-	-	-
XD2: Wet, rarely dry	0.55	300	C30/37	0.2	50	-	-	-	-	-	-	-	-	-	-
XD3: Cyclic wet and dry	0.45	320	C35/45	0.2	55	0.37	-	50	0.15	75	-	-	-	-	-
Corrosion induced by carbonation (exposure to air and moisture)															
XC1: Dry or permanently wet	0.65	260	C20/25	0.2	25	-	-	-	-	-	-	-	30	0.2	40
XC2: Wet, rarely dry	0.60	280	C25/30	0.2	35	-	-	-	-	-	-	-	30	0.2	45
XC3: Moderate humidity	0.55	280	C30/37	0.2	35	<0.40	-	-	0.15	< 50	-	-	30	0.2	40
XC4: Cyclic wet and dry	0.50	300	C30/37	0.2	50	-	-	-	-	-	-	-	-	-	-

Table 3.23b: Comparison of limiting values for corrosion resistance in RC structures with reference to exposure classes in EN 206-1

Exposure classification with reference to EN 206-1: 2000	Limiting values for resistance to corrosion of reinforcement														
	ACI 318: 2008					AS 3600: 2001/AS 1379: 2007					IS 456: 2000				
	Max. w/cm ratio	Min. cement kg/m^3	Min. strength (MPa)	Max. Cl^- ion % by mass of cement	Min. Cover (mm)	Max. w/cm ratio	Min. cement kg/m^3	Min. strength (MPa)	Max. chloride content	Min. Cover (mm)	Max. w/cm ratio	Min. cement kg/m^3	Min. strength (MPa)	Max. chloride content	Min. cover (mm)
Corrosion induced by chlorides from sea water (or air carrying salt originating from sea water)															
XS1: Exposed to airborne salt but not in direct contact with sea water	0.40	-	35	0.15	> 50	-	-	40	0.8kg/m^3	45	0.45	320	M30	0.6kg/m^3	45
XS2: Permanently submerged	0.40	-	35	0.15	> 50	-	-	40	0.8kg/m^3	45	0.45	320	M30	0.6kg/m^3	45
XS3: Tidal, splash and spray zones	0.40	-	35	0.15	> 50	-	-	50	0.8kg/m^3	50	0.40	360	M35	0.6kg/m^3	75
Corrosion induced by chlorides other than from sea water (i.e. contact with water containing chlorides or de-icing salts)															
XDI: Moderate humidity	0.40	-	35	0.15	> 50	-	-	-	-	-					
XD2: Wet, rarely dry	0.40	-	35	0.15	> 50	-	-	-	-	-					
XD3: Cyclic wet and dry	0.40	-	35	0.15	> 50	-	-	-	-	-					
Corrosion induced by carbonation (exposure to air and moisture)															
XC1: Dry or permanently wet	-	-	17	0.30	> 50	-	-	20	0.8kg/m^3	20	0.50	300	M25	0.6kg/m^3	30
XC2: Wet, rarely dry	-	-	17	0.30	> 50	-	-	-	-	-		-	-	-	-
XC3: Moderate humidity	-	-	17	0.30	> 50	-	-	-	-	-	0.50	300	M25	0.6kg/m^3	30
XC4: Cyclic wet and dry	-	-	17	0.30	> 50	-	-	32	0.8kg/m^3	40	0.45	320	M30	0.6kg/m^3	45

3.4 Summary of the main trend in the design standards

The review of the current state of practice in the design standards from selected countries discussed in preceding sections indicates several parameters that are commonly specified for durability purposes. They include (a) concrete materials i.e. cementititous material, aggregate types and size, mix proportions, (b) limits on deleterious substances, such as chlorides and sulfates, and (c) minimum compressive strength and cover depth to the reinforcing steel, with respect to exposure conditions. These parameters are specified based on the assumed minimum service life, e.g. 50 years according to EN 206-1. Experience demonstrates that compliance with these parameters only provide minimum quality for common structures. For special structures requiring long-term performance, especially in aggressive environment, more stringent parameters are imperative.

The method of specifying concrete based on w/cm and using compressive strength to characterize concrete for resistance to penetrability is commonly applied in the design standards. Although penetrability decreases with decreasing w/cm, a single parameter solely may not ensure penetrability since several other parameters including cementitious materials, aggregate content and size, and degree of hydration are involved. In this context, other measurements such as absorption, diffusion and permeability measurements should be employed in order to evaluate the quality of concrete against penetrability. Thus, pertinent sections in the design standards should undergo fundamental changes to include specifications that have a direct influence on the in-place performance of concrete.

Most design standards emphasise good construction practices, in particular for achieving adequate cover concrete requirements. Cover concrete is dictated by several factors such as placement, compaction and curing practices, therefore deviations are often observed although cover is normally checked and controlled prior to placing of concrete. The reason is, sometimes spacers may not provide appropriate tolerance over minimum required cover, or may occasionally be insufficient or wrongly placed causing steel to bend under the weight of concrete, or they may crush during concreting. Generally, good practices are not always achievable and therefore much emphasis should be given to correct materials selection with proper compatibility, appropriate exposure class, and by describing the procedures for achieving minimum specified cover depth.

Generally, prescriptive specifications in the design standards provide an indirect way towards durability and therefore the desired performance may or may not be described. Though several specified parameters are difficult or impossible to measure or verify, they may also conflict with the intended performance. These specifications are based on laboratory data and past experience therefore they cannot tell how long a structure will perform in service under selected exposure condition. In order to ensure adequate service life, a more rational approach based on performance that account for variability of material compositions and construction methods is imperative, something that most design standards do not provide.

The approach based on performance specification is considered as an alternative solution that may ensure durable RC structures. These specifications aimed at achieving relevant durability indicators that demonstrates the suitability of the concrete and its compositions with respect to the exposure conditions. They include PBS and an intermediate approach termed 'hybrid' specification. PBS is an integrated approach that links durability specifications, including durability indicators from relevant PTMs, and durability design through SLMs in order to estimate DSL of RC structures. In contrast, 'hybrid' specification relies on durability indicators chosen based on technical recommendations without explicitly given DSL. Using these specifications, the client and/or specifier could decide on the desired level of performance in a certain exposure condition and propose relevant durability indicators. On the other hand, the producer and/or contractor could develop desired concrete previously quantified and that satisfies the durability indicators limits set forth by the client and/or specifier.

Currently, there is no performance-based standard for concrete durability because of several existing challenges as will be discussed in the subsequent chapters. The performance requirements presented in few design standards are actually 'hybrid' specifications that still present some prescriptive requirements. A good example of 'hybrid' specifications document is CSA A23.1/23.2, where several performance limits and acceptance criteria are given.

In general, performance specifications seem relevant for solving many durability problems threaten the performance of RC structures. They are becoming more common in the construction industry as the behaviour of concrete during its life can be quantified prior to construction. These specifications promote innovations where the concrete producer and/or contractor can apply alternative techniques to produce durable concrete that meets the client's requirements.

CHAPTER 4: CURRENT STATE-OF-THE-ART OF PERFORMANCE SPECIFICATIONS

4.1 Introduction

The previous chapter introduced both prescriptive and performance specifications for concrete durability in various design standards. It concludes that most design standards are based on prescriptive requirements for concrete durability, with few having some elements of performance. This chapter gives a brief description of such performance elements in the design standards. It also describes the experience of performance specifications implemented in various project specifications.

Performance specifications can be described or defined in many different ways. Although the most useful definition is given in chapter one, there are also other definitions with similar meaning. These are described below:

- UK Highway Agency in "Developing performance specifications" (2003) defined a performance specification as follows:
  -"*Output measures* define the end product of works carried out on the network. This is usually in the form of a series of outputs that will deliver the desired outcome.
  -*Outcome measures* define the benefits that should be delivered as a consequence of the works carried out on the network. This will usually take the form of the level of service required".
- USA Federal Highway Administration (FHWA) in "Performance Specifications Strategic Roadmap" (2004) states that, "a performance specification defines the performance characteristics of the final product and links them to construction, materials, and other items under contractor control".
- The Canadian standard, CSA A23.1/23.2 (2009), states that "a performance concrete specification is a method of specifying a construction product in which the final outcome is given in mandatory language, in a manner that the performance requirements can be measured by accepted industry standards and methods. The processes, materials, or activities used by the contractors, subcontractors, manufacturers, and materials suppliers are then left to their discretion".

Several fundamental elements need to be analysed in order to determine whether a specification is performance-related or not. Several publications (FHWA, 2004; Carino et al.,

2010) have addressed some indicative requirements that describe performance for concrete durability. These requirements are outlined and shortly described below:

- *Quality characteristics and accountability*. Required quality characteristics should correspond with a reduction in prescriptive requirements while allowing innovative techniques to improve quality of production. Ideally, quality characteristics should be measurable and clearly tied to long-term performance. Moreover, stakeholders involved in the construction team should be held accountable only for those quality characteristics under their control.
- *Performance test methods (PTMs)*. Standard test methods or accepted reference standards that evaluate the quality characteristics of concrete in both fresh and hardened state should be defined. Moreover, the point of performance should be stated clearly i.e. when and where the tests should be performed, and who is responsible for testing. Some of the test methods might be difficult to implement because of cost, complexity, or lengthy time required in obtaining sufficient results. In this respect, surrogate test methods may be applied. Alternatively, a combination of materials of a known successful performance can be used.
- *Acceptance criteria*. The properties to be measured should clearly be defined with clear limits and if possible these properties should be statistically based. Whenever necessary, payment adjustments should be made depending on the acceptable levels of performance taking into account material and testing variability. In short, a specification should be able to determine whether specified parameters are met or not and the actions to be taken.

Performance specifications may be considered as an important link between design, construction, and long-term performance. They are considered as alternative specifications against predominantly prescriptive specifications, which have several shortcomings particularly on durability basis.

Currently, there is great interest worldwide towards performance specifications for concrete durability. In the following sections, an in-depth review of the efforts made towards performance specifications either in the design standards or in project specifications is presented. It is believed that the references obtained represent current state-of-the-art of performance specifications.

4.2 International Efforts on Performance Specifications for Concrete Durability

4.2.1 USA

4.2.1.1 ACI 318 (2008) *"Building Code Requirements for Structural Concrete"* contains some performance elements for concrete durability. Clause 4.5 of ACI 318 includes performance requirements for concrete exposed to sulfate conditions in accordance with ASTM C1012 *"Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution"*. The sulfate resistance limits and acceptance criteria provided are shown below.

→Moderate sulfate exposure class (*i.e.* S1) should have a maximum expansion less than 0.10% at 6 months.

→Severe sulfate exposure class (*i.e.* S2) should have a maximum expansion less than 0.05% at 6 months or less than 0.10% at 12 months.

→Very severe sulfate exposure class (*i.e.* S3) should have a maximum expansion less than 0.10% at 18 months.

The standard recommends hydraulic cements to conform to ASTM C1157 *"Standard Performance Specification for Hydraulic Cement"*, which covers cements for both general and special application. This standard classifies cements based on specific performance requirements including general use, high-early strength, heat of hydration and sulfate resistance. For specific details, reference should be made to ASTM C1157.

In its commentary, ACI 318 present ASTM C1202 *"Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration"* as a performance test method that evaluates concrete in exposure conditions requiring low permeability. However, there are no recommended limits or acceptance criteria presented.

4.2.1.2 Carino et al., (2010), "Report on performance-based requirements for concrete" discusses approaches for developing and implementing performance requirements as an alternative to predominantly prescriptive requirements in the design standards. In the report, essential elements of performance specifications including quality characteristics, test methods, and acceptance criteria are discussed. Alternative performance requirements are proposed for the prescriptive durability requirements in Table 4.3.1 of ACI 318. Specific details regarding proposed performance alternative may be obtained in the Phase II report (*i.e.* Guide to specifying concrete performance) developed by Bickley et al. (2008). For the purpose of discussion, Table 4.1 illustrates the proposed performance alternative in ACI 318 as recommended by Carino et al. (2010).

From Table 4.1, it should be noted that ASTM C1202 is used as a PTM for controlling penetrability for all exposure categories. Pre-qualification tests are used only for sulfate resistance, while field verification tests are required in addition to pre-qualification tests for freeze/thaw resistance.

Table 4.1: Alternative performance requirements in ACI 318 (Bickley et al., 2008)

Exposure category	**Exposure class**	**Prescriptive alternative for resistance to penetrability**		**Performance alternative for resistance to penetrability ASTM C1202 (Coulombs)**	**Performance alternative for specific exposure class**
		Max w/cm	**Min f_c (MPa)**		
Freezing and thawing	F0	-	-	-	-
	F1	0.45	31	2000	ASTM C666 durability factor ≥ 80% ASTM C457 Spacing factor ≤ 0.2 mm Air content ≥ 3.0%
	F2	0.45	31	2000	ASTM C666 durability factor ≥ 85% ASTM C457 Spacing factor ≤0.2 mm Air content ≥ 3.0%
	F3	0.45	31	2000	ASTM C666 durability factor ≥ 90% ASTM C457 Spacing factor ≤0.2 mm Air content ≥ 3.0%
Sulfate	S0	-	-	-	-
	S1	0.50	28	2500	ASTM C1012 0.05% at 6 months
	S2	0.45	31	2000	ASTM C1012 0.05% at 6 month or 0.10% at 12 months
	S3	0.45	31	2000	ASTM C1012 0.10% @ 18 months
Corrosion	C0	-	-	-	-
	C1	-	-	-	
	C2	0.40	35	1500	
Low permeability	P0	-	-	-	-
	P1	0.50	28	2500	

4.2.1.3 Bognacki et al. (2010), "Rapid chloride permeability testing's suitability for use in performance-based specifications: Concerns about variability can be mitigated", reported positive results with the experience of performance specifications developed in the Port

Authority of New York and New Jersey (PANYNJ). PANYNJ's specifications considered compressive strength, air and water content, and chloride permeability in accordance with ASTM C1202 for acceptance criteria. A survey carried out for PANYNJ's projects indicated that approximately 11% of the projects did not meet specified requirements, resulting in reduced payment, while 89% of the projects received full payment. Of this 89% projects, 35% of the projects exceeded the minimum requirements and resulted in bonus payments. In this regard, the paper recommends ASTM C1202 application in project specifications for acceptance criteria despite its variability. It argued that if appropriate limits could be specified, the concern about the variability can be eliminated. The paper concluded that with well-written specifications, quality concrete can be produced and placed where producers and/or contractors could be rewarded for quality production and discouraged from producing deficient concrete.

4.2.1.4 Sprinkel (2004), "Performance specification for High Performance Concrete overlays on bridges: Final report", described the experience gained by the Virginia Department of Transportation (VDOT) with the implementation of performance specifications for the construction of HPC overlay. In the report, payment adjustments were made based on measurements for compressive strength, air content, bond strength, and chloride permeability in accordance with ASTM C1202. Generally, VDOT specifications required chloride values of less than 1000 Coulombs at 28 days as acceptance criteria. Table 4.2 illustrates a typical example of payment adjustments based on the contract unit price for the concrete overlay.

Table 4.2: Example of payment adjustments for concrete overlay (Sprinkel, 2004)

Average percentage with limits (PWL), %	**Action**
91 to 100	Bonus payment
85 to 90	Payment at contract unit price
55 to 84	Payment reduction at contract unit price
< 55	Accept at 50% of the contract unit price or overlay removal at no cost

According to the report, application of performance specifications for HPC concrete overlay contracts showed successful performance when compared to the previous contracts with prescriptive specifications. In this regard, VDOT recommended application of performance specifications for future bridge overlay projects.

4.2.2 Canada

4.2.2.1 CSA A23.1/A23.2 (2009) *"Concrete materials and methods of concrete construction/Test methods and standard practices for concrete"* offers two options for the specification of concrete, i.e. performance and prescriptive. In the standard, performance specifications may be selected "when the owner requires the concrete supplier to assume responsibility for performance of the concrete as delivered and the contractor to assume responsibility for the concrete in-place". The prescriptive specification is selected "when the owner assumes responsibility for the concrete". Table 5 of CSA A23.1/A23.2, reproduced as Table 4.3, provides these options, where the roles and responsibilities of various parties is indicated.

Table 4.3: Alternative methods for specifying concrete in CSA A23.1/23.2

Alternative	The owner shall specify	The contractor shall	The supplier shall
(1) Performance: When the owner requires the concrete supplier to assume responsibility for performance of the concrete as delivered and the contractor to assume responsibility for the concrete in place	**(a)** required structural criteria, including strength at age; **(b)** required durability criteria, including class of exposure; **(c)** additional criteria for durability, volume stability, architectural requirements, sustainability, and any additional owner performance, pre-qualification, or verification criteria; **(d)** quality management requirements (see Annex J); **(e)** whether the concrete supplier shall meet certification requirements of concrete industry certification programs; and **(f)** any other properties that might be required to meet the owner's performance criteria	**(a)** work with the supplier to establish the concrete mix properties to meet performance criteria for plastic and hardened concrete, considering the contractor's criteria for construction and placement and the owner's performance criteria; **(b)** submit documentation demonstrating the owner's pre-qualification performance requirements have been met; and **(c)** prepare and implement a quality control plan to ensure that the owner's performance criteria will be met and submit documentation demonstrating the owner's performance requirements have been met	**(a)** certify that the plant, equipment, and all materials to be used in the concrete comply with the requirements of this Standard; **(b)** certify that the mix design satisfies the requirements of this Standard; **(c)** certify that production and delivery of concrete will meet the requirements of this Standard; **(d)** certify that the concrete complies with the performance criteria specified; **(e)** prepare and implement a quality control plan to ensure that the owner's and contractor's performance requirements will be met, if required; **(f)** provide documentation verifying that the concrete supplier meets industry certification requirements, if specified; and **(g)** submit documentation to the satisfaction of the owner, demonstrating that the proposed mix design will achieve the required strength, durability, and performance requirements
(2) Prescription: When the owner assumes responsibility for the concrete	**(a)** mix proportions, including the quantities of any or all materials (i.e., admixtures, aggregates, cementing materials, and water) by mass per m^3 of concrete; **(b)** the range of air content; **(c)** the slump range; **(d)** use of a concrete quality plan, if required; and **(e)** other requirements	**(a)** plan the construction methods based on the owner's mix proportions and parameters; **(b)** obtain approval from the owner for any deviation from the specified mix design or parameters; and **(c)** identify to the owner any anticipated problems or deficiencies with the mix parameters related to construction	**(a)** provide verification that the plant, equipment, and all materials to be used in the concrete comply with the requirements of this Standard; **(b)** demonstrate that the concrete complies with the prescriptive criteria as supplied by the owner; and **(c)** identify to the contractor any anticipated problems or deficiencies with the mix parameters related to construction

Annex J of CSA A23.1/A23.2 provides a brief description and guidance on selecting either prescriptive or performance when specifying and ordering concrete in accordance to Table 4.3. This section describes the performance option presented in Annex J of CSA A23.1/A23.2, in summary, with respect to its achievement in enhancing performance. Key elements discussed in the Annex are described below.

(a) *Definitions*: Annex J set out key terms and definitions that should be considered when specifying concrete for performance. Some of the terms and definitions include prescriptive and performance specifications, quality control, quality control plan and quality assurance.

(b) *Performance criteria*: Annex J set requirements for concrete in both fresh and hardened states. In terms of fresh state, essential criteria considered include workability, uniformity, placeability, setting time, etc. These criteria are of the interest to the concrete producer and/or contractor. In case of hardened state, essential criteria are strengths and rate of strength development, durability in the expected exposure conditions, and other criteria including aesthetics, surface texture, etc. Most of these criteria are of the interest to the specifier and/or owner, but in some cases the concrete producer and/or contactor may also be interested in these criteria. An important element to consider is the performance requirements that should be satisfied and measured by acceptable industry standards.

(c) *Roles and responsibilities*: Describes the roles and responsibilities of each stakeholder involved in the construction industry. It also describes the coordination between them in order to ensure that the final product meets the performance criteria and that the quality control processes are compatible and demonstrate compliance.

(d) *Guidance on selecting an alternative specification*: Describes the owner's risks involved in selecting either of the specification. When prescriptive requirements are selected, the owner should make decisions in order to balance between capital investment and long-term maintenance costs. In this regard, the owner, through the consultant, takes responsibility to ensure materials and methods prescribed are properly followed. In performance basis, the owner prescribes performance requirements and relies on the concrete contractor and/or producer to provide materials and methods to achieve the required performance. In performance basis, issues related to quality management, component of specifications and verification process are considered imperative.

Importantly, CSA A23.1/A23.2 includes various test methods with well-developed limits for acceptance for evaluating performance of *in-situ* concrete. These are given below.

(a) Clause 4.1.1.10.3 of CSA A23.1/A23.2 recommends ASTM C1202 for determining chloride permeability for extreme chloride exposure. The following Coulomb values should be obtained for acceptance criteria:

→Exposure class C-XL, a maximum average of 1000 Coulombs or less at 56-days of age should be obtained, with no single value greater than 1250 Coulombs.

→Exposure class C-1, a maximum average of 1500 Coulombs or less at 56-days of age should be obtained, with no single value greater than 1750 Coulombs.

(b) Clause 4.3.3.3 of CSA A23.1/A23.2 recommends ASTM C457 *"Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete"*, for determining air-void system for concrete exposed to freeze/thaw attack i.e. class F-1, C-XL, and C-1. The following requirements should be obtained:

→Average spacing factor less than 0.23 mm, with no single test greater than 0.26 mm

→For concrete with w/cm of 0.36 or less, the average spacing factor should be less than 0.25 mm, with no single test greater than 0.30 mm.

(c) Table 3 of CSA A23.1/A23.2 provides limits and acceptance criteria in accordance with ASTM C1012 for concrete exposed to sulfate attack. The performance requirements are included below:

→Sulfate exposure class S-1 (*i.e.* very severe) and class S-2 (*i.e.* severe) should have a maximum expansion less than 0.05% at 6 months or 0.10% at 12 months.

→Sulfate exposure class S-3 (*i.e.* moderate) should have a maximum expansion less than 0.10% at 6 months.

4.2.2.2 Ministry of Transportation Ontario (MTO). The Ontario Provincial Standard Specification, OPSS.PROV 1350 (2010), is a provincial-oriented specification covering performance specifications with respect to material requirements and methods for proportioning, test methods, acceptance criteria, and payment adjustments for normal structural concrete and HPC. In the specification, parameters such as compressive strength, air-void system, and chloride permeability are tested using relevant test methods. MTO specifications also include a penalty-bonus system in order to ensure compliance and consistency of measured parameters. Table 4.4 illustrates a typical example of penalty-bonus provisions developed by MTO specifications for normal structural concrete.

Table 4.4: Example of payment adjustments for normal concrete (OPSS.PROV 1350, 2010)

Measured parameters	**Action**
Air Void System in accordance with ASTM C457	
Air content > 3.0 %, and Spacing factor of 0.23 mm or less	Full payment or Bonus payment based on a combination of two measured parameters
Air content < 3.0 % or Spacing factor > 0.23 mm	Owner may require removal of the concrete or keep the concrete in place at a reduced payment.
Rapid Chloride Permeability (RCP) in accordance with ASTM C1202	
Average Coulombs ≤ 1000	Fully payment, no bonus available
Average Coulombs > 1000 and < 2000	Accepted with a price reduction
Average Coulombs > 2000	Unacceptable and shall be removed and replaced at the contractor's expense

4.2.3 Europe

4.2.3.1 EN 206-1 (2000) *"Concrete – Part 1: Specification, Performance, Production and Conformity"* considered the use of performance specifications for concrete durability prior its publication. Notwithstanding this the committee CEN/TC 104 decided to implement prescriptive specifications based on the argument that the test methods were at that time not yet sufficiently developed to be detailed in the standard. However, clause 5.3.3 of EN 206-1 permits the use of performance requirements for countries which have sufficiently developed confidence with the test methods and acceptance criteria.

Clause 5.2.5.3 of EN 206-1 presents the equivalent performance concept that permits the modifications of the prescriptive requirements for minimum cement content and maximum w/cm. It is applicable to a specific combination of cement plus SCMs as long as the concrete provides equivalent performance regarding the durability when compared to the reference concrete for the same exposure class.

Annex J of EN 206-1, "Performance-related design methods with respect to durability" claims to provide guidance regarding performance specifications for concrete durability. However, no guidance on how to choose performance requirements and criteria seems to be presented. It only provides a definition of performance methods and guidelines or when this option should be considered. Annex J may be considered as a good summary since it provides guidance for many parameters to be considered when developing performance specifications.

4.2.3.2 BS 8500-1 (2006) *"Concrete – Complementary British Standard to EN 206-1 – Part 1: Method of specifying and guidance for the specifier"* offers five approaches to the specifier for the specifications of concrete. These approaches are shortly described below:

(i) *Designated Concrete*: For specific uses. An essential element of this approach is that the concrete producer is responsible for production control and issues certification proof of conformity criteria. It is only applicable where the third-part certification is selected as an option of specifying the concrete. This option can be used in a wide range of applications, commonly for building works.

(ii) *Designed Concrete*: Offers more flexibility to the specifier than designated concrete. It covers every application and therefore considered as an alternative to designated concrete. Using the intended service life and the minimum cover depth, the limiting values of composition are determined for each of the identified exposure classes.

(iii) *Prescribed Concrete*: This approach allows the specifier to prescribe the exact composition and constituents of the concrete. This excludes compressive strength requirements or any other performance requirements, and so this option has limited application.

(iv) *Standardized prescribed concrete*: This approach is appropriate where concrete is site-batched on a small site or obtained from a ready mixed concrete producer who does not have accredited third-party certification.

(v) *Proprietary concrete*: This approach is appropriate where it is required that the concrete achieves a performance, using defined test methods, outside the normal performance requirements for concrete. Proprietary concrete is selected in consultation with the concrete producer and the project specification.

The approaches described are very complex for use in determining and ordering concrete and therefore a specialized computer program is recommended in some cases (Harrison, 2003). In order to understand the above approaches, reference should be made to BS 8500-1, Introduction, for further clarification.

4.2.3.3 Becker and Foliente (2005), in a report "Performance Based Building Thematic Network, PBB International State-of-The-Art, PeBBu 2nd International SotA report", which involved more than 70 international organisations aiming at facilitating and enhancing the existing performance-based building (PBB) research and activities. PBB is a building market environment in which all stakeholders in the various phases of building process address the need to ensure performance-in-use of buildings as an explicit target. PBB is expected to facilitate the development and introduction of innovative techniques to enhance the overall quality of buildings. Its implementation can be achieved by using performance-based procedures in design and construction but may also include more convectional procedures that are based on approved prescriptive provisions, which are know to supply given levels of performance.

According to the report, it is expected that an integral application of PBB in the long-term may result in overall cost savings of about 25% as compared to traditional modes of construction. Based on several advantages, PBB concepts have been adopted in some regulatory frameworks such as the European New Approach and the accompanying CPD Directive, the Building Code of Australia and New-Zealand, and are in the process of being implemented in the Canadian Building Code. Moreover, some basic concepts of PBB have been formulated in ISO SC 03, ISO SC 15, and ASTM E06, focusing on the delineation of performance attributes and user needs, methodological aspects of performance requirements derivation, and preparation of performance-based design briefs.

In reality, the concept behind PBB is similar to performance-based specification (PBS) concept that covers every concrete construction. However, PBB is specifically focused on the performance of buildings only and not the performance of general concrete construction e.g. long-span bridges, bored tunnels, or any special structure. For PBB, issues related to procurement, contracting, delivery, management and maintenance of buildings are essential elements considered.

4.2.3.4 UK Highway Agency (2003), "Developing performance specifications", describes the key issues involved in developing performance specification and presents three strategic options, which are: (a) developing existing specifications, where the current mix of prescriptive and output-based specification is retained, but with greater emphasis on performance through innovation, (b) performance specification for maintenance only, where specifications are expanded to include design and delivery of major maintenance and renewal of networks assets, and (c) full performance specifications, where more freedom for the suppliers to implement innovative solutions is expected in addition to risk transfer between the agency and the suppliers.

4.2.4 Australia

4.2.4.1 AS 1379 (2007) *"Specification and Supply of Concrete"* includes two classes of concrete namely, 'Normal Class' and 'Special Class' concrete. Several requirements for 'Normal Class' concrete that should incorporate other requirements in AS 3600 were described in previous chapter i.e. chapter 3. This chapter only concentrates on 'Special Class' concrete as it is specified either in prescriptive or performance when ordering concrete. According to AS 1379, 'Special Class' concrete refers to concrete that require additional or different characteristics from 'Normal Class' concrete that may not be capable of being produced at all plants or locations in Australia. Clause 1.5.4 of AS 1379 states that "when concrete is specified as 'Special Class' and any property other than strength grade is specified as the principal criterion, or the proportions of the mix are specified, it shall be designated by an appropriate alphanumeric code, agreed between the supplier and the customer to indicate the criterion".

Appendix B of AS 1379 claims to provide guidance to the specifications of 'Special Class' concrete. In reality it does not, but rather indicates some of the parameters to be specified and describes the consequences when 'Special Class' concrete is selected. Importantly, Appendix C of AS 1379 describes the means of compliance with the requirements presented in the standard. The following includes the means by which compliance with the standard can be demonstrated by the manufacture and/or supplier: (a) statistical sampling that enables decisions to be made about the quality of product after inspection or testing, (b) product certification that gives independent assurance of the claim by the manufacture and/or supplier that products comply with the stated standard, (c) manufacturer and/or supplier's quality management system that may provide necessary confidence that the specified requirements will be met, and (d) other means of assessment that demonstrate compliance with the standard in case other means are found inappropriate. This generally may refer to manufacture and/or supplier's guarantee of product conformance.

4.2.4.2 Day (2005), "Perspective on Prescriptions" cites several examples showing successful achievement of Australian specifications in producing quality concrete inside and outside Australia. According to Day, current performance specifications (*i.e.* AS 1379) in Australia represent a win-win situation when compared to prescriptive specifications. For all stakeholders in the concrete industry the following can be gained.

(i) The producer gets more freedom with regard to materials selection to produce economical mixture proportions at a smaller control margin, as the specification does not place any restriction.

(ii) The producer can provide a wide range of test data on heat generation, shrinkage, pumpability, and other properties relevant to the contractor.

(iii) The owner gets a cost effective, uniform, and reliable concrete with no effort or difficult decisions.

4.2.5 India

4.2.5.1 Kulkarni (2009), "Exposure classes for designing durable concrete" suggests several changes to the Indian Standard (IS 456, 2000), in particular for the existing definitions of exposure classes, as one of the ways to address durability requirements in India. The proposed definitions of exposure classes have been widened and made more rational by aligning them to the anticipated degradation mechanisms. Limiting values corresponding to the proposed exposure classes are also addressed although they need to be validated with systematic laboratory work.

4.2.5.2 Santhanam (2010), "Evolving performance specifications for concrete construction in India" analyses durability provisions in the Indian Standard (IS 456, 2000) and highlights its shortcomings. In addition, a way forward towards performance specifications for concrete durability in India is proposed. This includes four steps which are: (a) background work, which evaluates durability provisions in IS 456, (b) preparation of distress map for different geographical locations across India, (c) testing methods for performance specifications, and (d) final analysis and white paper on durability specifications and beginning of field implementation. However, the final step would be revised based on the inputs from the field after initial implementation.

4.2.6 South Africa

4.2.6.1 Alexander et al. (2008), "A framework for the use of Durability Indexes in performance-based design and specifications for reinforced concrete structures", presents a framework for the use of DI tests towards developing performance specifications in South Africa. Steps to establish an integrated approach that incorporates appropriate DI values and SLMs to estimate DSL of RC structure are described. Procedures for implementing DI values as a quality control measure are also recommended. Though the paper proposed DI limits to be implemented in the specifications, it recommends the values to be used developmentally as more field data are still required for validation. Similar work regarding development of appropriate DI values to be implemented in the specifications when improved SLMs become available was done by Alexander and Beushausen (2009). Similarly, recommendations are given towards generating correlations between DI values and actual structural performance over time.

4.2.6.2 Alexander (2009), "Developments in South African code provisions for concrete durability", is a currently unpublished paper primarily concerned with the need to redraft the South African Standard i.e. SANS 10100-2 to address durability requirements. It examines current durability provisions in USA, European, and South African standards in general terms and concludes that they are all prescriptive. It discussed current developments towards redrafting South African Standard to include durability requirements. A proposal is given towards implementing hybrid specifications in lieu of prescriptive requirements or actual performance-based specifications.

4.2.6.3 Alexander and Stanish (2005), "Durability design and specification of reinforced concrete structures using multi-factor approach", describes multi-factor approach to achieve durability of RC structures. The approach considered issues related to environment, design and materials. The paper provides seven steps towards performance specifications:

1. "Define exposure classes related to the mechanism(s) of deterioration
2. Derive a quantitative design methodology, including definition of end of design life
3. Develop test methods that relate to the input parameters of the design method
4. Produce provisional conformity criteria and calibrate against traditional solutions
5. Establish limitations of test applicability
6. Ensure production control and acceptance testing
7. Conduct full-scale trials and long-term monitoring to confirm conformity requirements."

Despite the fact that the concept behind SLMs is considered the best approach, it is found to be too sophisticated for most specifiers and therefore the paper recommends a deemed-to-satisfy approach.

4.2.6.4 The South African National Road Agency Limited (SANRAL) (2010) has adopted the use of performance specifications in constructing and improving concrete structures of the road network in South Africa. It has been actively use the DI tests for oxygen permeability, water sorptivity and chloride conductivity for improving the quality of national infrastructure. SANRAL describes acceptance criteria for strength and durability requirements, which involve full acceptance, condition acceptance and rejection. On attaining the specified limiting value for strength and DI values, full acceptance and payment is made. Failure to comply with the limiting values may result in condition acceptance with reduced payment or rejection where the contractor is required to remove the concrete at own expense. Table A.3 in the Appendix illustrates acceptance criteria developed by SANRAL for durability with respect to oxygen permeability and cover depth requirements.

4.3 Summary of main trends on Performance Specifications

The review of the current state-of-the-art on performance specifications from selected countries in the preceding sections indicates an interest towards performance specifications for concrete durability. Several highway agencies in USA, Canada, and South Africa have applied performance specifications in various projects and the results are encouraging. Despite such interest, few design standards have successful implemented performance specifications termed 'hybrid' for concrete durability. A good example is CSA A23.1/23.2 that specifies compressive strength and measurements related to various durability indicators through PTMs such as ASTM C1202, ASTM C457 and ASTM C1012.

In hybrid specifications, owner and/or specifier could decide on the desired level of performance in a certain exposure condition and propose appropriate durability indicators using relevant test methods. The concrete producer and/or contractor could develop desired concrete previously qualified and that satisfies durability indicator set forth by the owner and/or specifier. These specifications may be considered as an efficient and relatively simple way to extend the service life of RC structures at present where several barriers exists in implementing PBS. PBS is an integrated approach that links durability specifications, including durability indicators from relevant PTMs, and durability design through SLMs in order to estimate DSL of RC structures. This is the focus of the current research worldwide and is expected to be a powerful tool for evaluating concrete performance for durability.

There are numerous challenges confronting full implementation of PBS which need to be addressed in current research of deterioration modelling. These include (a) lack of reliable, consistent and standardized test methods which can evaluate concrete performance in time or routinely, (b) lack of adequate SLMs which can capture the main aspects involved in deterioration including environment-specific factors, and (c) lack of experience in developing sufficient performance requirements with appropriate acceptance criteria.

CHAPTER 5: PERFORMANCE TESTING AND SERVICE LIFE MODELS

5.1 Introduction

The previous chapter presented the international efforts on performance specifications for concrete durability. It described the efforts made and challenges involved towards performance specification implementation in the design standards. Among the issues confronting performance specification implementation is Performance Test Methods (PTMs). These are an important link to consider prior other elements such as acceptance criteria, payment adjustment factors and life cycle cost analysis. This chapter therefore presents various PTMs developed in different countries for assessing the performance of concrete for durability. It focuses on the test methods either implemented in the design standards or in project specifications. Detailed descriptions regarding guidelines, calibration, and sample preparation are beyond the scope of this chapter. However, key elements regarding suitability and limitations of individual test methods are considered vital and therefore discussed in this chapter.

As described by the Concrete Society (CS 109, 1996), PTMs may be defined as:

- Tests that directly assess the resistance of concrete to a standardized deterioration process, e.g. freeze/thaw attack, sulfate attack, carbonation and chloride process, or
- Tests that directly assess material parameters such as temperature, air content etc., which in turn, impart resistance to concrete to withstand environmental actions.

In the South African context, PTMs can further be described as tests that directly assess the quality of the cover layer using parameters which are linked with transport mechanisms that influence deterioration processes (Alexander and Stanish, 2005). These parameters are regarded as 'Durability Indexes'.

PTMs can be grouped into two categories as described below, each of which plays a significant role.

(i) Pre-qualification tests on concrete mixtures:

These tests are mostly used in the design standards because of their simplicity in obtaining results, relatively low cost and higher precision levels. These tests do not account for material or construction variability and therefore they may not indicate whether the as-built concrete structure complies with the performance requirements or not. Many of these tests are not durability tests though they may be used as an indicator of fluctuating quality. Some of these tests include slump, air content and strength tests.

(ii) As built quality control test methods, including *in-situ* tests:

These tests are used to verify key characteristics of concrete that relate to the desired performance. They may also be used to verify whether the as-built concrete structure is similar to the pre-qualified mixture. They may be carried out directly *in-situ* on the structure, or on samples derived from actual construction. These tests are now considered as an important link in addition to the pre-qualification tests. Few design standards have succeeded to implement these tests with suitable limits for acceptance. Commonly quality control tests include rapid chloride permeability, rapid migration, sorptivity, bulk diffusion tests, etc.

Ideally, PTMs intend to provide durability indicators that would be used as inputs into SLMs as an integrated approach towards durability design and specification termed ‘performance-based specifications’ (PBS). The link between PTMs and SLMs is vital towards durability design and specification. It gives the designers and/or specifiers of RC structures the confidence to make robust durability predictions. Since SLMs are increasing and gained acceptance in the construction industry, it is important to highlight their relevance in estimating DSL and the challenges confronting their applications. This chapter also describe some of SLMs currently exist in later sections, with respect to their strengths and limitations.

The following sections include a review of various PTMs used for evaluating performance of concrete for durability. Since most performance specifications are based on a predominant transport mechanism, the focus is limited to the test methods that relate to penetrability.

5.2 Performance Test Methods for Resistance of Concrete to Penetration

By measuring penetrability properties of concrete such as permeability, absorption and diffusion, the resistance of concrete against the ingress of deleterious substances can be assessed. In this way, standard penetrability tests that measure the rate of penetration or related indices is imperative to the development of performance specifications. Several of these tests exist with some having poor precision and which take time to give results, while others do not relate directly to the field performance (Stephen et al., 2010). The following sections present common PTMs that evaluate the resistance of concrete with respect to diffusion, absorption and permeability as the main transport mechanisms.

5.2.1 Performance Test Methods for Chloride Resistance

Several PTMs are available for assessing the resistance of concrete to chloride ingress, especially in marine and de-icing salt conditions where corrosion of reinforcing steel is critical. These tests may provide either a chloride diffusion coefficient or an index of the resistance of concrete to chloride penetrability. Such values can be used as inputs to SLM for estimating DSL of RC structures.

The most common PTMs used for determining the resistance of concrete to chloride ingress include ASTM C1202 *"Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration"* and ASTM C1556 *"Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures.*

In ASTM C1202, a 60-volt DC potential is applied through a water-saturated concrete specimen over a 6-hour period to obtain the charge passed in Coulombs. The specimen with nominal diameter of 100 mm and thickness of 50 mm is placed between two cells containing sodium chloride (NaCl) and sodium hydroxide (NaOH) solutions. Schematically this is shown in Figure 5.1.

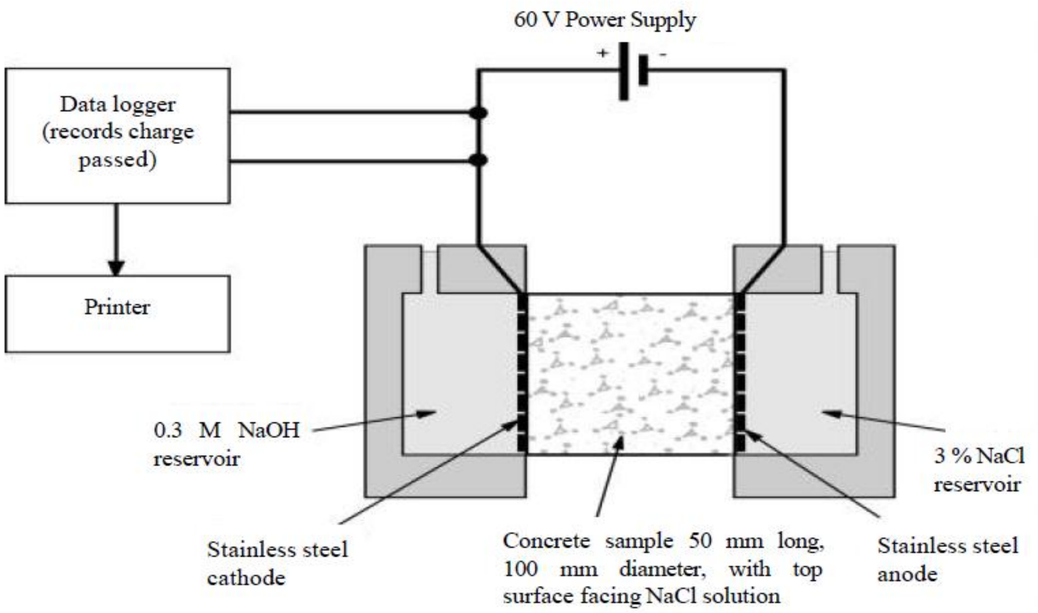

Figure 5.1: Schematic of ASTM C1202 test setup (Stanish et al., 1997)

ASTM C1202 measures neither permeability nor migration but is really a measure of concrete resistivity. It evaluates the electrical conductance of concrete by providing an indication of resistance of concrete to chloride ion penetration as the total charge passed.

Concrete is classified from negligible to high based on total charge that passed through the specimens as shown in Table 5.1.

Table 5.1: Chloride ion penetrability based on charge passed (ASTM C1202)

Charge passed (Coulombs)	Chloride ion penetrability
> 4000	High
2000-4000	Moderate
1000-2000	Low
100-1000	Very low
< 100	Negligible

Although ASTM C1202 has been adopted as a standard test, there have been several criticisms regarding its applicability (Andrade, 1993; Yang, 2004; Nanukuttan et al., 2007). The major criticisms include (a) the current passed is related to all ions in the pore solution, not just chloride ions, (b) the measurements are made before steady state migration is achieved, and (c) the high voltage leads to an increase in temperature, especially for low quality concretes. Moreover, research indicated that the presence of admixtures containing ionic salts, e.g. calcium nitrite in concrete, may affect the results obtained (Obla and Lobo, 2007). All these criticisms lead to loss of confidence with the test method for measuring chloride ion permeability. Despite the strong criticisms, this test method has shown a fair correlation between concrete resistivity and concrete permeability (Trinh Cao and Meck, 1996) with lower Coulomb values indicating greater resistance to chloride ingress.

ASTM C1556 bulk diffusion test measures the apparent chloride diffusion coefficient i.e. the rate of chloride ingress into concrete. Specimens taken from cores or cast cylinders are sealed on all faces except one and kept saturated in calcium hydroxide solution. The specimen is then placed in sodium chloride solution for at least 35 days where chloride ions enter the specimen by a diffusion process. After exposure, the surface is ground or milled off in a series of thin layers each about 1-2 mm thick as shown in Figure 5.2. Each of the powdered layers is collected separately and analyzed for acid-soluble chloride content. The chloride content is plotted as a function of depth, and a numeric solution to Fick's 2^{nd} law is fitted to the data to determine the apparent diffusion coefficient and chloride surface concentration.

Despite the fact that ASTM C1556 may provide inputs for SLMs, is difficult to carry out and rather involved taking about 3 to 4 months to obtain results (Lee and Chisholm, 2005). Moreover, grinding thin layers is very delicate and may lead to high variability of the test results. This test is considered unsuitable for mix development and quality control, as it is expensive and cannot be performed by most commercial laboratories. However, it is considered more suitable for research purposes than for routine application by the industry (Obla et al., 2005). Table 5.2 summarises the individual test methods for chloride resistance with respect to their strengths and weaknesses.

Figure 5.2: Milling ASTM C1556 specimen in 1 mm steps (Stephen et al., 2010)

Table 5.2: Performance test methods for resistance to chloride penetrability

Test Method	Duration to complete testing	Comments
RCPT ASTM C1202	28-56 days	-Several variability may result from using SCMs, corrosion inhibiting admixture *e.g.* calcium nitrate, or due to pore solution conductivity, temperature, and cover concrete thickness, which may affect test results. -Does not relate directly to corrosion activity. -Suitable for monitoring the consistency of a single mixture, but unsuitable for comparing mixtures with different binders. -Widely used as a quality control test since it provides a rapid index that correlates with chloride ion penetrability, and it is repeatable and reproducible as well.
ASTM C1556	90 days	-Suitable more for research purposes than routinely use by the industry due to complexity, cost and time. -Suitable for relative comparison of concrete mixtures. -Not suitable for *in-situ* concrete that has been exposed to chlorides. -Until experience is gained by more commercial labs, it is not recommended in the specifications for pre-qualification or quality control purposes.

5.2.2 Performance Test Methods for Measurement of the Rate of Absorption

Absorption tests have not been used so commonly to measure chloride resistance of concrete possibly because they are not a good indicator of chloride resistance. However, they have been used as a measure for durability in certain elements such as concrete pipes which require low penetrability. ASTM C1585 *"Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes"*, AS 1012.21 *"Methods of testing concrete - Determination of water absorption and apparent volume of permeable voids in hardened concrete"*, and Initial Surface Absorption Test (ISAT) in accordance with BS 1881: Part 208 (1996), are each a measure of absorption rate of concrete.

ASTM C1585 determines the rate of absorption of water indirectly through the mass gain in unsaturated concrete specimen over time. The typical specimen size is 100 mm diameter and 50 mm thick, cured in an oven for 3 days, at a temperature of 50°C and internal relative humidity of 80%. The specimen is then sealed in a container and stored at 23°C for 15 days to allow the internal relative humidity of the specimen to come to equilibrium. The circular sides of the specimen are sealed to ensure uni-directional absorption from the bottom face which is uncovered while the top face is covered to reduce evaporation. Schematically this is shown in Figure 5.3. The specimen is removed from the water at increasing time intervals and re-weighed to determine the change in mass over time. Based on the observations, mass of water is plotted against the square root of time and the rate of water absorption is determined from the slope of the straight line produced.

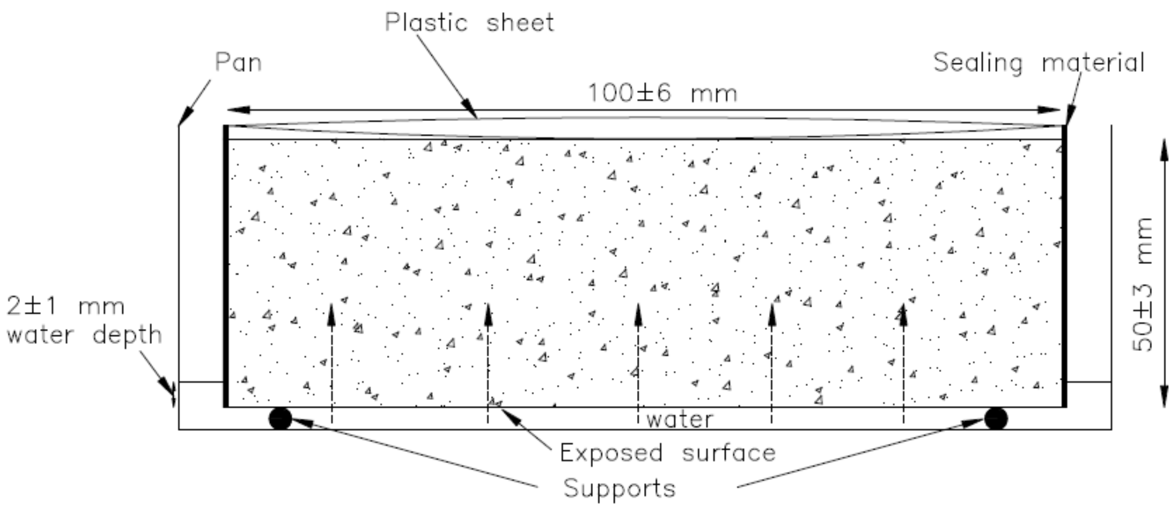

Figure 5.3: Schematic of procedure for ASTM C1585 (ASTM C1585, 2004)

The initial Surface Absorption Test (ISAT) measures the rate of water absorption into the surface layer of the concrete. In this test, a cup with a minimum surface area of 5000 mm^2 is sealed to the concrete surface and filled with water. The rate at which water is absorbed into the concrete under a pressure head of 200 mm and temperature of 20°C is measured by movement along a capillary tube attached to the cup at an interval of 10, 30 and 60 minutes. Schematically this is shown in Figure 5.4.

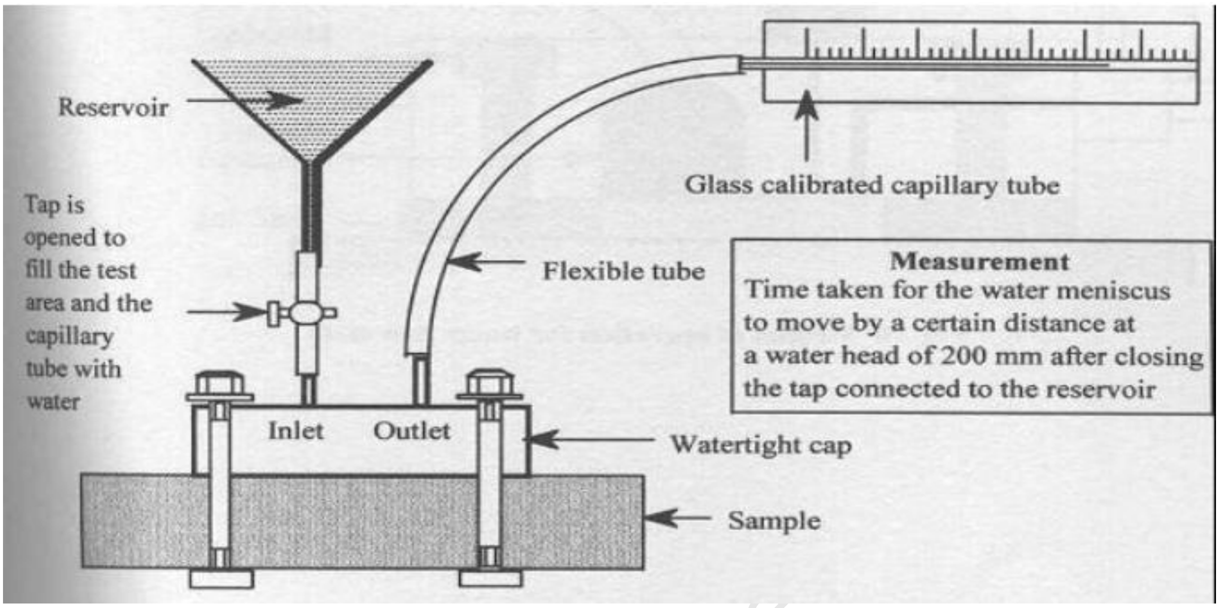

Figure 5.4: Initial Surface Absorption Test (ISAT) test setup (BS 1881: Part 208, 1996)

Though this test is portable and inexpensive, it has been reported to be sensitive to small changes in concrete mix constituents, strength grade and curing methods. The major limitation with this test includes the difficulty of ensuring watertight fixing *in-situ*. Though tests on oven dried samples give reasonably consistent results, it is different for other cases such as for *in-situ* concretes where the results are less reliable (Holmes et al., 2009).

The Australian Standard AS 1012.21 test method has been adapted from ASTM C642 method that measures volume of permeable voids (VPV) as a percentage of the volume of the solid. However, AS 1012.21 measures the apparent volume of permeable voids (AVPV), as a percentage of both solid and voids. The cylinder samples are cured in limewater for 14 days following by air curing for 42 days at 23°C until tested. For testing water absorption, the specimens are oven-dried at 105°C for 24 hours and then immersed in water for at least 48 hours. Test results may be affected by numerous factors including air entrainment, compaction, curing, age of concrete specimen, and exposure conditions such as carbonation. As with ASTM C1585, compliance absorption limits with this test method have not yet been determined (CCAA, 2011).

Although several absorption tests are currently available in the industry, all have been shown to be affected by *in-situ* moisture of concrete when tested (Bickley et al., 2006b). Table 5.3

summarises the test methods for measuring the rate absorption with respect to their strengths and weaknesses.

Table 5.3: Performance test methods for measurement the rate of absorption

Test Method	Duration to complete testing	Comments
ASTM C1585	30-60 days	-Limited evaluation of the quality of curing when either the finished or formed surface is tested -Influenced by SCMs and curing. -Affected by high saturation of the *in-situ* concrete. -Acceptable absorption limits have not yet been determined.
ISAT BS 1881:1996	10 minute	-Portable, inexpensive and suitable for field measurement of in-situ concrete. -Provides reasonably consistency results on oven dried samples, but in other cases results are less reliable. -Affected by high saturation of the in-situ concrete. -localised material differences with the same concrete can cause variation.
AS 1012.21	14-45 days	-High temperature affects both the viscosity and the mobility of the water molecule. -Air entrainment, curing, age and exposure conditions such as carbonation may affect test results

5.2.3 Performance Test Methods for Resistance to Specific Durability Problems

Design standards such as CSA A23.1/23.2 and ACI 318 have successfully implemented tests that evaluates resistance of concrete to specific durability problems i.e. sulfate and freeze/thaw attacks. The limits and recommended acceptance criteria with each test method were discussed in previous chapter i.e. chapter 4, for both design standards. These sections only presents an overview of the selected standardized test methods for evaluating the resistance of concrete exposed to such attack mechanisms.

5.2.3.1 Performance Test Methods for Resistance to Freeze/thaw Attack

Test methods for determining the resistance of concrete to cycles of freeze/thaw are relatively expensive, difficult to employ and consume time to obtain results. In this way, standardized test methods that can evaluate the resistance of concrete to cycles of freeze/thaw are normally employed. This includes ASTM C666 "*Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing"* and ASTM C457 *"Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete"*.

ASTM C666 evaluates the resistance of saturated concrete specimen by exposing it to cycles of freeze/thaw and measure degradation. It may be used to compare various mixes based on control mixes of known performance record in service, hence suitable for pre-qualification.

Though it is widely used with different acceptance criteria, it has been criticized for being overly aggressive (Bickley et al., 2006b).

ASTM C457 is capable of measuring air content of the fresh concrete and spacing factor of the hardened concrete and therefore can be used for both pre-qualification and for quality control. Though it is slow and tedious to perform, it may be used to assess the performance of new or unknown materials or used routinely for testing structures. Table 5.4 provides further information regarding the individual test methods for resistance to freeze/thaw attack.

5.2.3.2 Performance Test Methods for Resistance to Sulfate Attack

Several standardized test methods are available for evaluating the resistance of concrete exposed to sulfate conditions. These include ASTM C1012 *"Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution"*, AS 2350.14 *"Methods of Testing Portland and Blended Cements - Length Change of Portland and Blended Cement Mortars Exposed to a Sulfate solution*" and CSA A3004-C8 *"Test Method for Determination of Sulfate Resistance of Mortar Bars Exposed to Sulphate Solution"*. The theory behind these test methods is similar where combinations of SCMs and Portland or blended cements are evaluated to determine the resistance of concrete to sulfate attack. Generally, a mortar bar is immersed in sulfate solutions and expansion is measured over time and so provides a performance requirement for pre-qualifying SCMs of the concrete for sulfate resistance. Table 5.4 provides further information regarding the individual test methods for resistance to sulfate attack.

Table 5.4: Performance test methods for resistance to Specific Durability Problems

Test Method	Duration to complete testing	Comments
Freeze/thaw resistance		
ASTM C666	4 months	-Can show the individual benefits of air-entrainment and frost resistant aggregates. -Expensive and takes long to complete especially when a new mixture design needs to be qualified. -It has been criticized for being overly aggressive.
ASTM C457	14 days	-Measures air content and spacing factor of the hardened concrete. -May be used to assess the performance of new or unknown materials. -It is slow and tedious to perform, which make it susceptible to human error and sometimes requires judgement of an expert technician.
Sulfate resistance		
ASTM C1012	6 or 18 months	-Evaluates sulphate resistance of concrete mixtures using alternative combinations of SCMs. -Limits with each test method is covered in chapter four.
AS 2350.14	8 months	
CSA A3004-C8	6 or 12 months	

5.3 South African Performance-based Test Methods for Concrete Durability

Three Durability Index (DI) tests, i.e. Chloride Conductivity Index (CCI), Oxygen Permeability Index (OPI) and Water Sorptivity Index (WSI), that characterize the potential durability of concrete based on measuring transport properties of the cover depth, have been developed in South Africa (Alexander, et al., 1999). These tests are simple and practical to perform, and they may be applied either in the laboratory using concrete specimens or cores taken from the as-built structures. These tests are suitable for evaluating materials and mixture proportions for design purposes and for quality control of concrete on site. Detailed descriptions regarding test procedures for each DI test can be obtained from the DI test manual (UCT, 2010). However, the basic principles of each DI test are described in this section including a short brief that illustrates the strengths and weaknesses of each test.

The chloride conductivity test measures the conductive ionic flux through a concrete disc under a 10 V potential difference which is then related to the chloride diffusion properties of the concrete (Streicher and Alexander, 1995). The test consists of a two-cell conduction rig, each cell containing a 5M sodium chloride solution as shown in Figure 5.5. The disc specimen is pre-conditioned by being oven-dried at 50°C followed by 24 hours vacuum saturation in the 5M sodium chloride solution. The chloride conductivity is determined by measuring the instantaneous current flowing through the disc specimen.

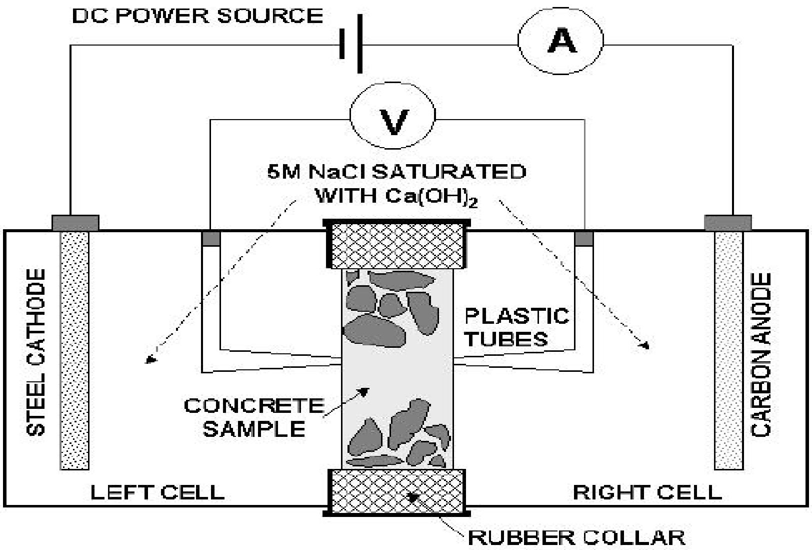

Figure 5.5: Schematic diagram of chloride conductivity Index (CCI) test (Ballim et al., 2009).

Correlations between diffusion coefficients and CCI values at 28 day have shown good results over a broad range of concretes (Alexander and Beushausen, 2009). The chloride conductivity index obtained has also shown a satisfactory performance when used in a chloride prediction model such as the UCT Model for estimating DSL of RC structures. A direct relationship has been shown between chloride index values and the decrease in chloride conductivity with the addition of SCMs. Nevertheless, CCI indicated poor reproducibility based on a round robin test carried out by Stanish et al. (2006). In this regard, more data from various construction sites are required in order to develop correlations between CCI values and the actual performance of the structure.

The Oxygen Permeability Index test consists of measuring the pressure decay of oxygen passed through a 30 mm thick slice of 70 mm diameter core of concrete placed in a falling head permeameter. An illustration of the test apparatus is shown in Figure 5.6. The OPI is the negative logarithm of the D'Arcy coefficient of permeability, where a high index value indicates less permeable concrete. The oxygen permeability test assesses the overall micro and macro-structure of the outer layer of the concrete. Thus the test can be used to assess the effectiveness of curing of concrete and compaction (Alexander et al., 2010).

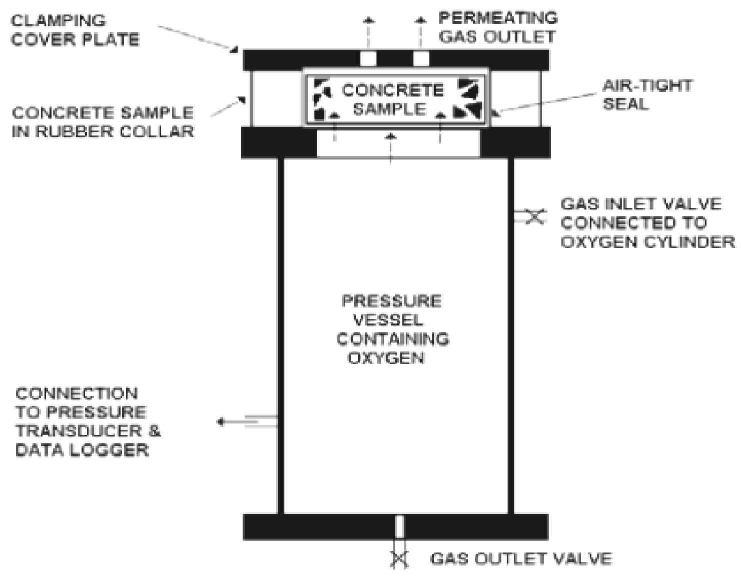

Figure 5.6: Schematic diagram of Oxygen Permeability Index (OPI) test (Ballim et al., 2009)

Although a good correlation between OPI values and carbonation resistance has been developed, further work is still required in order to develop more refined values that relates to the actual performance of the structure (Alexander et al., 2008). Based on work carried out by Stanish et al. (2006), the OPI values were found to be reproducible. In this regard, OPI values may be used as a design parameter to ensure durability of RC structures exposed to carbonating conditions.

The water sorptivity test measures the rate of movement of a water front through the concrete under capillary suction. The specimen is pre-conditioned at 50°C to ensure low moisture contents at the start of the test. The test procedures and the determination of the rate of absorption are similar to ASTM C1585 presented in section 5.2.2.

Similar to OPI, WSI can be used to assess the effectiveness of curing of concrete and compaction. According to work carried out by Stanish et al. (2006), WSI values were found to be reproducible. This test has shown to be sensitive to the outer layer of the concrete and therefore may be used in durability specifications as a site control parameter (Alexander et al., 2008). However, WSI has not been linked to deterioration mechanisms so far and therefore it is not used as a design parameter. Similar to other test methods discussed in section 5.2.2, WSI values has shown to be dependent on concrete specimen age, curing practices and exposure conditions such as carbonation and marine salts. Table 5.5 summarises individual DI tests with respect to their strengths and weaknesses.

Table 5.5: South African performance test methods for Concrete Durability

Test Method	Duration to complete testing	Comments
CCI Durability Index	28-35 days	-Provide input data for chloride prediction models. -Results correlate well with diffusion coefficient based on international test methods. -Sensitive to materials effects and construction practices. -Have poor reproducibility values.
OPI Durability Index	28-35 days	-50 year carbonation depths may be determined by carbonation prediction model. -Detect difference in w/b ratio, binder type and curing condition at higher significant level. -Results correlate well with other test methods such as Cembureau and Torrent Permeability Tester. -Have good reproducibility values.
WSI Durability Index	28-35 days	-Can be used as a quality control test of concrete on site. -Specimen age and construction practices such as curing may affect results.

5.4 Service Life Prediction Models

Much work has been done that deepens the understanding of the basic mechanisms that control the movement of deleterious substances into concrete. In addition to this remarkable work that gives a better chance to improve the current design standards, it has resulted in the development of various service life models (SLMs). These models can be used to estimate DSL of RC structures as a function of materials and environmental conditions.

The models can be categorized as empirical models which may be either deterministic or probabilistic, and physical models which are often deterministic (Oslakovic et al., 2010). Deterministic models often use single input parameters that are justifiable and the output is a single value that can be assessed against the design requirements. On the other hand, probabilistic models require variable inputs as distribution functions and the output is a service life distribution which can be assessed statistically to estimate the service life. There are more complicated models such as the one developed by Gannon and Tikalsky (1999), which combines both deterministic and probabilistic models using Monte Carlo statistical simulation.

Most SLMs currently available such as *Life*-365 developed in USA, AGEDDCA developed by the Concrete Society in UK, UCT Model developed in South Africa, etc, are deterministic models. They account for high levels of chlorides and relative humidity resulting from marine or de-icing salt conditions. These models are based on Fick's 2nd law of diffusion that estimates chloride concentration as a function of time and depth, and time to corrosion initiation (Que, 2007). Assumptions used in each model with respect to input parameters are not necessarily similar. Some models e.g. AGEDDCA and *Life*-365 use default parameters such as exposure condition and temperature, which are linked to a certain geographical location and therefore difficult to adjust. In other models such as Holcim (CIM) the parameters may be customized, but there are several restrictions particularly with the application of different SCMs. Table 5.6 illustrate some of SLMs with respect to their relevance in estimating DSL of RC structures.

5.6: SLMs for estimating DSL of RC structures (Lee and Chisholm, 2005)

Service Life Model	Comments
AGEDDCA (Concrete Society, UK)	-Include a number of refinements to Fick's law prediction models e.g. taking into account additional durability enhancement measures such as surface coatings and corrosion inhibitors -The database for deriving parameters is more extensive and encompasses the broadest rage of cement types -No predefined exposure classes or value limits for surface chloride concentration -Reference chloride coefficient derived from exposure profiles and depend on w/b and % of SCMs
Life – 365 USA	-Simple to use and flexible, with a manual describing both operation and assumptions involved, including the test methods used to derive the input parameters -Use default parameters such as exposure condition and temperature that are linked to certain geographical condition and therefore difficult to adjust
CIM Holcim (New Zealand) Ltd	-Reference diffusion coefficient derived from exposure profiles and linked to concrete design strength -The time-reduction index reflects the percentage of blended cement -It is simple to use and allow user customisation of input parameters -It does not link surface chloride concentration with predefined exposure classes -Restricted applicability i.e. no default values for SCMs e.g. FA

Many proposed deterministic models are very detailed and require considerable data and knowledge to be easily applied. Unfortunately, there is lack of extensive knowledge among engineers with respect to wide variety of material properties and site-specific conditions in order to apply the models properly. Moreover, there are still several challenges with the models themselves. For instance, the models assume parameters such as concrete quality, cover depth and the environmental conditions to be constant that gives a single value of service life (Khatri and Sirivivatnanon, 2004). This is not always the case as these parameters often vary and may cause significant variation in the value of service life.

In reality, concrete structures may be exposed to different environments with multiple transport mechanisms. In this way, chloride ingress may occur by mechanisms other than diffusion as often assumed in the deterministic models (Clifton, 1993). Hydraulic flow of chloride solution due to a gradient of water pressure, capillary suction of chloride-bearing solution in an unsaturated pore system resulting from the surface tension of pore walls, convection of chloride solution due to wick action, etc., may all be present and difficult to accommodate in a single model (Edwardsen, 1995; Akita and Fujiwara 1995; Buenfeld et al., 1995). In this situation, these models do not comprehensively characterise the real transport processes and therefore do not actually represent the structure in the actual environmental conditions.

The models reflect only a statistical average of specified performance requirements for a particular concrete in a certain exposure condition (Lee and Chisholm, 2005). Therefore there is always a constraint due to environmental-specific factors that may give different result when the models are applied in different environments.

The probabilistic models assumed that if input parameters are properly selected, reasonable prediction results may be obtained (Tang, 2013). However, this might be a difficult task since the input parameters depends on the test methods which often have different values, even within similar test method. In this way, the results obtained from probabilistic models often underestimate the actual chloride ingress.

Most SLMs recently developed cover penetrability of deleterious substances for un-cracked concrete only. Nevertheless, cracks frequently occur in concrete structures as a result of load-induced stresses. It is therefore inappropriate to consider only un-cracked concrete for modelling concrete for penetrability. In this context, further research is still required to establish the influence of various cracks and the penetrability in modelling the service life of concrete structures.

Currently, SLMs only simplify the reality and therefore do not estimate exactly DSL of RC structures. The input parameters to SLMs involve several assumptions and often incorporate safety factors to reflect simplifications. More field data from practical experiences and periodic monitoring of structures in actual environment are required in order to provide realistic quantification of essential input parameters so as to validate their application. A comprehensive SLM that addresses from first principles the various transport mechanisms to which a concrete is exposed may overcome the current constraints. Generally, SLMs may provide useful information regarding the early onset corrosion that allows one to properly schedule the required maintenance.

5.5 Concluding Remarks

The concrete pore structure is known to be of significant importance with respect to durability. Since durability of concrete depends on its penetrability, measurement of relevant transport properties such as permeability, diffusion or absorption is imperative. This can be achieved by means of various PTMs currently available in the industry that may characterize the potential ingress of deleterious substances into concrete.

Several PTMs commonly used in specifications have shown some limitations. Some tests have poor precision, poor reproducibility or require a long time to complete, while others do not relate directly to the field performance. It is therefore essential to overcome these limitations by understanding the factors causing them prior to apply them in the specifications. A thorough knowledge regarding the principle operation of individual test method cannot be overstated when need to apply them in the specifications, either independently or in combination.

Ideally, PTMs intend to provide durability indicators that would be used as inputs into SLMs as an integrated approach towards durability design and specifications termed 'performance-based specifications'. However, several PTMs currently used in the specifications have been questioned for their validity to provide such indicators based on accelerated test data, particularly because of the inherent variability. ASTM C1202, despite strong criticisms. It has been used widely for specification, quality control and acceptance purposes. Though it shows a fair correlation between concrete resistivity and concrete permeability, equal Coulombs values obtained do not always describes similar resistance to chloride penetrability.

Though there is no universal test method and therefore different situations may require different test methods, South African DI tests have shown positive competence over other PTMs reviewed. These tests are suitable for evaluating materials and mixture proportions for design and quality control. The integrated approach that link DI values and SLMs have shown satisfactory result that makes DI tests suitable under performance-based specifications.

Although numerous SLMs exist, some are easy to use but with limited application, while others require more input data and therefore greater user expertise. Moreover, estimation of service life for RC structures is influenced by several unpredictable factors including environmental actions that often vary. Therefore relying on SLMs that consider constant values for concrete quality, cover depth and the environmental conditions may not always represent the actual service life. Comprehensive SLMs that address from first principles the various transport mechanisms to which a concrete is exposed are necessary. However, this depends on the availability of data from periodic monitoring of structures in actual environmental conditions, which is the focus of current research worldwide.

Generally, performance-based specifications that integrate durability specifications (*i.e.* durability indicators from relevant test methods) and durability design through SLMs to estimate DSL of RC structures is the way forward. Challenges that currently exist with respect to adequate PTMs and SLMs are being addressed through research. It is expected in near future that the performance-based concept will be a powerful tool for solving many durability problems current exist in the construction industry.

CHAPTER 6: CASE STUDIES

6.1 Introduction

The previous chapters described approaches for durability specifications in the design standards and in project specifications. It has been shown that prescriptive approach is commonly used in most design standards while performance approach has been used in few design standards and in project specifications. This chapter therefore presents some of the case studies where performance specifications have been applied, both hypothetical and practical. It also describes PTMs used in practical cases with respect to their relevance in achieving the required performance. The hypothetical case study is taken from USA, where a comparison was made between mixtures developed using current prescriptive requirements in ACI 318, and mixtures developed to satisfy performance criteria. The practical case studies are taken from projects conducted in South Africa, and from Abu Dhabi project in the Arabian Gulf.

Other projects including bridges, viaducts and marine structure developed in Europe particularly in UK, Portugal, and France that demonstrate performance applications are included. Though sufficient information with respect to detailed performance specifications was unavailable, specified durability parameters and tests performed to assess the limits from project to project are presented.

6.2 Hypothetical Case Study: HPC Bridge Deck Construction - Specifications

Obla and Lobo (2006) prepared concrete mixtures according to prescriptive requirements and compare to mixtures developed to satisfy the intended performance for HPC bridge deck construction. A control mixture for HPC bridge deck construction was designed according to the prescriptive specifications developed by one of the major state Departments of Transportation (DOT) in USA. These specifications include a 28 day compressive strength, maximum w/cm, minimum total cementitious content, specified dosages for FA and SF, slump, air content, and maximum chloride permeability value in accordance with ASTM C1202. In contrast, three performance mixtures were developed to target slump values of 102 mm to 152 mm, air content of 4% to 8%, compressive strength of 27.6 MPa, and chloride permeability of 1500 Coulombs after 45 days of moist curing. For the performance mixtures, no restrictions on material compositions and proportions were given but rather the mixtures were evaluated based on achievement of a maximum chloride permeability value.

6.2.1 HPC Bridge Deck Construction: Mix design, Testing and Results

Table 6.1 presents the four mixtures, where BR-1 is the control prescriptive mixture while BR-2 to BR-4 was mixtures developed to satisfy performance criteria. Lower total cementitious content was used for the performance mixtures when compared to the control mixture. Similar w/cm was used for mixtures BR-1 to BR-3 while a lower value was used for mixture BR-4. Different SCMs contents were used for each mixture as indicated in the table with additional replacement of Utra-Fine FA for mixture BR-4. The mixtures were designed to a target w/cm where high range water reducing (HRWR) admixture was used to adjust the desired slump.

Fresh concrete mixtures were tested for slump, air content and temperature according to relevant ASTM Standards. Hardened concrete properties were tested using various durability tests including ASTM C1556, ASTM C1585, ASTM C1202, etc. However, only ASTM C1202 is used to evaluate concrete performance in this section.

Table 6.1: Details for mixture proportions and testing for HPC (Obla and Lobo, 2006)

Experimental parameters	**Control mixture**	**Variable mixture proportions**		
Mixture	**BR-1**	**BR-2**	**BR-3**	**BR-4**
1: Calculated mixture proportions				
Cement (OPC), kg/m^3	330	249	182	244
Fly Ash (class F), kg/m^3	63	88	0	86
Silica Fume, kg/m^3	30	14	0	0
Slag, kg/m^3	0	0	182	0
Utra-Fine Fly Ash, kg/m^3	0	0	0	20
Total cementitious content, kg/m^3	**423**	**351**	**364**	**350**
Coarse aggregate (No. 67), kg/m^3	1079	1123	1177	1115
Fine aggregate, kg/m^3	672	701	734	696
Water content, l/m^3	165	137	142	125
w/cm ratio	**0.39**	**0.39**	**0.39**	**0.36**
AEA, %	0.02	0.03	0.02	0.02
Type A WR, %	0.2	0.2	0.2	0.2
Type F HRWR, %	0.7	0.5	1.0	0.6
2: Tests on fresh concrete properties				
Slump, mm	102	127	127	146
Air content, %	4.6	7.2	4.7	7.6
Temperature at delivery , °C	38.4	38.4	36.1	38.4
3: Tests on hardened concrete properties				
Compressive strength at 28 day, MPa	51.6	46.9	61.9	49.5
Chloride permeability (ASTM C1202), Coulombs				
45 days	**1563**	**1257**	**1126**	**1244**
110 days	541	434	541	479
180 days	327	275	375	242

6.2.2 HPC Bridge Deck Construction: Discussion and Conclusion

Although the mixtures were not linked to any exposure condition, they use lower than the specified w/cm presented in ACI 318 for any category of exposure. The four mixtures were found to have slump values that varied between 102 mm and 146 mm, which are within the specified values. In terms of air content, values varied between 4.6% and 7.6%, which are within specified values and comply also with the minimum requirements in ACI 318. For concrete temperature, values between 36.1°C and 38.4°C were obtained, which are slightly higher than the maximum temperature of 35°C required by ACI 318 at time of delivery. Nevertheless, ACI 318 generalized temperature at delivery for all concretes and there is no distinction between HPC and normal concrete temperature.

In the case of hardened concrete properties, the four mixtures significantly exceeded the specified compressive strength of 27.6 MPa. In terms of durability requirements, specified chloride permeability was successfully achieved by all the performance mixtures except for the prescriptive mixture BR-1, which had a slightly higher value of 1563 Coulombs. It should be noted that this is still a low value according to ASTM C1202 (refer to chapter 5, Table 5.1). Very low values between 242 and 375 Coulombs were obtained for all mixtures after 180 days of moist curing, which means relatively low permeable concrete. Presumably, moderate or high Coulombs values would be expected especially for mixture BR-1 if tested at 28 days because of applications of some SCMs in particular FA, which required considerable time to develop sufficient properties.

In broad comparison of material cost, it was observed that performance mixtures had a lower materials cost, between 15% and 23%, when compared to the prescriptive mixture. Importantly, it has been observed that the performance mixtures provided a freedom of material optimization where concrete producers could develop concrete meeting the intended performance. Moreover, performance mixtures had shown similar or better results in terms of chloride permeability when compared to the prescriptive mixture. This validates applicability of ASTM C1202 since lower Coulombs values indicated better resistance to chloride ingress.

6.3 Practical Case Studies

Two case studies are considered in this section in order to demonstrate practical application of performance specifications for concrete durability. One of the practical case studies is the application of DI tests towards upgrading the national road network in South Africa that include production of precast RC barriers. The precast barrier elements are to be used temporarily to divide individual lanes during construction, or to be used as permanent highway barriers between opposite lanes. The production of precast concrete elements were conducted in Cape Town and were expected to be situated in an inland area, characterized by South African carbonation conditions. Durability requirements i.e. OPI values for the precast elements were related to SLM for carbonation-induced corrosion to estimate DSL of precast concrete elements.

Abu Dhabi project in the Arabian Gulf is another practical case study, where a RCP test in accordance with ASTM C1202 was used as part of durability specifications to determine its appropriateness. The Gulf region is considered the most corrosive place in the world (Haque et al., 2007) and so corrosion of the reinforcing steel is regarded as the main durability element controlling the service life of RC structures. Since authorities in the region requires extended service life of about 75 to 100 years for key infrastructure, with minimum maintenance and life cycle cost, the project considered two SLMs i.e. UCT Model developed in South Africa (Mackechnie, 2001) and Life 365 developed in USA (Thomas and Bentz, 2000) for estimation of DSL.

6.3.1 Application of DI tests: Project Specifications for DI Parameters

Carbonic environments that requires OPI values specifications in South Africa include XC3 (*i.e.* Moderate humidity, 60-80%) and XC4 (*i.e.* Cyclic wet and dry) as adopted in EN 206-1 (Alexander et al., 2008). Common OPI values for South African conditions ranges from 8.5 to 10.5, where a higher value indicates a concrete of potentially higher quality. This project specified a minimum compressive strength of 35 MPa and a cover depth of 40 mm to satisfy SANRAL requirements presented in Table A.3 in the Appendix.

6.3.1.1 Application of DI tests: Mix design and materials parameters

Trial mixes were performed at the University of Cape Town to obtain an optimum balance between mix proportions and durability requirements. Cementitious binder content of 425 kg/m^3, 380 kg/m^3 and 340 kg/m^3 consists of 65% OPC and 35% Corex Slag were tested to identify minimum binder content that meet durability requirements. Two sets of trial mixes were made, one containing enough superplasticiser targeting a slump of 75 mm at a constant water content of 180 *l*/m^3, and other trial mixes containing maximum amount of superplasticiser as specified by the supplier. Mix design properties and OPI test results obtained after 28 days are presented in Table 6.2 and Table 6.3 respectively. Note that the mix proportions in Table 6.3 are equivalent to those in Table 6.2, except for water content, superplasticiser content, w/b ratio, and slump.

Table 6.2: Mix design series A: constant water content of 180 l/m^3
(Beushausen and Alexander, 2006b)

Mix	**A1**	**A2**	**A2 (Control)**	**A3**
OPC 42.5 CEM I, kg/m^3	275	247	247	221
Corex Slag, kg/m^3	150	133	133	119
Binder content, kg/m^3	**425**	**380**	**380**	**340**
Sandstone, 19 mm, kg/m^3	1150	1150	0	1150
Greywacke, 19 mm, kg/m^3	-	-	1050	-
Dunesand, kg/m^3	439	464	464	489
Crusherdust, kg/m^3	188	199	199	209
Water , kg/m^3	180	180	180	180
w/b ratio	**0.42**	**0.47**	**0.47**	**0.53**
Superplastiziser, m*l*/m^3	1333	-	-	-
Slump, mm	80	80	105	100
OPI (log scale)	**10.2**	**10.0**	**10.1**	**10.0**

Table 6.3: Mix design series B: Maximum use of superplasticiser
(Beushausen and Alexander, 2006b)

Mix	**B1**	**B2**	**B2 (Control)**	**B3**
Water , kg/m^3	177	173	173	153
Superplastiziser, m*l*/m^3	2125	1900	1900	1700
w/b ratio	**0.42**	**0.46**	**0.46**	**0.45**
Slump, mm	65	70	120	90
OPI (log scale)	**10.4**	**10.3**	**10.3**	**10.3**

6.3.1.2 Application of DI tests: Discussions and Conclusions

As shown in Table 6.2, all the mixes had relatively high OPI values ranging from 10.0 to 10.4, and were well above the required specification of 9.7. Though all mixes showed good results, the precast manufacture decided to use a binder content of 425 kg/m^3, water content of 175 *l*/m^3, and sufficient superplasticiser content for early strength requirements that would facilitate demoulding.

Data from *in-situ* quality control tests undertaken to ensure consistency quality indicated that the precast barrier elements met design specifications, with exceptional of element 309 as indicated in Figure 6.1. The elements which are close to the specified threshold of 9.7 indicated that the concrete was not ‘over designed’, while element 309 may require remedial measures or penalty when further tests from the same batch proved so.

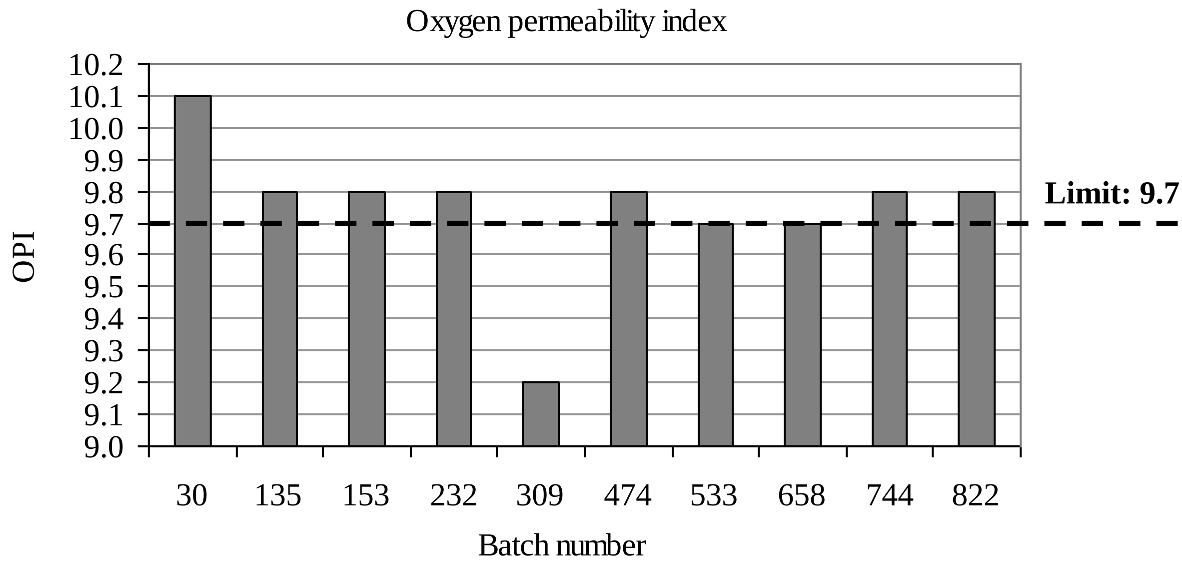

Figure 6.1: Quality Control: OPI test results from core samples removed from precast elements (Beushausen and Alexander, 2006b)

Generally, application of DI tests has resulted in the manufacture of precast elements through improved production procedures in terms of compaction and curing. DI tests increased awareness on durability requirements that helps to improve the overall quality of concrete production. It may be concluded that DI tests helps to assess the influence of various mix characteristics on durability properties in the design phase, and serves as a basis for ensuring consistency quality control during the construction phase. Though DI tests may be used effectively to ensure quality control of *in-situ* structures at this stage, more refined data that would be linked to SLMs are still required to validate the approach.

6.3.2 Practical Case Study: Abu Dhabi Project Description

The project consisted of precast elements, placed side-by-side to form a 19 km long perimeter sea wall on the coastline that is characterized as GM (*i.e.* Gulf Marine) exposure according to Table A.4 in the Appendix. In terms of macroclimate, the Gulf region is classified as a hot-humid zone as shown in Figure A1 in the Appendix. The project specifications required 100 years design life with 25 years period before first maintenance.

The precast elements were monitored by strength tests and various durability tests including RCPT in accordance with ASTM C1202, water absorption in accordance with BS 1881: Part 124 (1988), water permeability test in accordance with DIN 1048: Part 5 (1991), and Initial

Surface Absorption Test (ISAT) in accordance with BS 1881: Part 208 (1996). This section only concentrates on RCPT as a quality control test for resistance to chloride ingress.

6.3.2.1 Abu Dhabi Project: Specifications, Mix Design, Results and Conclusion

Durability of precast elements was addressed at the design stage by selection of appropriate mix design and specifications as shown in Table 6.4. According to Haque et al. (2006), cementitious binder content raging from 300 to 400 kg/m^3 and chloride content about 0.2% may suffice durability requirements in the Gulf region. However, cementitious binder of 410 kg/m^3 was selected and w/cm of 0.35 was used for the trial mix.

Table 6.5 indicates the results obtained from the trial mix. It can be shown that the trial mix comply with ASTM C1202 specifications (refer chapter 5, Table 5.1), where low and lower values were obtained. In this context, the trial mix was then approved and used as benchmark for developing acceptance limits for chloride permeability for both cubes and cores taken *in-situ*. Both material potential and as-built margins that accounts for coefficient of variation were established according to Table 6.6 in order to develop appropriate acceptance limits.

Table 6.4: The trial mixture for the pre-cast L-shaped sea walls (Heath and Alexander, 2012)

Mix design for 1 m^3 of Ready Mixed Concrete				
Materials	**Type**	**Source**	**Materials, %**	**Weight, kg/m^3**
Cementitious	Portland Cement	RAK Cement Factory	70	287
	Fly Ash	TRACE Ex. RAK Cement Factory	25	102.5
	Silica Fume	Elkem Ex. RAK Cement Factory	5	20.5
		Sub-Total	**100**	**410**
w/cm ratio				**0.35**
Aggregates	20 mm Crushed	Stevin Rock – RAK Limestone	33	619.7
	10 mm Crushed	Stevin Rock – RAK Limestone	22	412.4
	5 mm Crushed Normal	Stevin Rock – RAK Limestone	32	600.8
	Dune Sand Natural-Red	Al Ain UAE (Limestone & Quartz)	13	243.3
		Sub-Total	**100**	**1876.2**
Water content	Drinking water	Abu Dhabi Distribution Company	-	151.7
Admixture	SP 2000 Superplasticiser	Fosroc	-	10.6
		Total		**2448.5**

Table 6.5: Concrete trial mixture tests results (Heath and Alexander, 2012)

Parameter measured	Test method	Results (Average)
1: Tests on fresh concrete properties		
Slump, mm	BS 1881: Part 102: 1983	210
Acid-soluble chloride, %	BS 1881: Part 124: 1988	0.01[1)]
Delivery temperature, °C	BS 5328: Part 4: 1990	29.1[2)]
2: Tests on hardened concrete properties		
Compressive strength, MPa	BS 1881: Part 116: 1983	3 day → 39.0 7 day → 48.8 14 day → 58.2 28 day→ 65.1
RCPT, Coulombs	ASTM C1202: 2007	14 day → 1809.4[3)] 28 day → 872.6[3)]

Notes:

1. [1)] Average of 2 cubes; [2)] Average of 8 samples; [3)] Average of 6 cubes.

Table 6.6: Acceptance criteria for concrete quality (Heath and Alexander, 2012)

Parameter measured	Acceptance criteria
Compressive strength	<5% of test results < the characteristic strength (45MPa)
RCPT (28 day cores)	<10 % of test results > the characteristic value 2000 Coulombs
RCPT (28 day cubes)	<10 % of test results > the characteristic value 1020 Coulombs

During project execution, RCP values were found to have large variability, e.g. high as 8000 Coulombs on *in-situ* cores and as high as 5000 Coulombs on cubes, values that are higher than ASTM C1202 specifications. However, the variability was reduced to acceptable levels as indicated in Figure 6.2, after all the affected parties became familiar with test procedures.

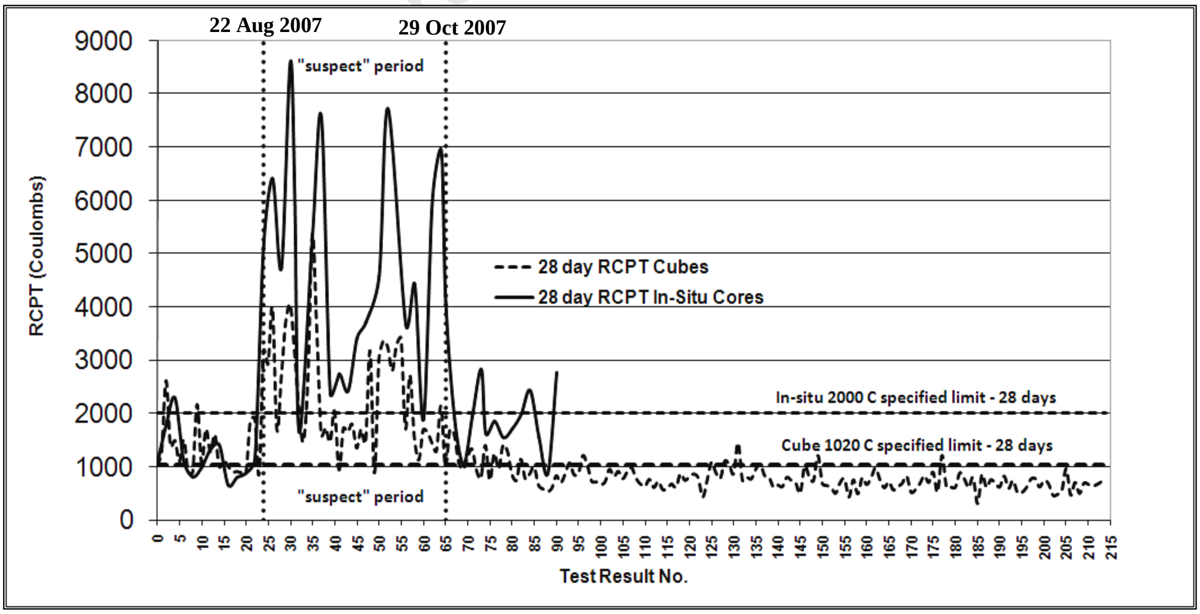

Figure 6.2: RCPT results for Abu Dhabi marina concretes (Heath and Alexander, 2012)

In terms of service life design, UCT Mode was adopted where chloride surface concentration (C_s), diffusion coefficient (D_c), and time to onset corrosion at various cover depths was determined. Taking into account the temperature and salinity conditions of Gulf region, four scenarios for surface concentration and diffusion coefficient were developed as shown below:

- Scenario 1 (C_s = 6% D_c = 1.1x10^{-9} cm^2/s); Scenario 2 (C_s = 6% D_c = 2.2x10^{-9} cm^2/s)
- Scenario 3(C_s = 10% D_c = 1.1x10-9 cm^2/s); Scenario 4 (C_s = 10% D_c = 2.2x10^{-9} cm^2/s)

The scenarios were used to determine the service life at various cover depths of concrete for the walls based on a chloride conductivity of 0.42 mS/cm and a chloride threshold of 0.40% by mass of binder. Figure 6.3 represents the results obtained. For the worst condition i.e. scenario 4, results indicated 100 years of service life free from corrosion at a cover depth of 75 mm, while with similar cover depth at a favourable condition i.e. scenario 1, more than 225 years service life was predicted. The results shown are only the indicators of service life as they were measured within a short period of time. In this way, it was suggested to develop more results under the same elements for a period of at least one year so as to obtain at least realistic estimate of service life.

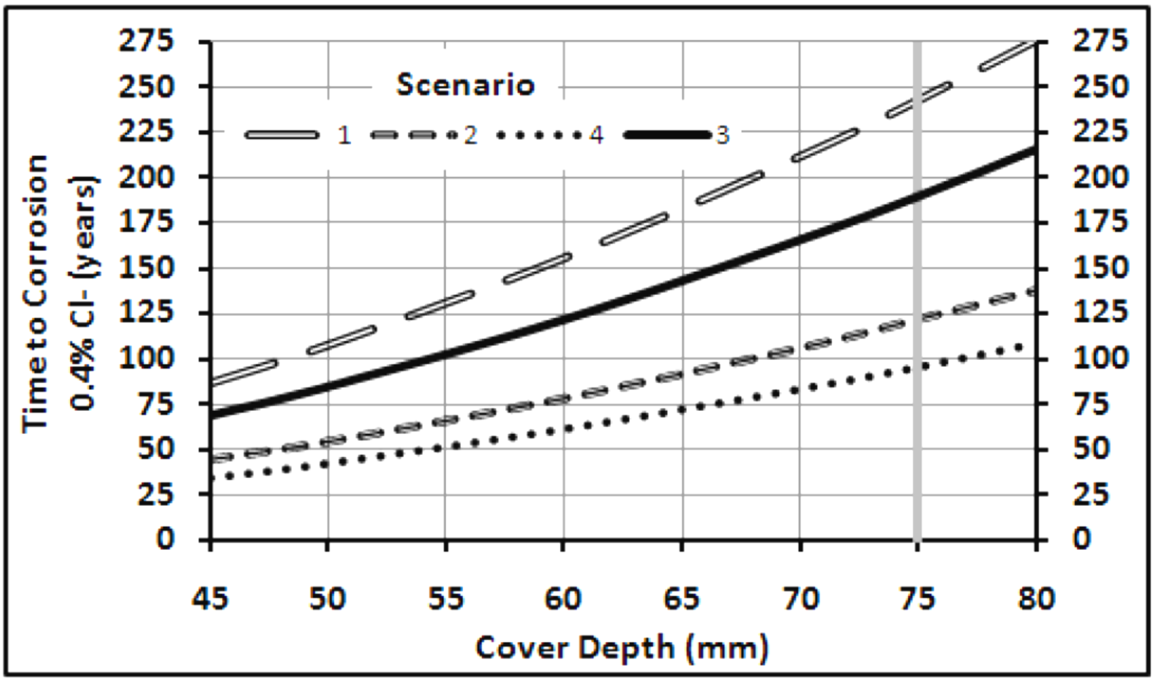

Figure 6.3: Initial Service Life prediction (Heath and Alexander, 2012)

In conclusion, chloride permeability results in accordance with ASTM C1202 have shown positive results in determining chloride ingress. Moreover, the study has shown that the chloride values may be used as an alert to the contractor regarding any concrete mix within 28 day related to durability issues. Based on the Abu Dhabi project, it has been confirmed that RCP test in accordance with ASTM C1202 is appropriate for concrete durability specifications.

6.4 Other Examples of Projects with Performance Specifications for Durability

This section presents some of practical applications of performance specifications for various projects in different countries. In Table 6.7, prescriptive requirements relevant for specific design standards and performance parameters are both presented. As shown in the table, positive results with respect to specified durability parameters were obtained, particularly for chloride diffusion coefficient and chloride permeability. For instance, chloride diffusion coefficient less than 2.5 x 10^{-12} m^2/s when tested at 28 days indicates extremely high chloride penetrability resistance as described by Nilsson et al. (1998). In terms of chloride permeability, values between 100 and 1000 Coulombs are considered very low values according to ASTM C1202 and hence more resistant to chloride ingress. Generally, several specified durability parameters had been achieved with a proper margin.

Table 6.7: Examples of projects with performance specifications for concrete durability (Holmes et al., 2009)

Projects	Specified parameters				Specified durability parameters				Durability parameters results		
	Binder type	Min. cement kg/m^3	Max. w/cm ratio	Min. strength (MPa)	Gas permeability (m^2)	Chloride diffusion coefficient (m^2/sec)	Chloride permeability (coulombs)	Specified service life (years)	Gas permeability (m^2)	Chloride diffusion coefficient (m^2/sec)	Chloride permeability (coulombs)
Vasco da Gama Bridge Portugal 1995-1998	PM (seawater) cement containing FA	400	0.33 to 0.42	B 40 B 45 B 50	< 1 x 10^{-17} (28 days)	< 1x 10^{-12} (28days)	< 1500 (at 28days) < 1000 (at 90days)	120	0.7-0.3x10^{-17} (28-90days) $\leq$ 0.01x10^{-17} (at 18months)	1-4x10^{-12} (28-90 days) 0.2-0.8x10^{-12} (at 18 months)	-
Extension of Condamine Port floating dyke France 1999-2002	CEM I PM (seawater) + FA + SF	425	0.35	B 54 B 65	< 1 x 10^{-16} to 1 x 10^{-17} (28 days)	< 5x10^{-12} (B54) < 1x10^{-12} (B65)	1000-2000 (B54) 100-1000 (B65)	100	5.54x10^{-19} to 1.25x10^{-18}	-	377 - 401
Medway Viaduct (UK) 1998-2001	CEM I + slag or CEM I + FA	325 to 350	0.45 to 0.50	C 40 to C 60	-	< 1 x10^{-12}	-	100	-	-	-

CHAPTER 7: THE WAY FORWARD FOR SOUTH AFRICAN DURABILITY SPECIFICATIONS

7.1 Introduction

A number of publications that address durability specifications in the South African context have appeared in recent years (Alexander et al., 2008; Alexander and Beushausen, 2009; Alexander et al., 2010). However, no published work has been done to incorporate such specifications in the South African Standards, in particular SANS 10100-2 (2009), which provides the basic principles for concrete production. This portion of the study therefore paves the way towards durability specifications implementation in SANS 10100-2.

SANS 10100-2 lags behind in many respects related to durability compared to standards such as CSA A23.1/23.2 (2009) and AS 1379 (2007), which give options in using either prescriptive or performance specifications. When considering alternative approaches to specifying concrete in SANS 10100-2, in particular on durability, it is essential to examine the current state of practice of the standard. Though SANS 10100-2 has been revised previously (*see* chapter 3) in general terms in comparisons to other standards, it is imperative to examine in detail specific clauses that relate to durability requirements. In the following section, specific clauses that address durability requirements in the standard are examined.

7.2 A critique on the Durability Specifications in SANS 10100-2 (2009)

This section highlights specific areas that relate to durability requirements and outlines the necessary durability specifications required to bring the standard up-to-date following international trends. Moreover, tables that summarise specific details in each clause are presented with respect to their shortcomings and recommended measures.

(i) *Design Service Life (DSL):*

The requirements presented in SANS 10100-2 are not linked to any DSL when compared to other design standards such as EN 206-1 where the requirements are based on the assumed design life of 50 years. It is reasonable to assume that the requirements are based on experience or laboratory data and therefore it is not possible to predict the service life when complying with the requirements. It is therefore essential to include a requirement for minimum service life period and link this to the durability requirements. Moreover, some flexibility in terms of material compositions and proportions should be allowed when more than the specified DSL is required.

(ii) *Environmental Exposure Conditions:*

Exposure conditions in SANS 10100-2 are given in qualitative terms such as ‘moderate’, ‘severe’, etc., which are very subjective. There are no clear parameters that guide the specifier on how to choose appropriate exposure class. Since strength and durability requirements follow from exposure classifications, the exposure conditions in SANS 10100-2 need to be expanded into classes and linked to the form of deterioration. Developing a detailed exposure classification in SANS 10100-2 is an important step towards performance specifications implementation in the standard.

(iii) *Material Constituents and Proportions:*

Material-related durability requirements including aggregates, water, admixtures, and cementitious binder are specific durability aspects. They may be dealt with effectively using South African specifications relevant for specific materials as presented in SANS 10100-2. Though chloride and sulfate content limits for fresh concrete mixture are provided, they are never constant and vary significantly depending on the concrete penetrability. In this way, measurements with respect to penetrability are imperative in order to ensure resistance to chloride and sulfate attacks.

Despite the importance of w/b with respect to permeability, it is not a critical requirement in SANS 10100-2 for many exposure conditions, except for freeze/thaw attack and for concrete requiring low permeability. This requirement should be expanded into more exposure conditions in order to provide physical resistance for various durability problems such as sulfate attack, chloride attack, chemical attack, etc.

SANS 10100-2 includes an upper limit binder content of about 550 kg/m^3, which seems to be a very high limit. Though this may help to prevent issues like thermal cracking and additional costs that may result from using higher values than specified, minimum values linked to different exposure classes should also be included. Inclusion of minimum values of binder content in SANS 10100-2 may prevent the risk of using lower than recommended values.

Specific clauses for material-related durability requirements and some recommended measures required to bring SANS 10100-2 up-to-date following international trends are summarised in Table 7.1.

Table 7.1: Clauses with material-related durability requirements in SANS 10100-2

Clause	Measured parameter	Details	Remarks
3.3	Conditions of exposure	Moderate, severe, very severe, and extreme conditions are identified	**Need to be revised following international trends*
Material constituents - Specific durability requirements			
4.1	Cementitious binder	No restrictions given to cements or SCMs when so required for specific project	Specific durability aspects are dealt with effectively using South African specifications relevant for specific material as given in the standard
4.2	Water	Water shall be free from acids, chlorides, alkalis, and organic matter	
4.3	Aggregates	Aggregate should be clearly specified in terms of their types and sources based on the results of previous use	
4.4	Admixtures	Chloride content of admixture should be < 2% of admixture or < 0.03% of cementitious binder expressed as (m/m) of chloride ions	
Material Proportions – Pre-qualification			
6.1.7	Chloride content	Chloride content expressed by mass of cementitious material should be < 0.2% when additional chloride ingress is expected, or < 0.4% for no additional contamination	**The limits are very high. Should be < 0.1% for RC and < 0.4% for mass concrete*
6.1.8	Sulfate content	Total acid-soluble sulfate content, expressed as $S0_3$, should be < 4% (m/m) of the cementitious binder in the mix	**Standardized test methods are required to determine maximum allowable expansion for sulfate resistance.*
6.1.9	Alkali-silica reaction	Total alkali-content of the concrete should be limited to 0.6% of $Na_{2}0$-equivalent	Limit complies with other standards *e.g.* IS 456
6.2.1.2	Permeability resistance	w/b limited to 0.50 for general concrete elements requiring low permeability while 0.53 is given for exposure to freeze/thaw attack	**Should be modified based on improved exposure classes developed in the international standards*
6.3	Cemetitious binder content	Limited to 550 kg/m^3	**Lower limits should also be included following modified exposure classes*

Notes:

1. * Should be amended to include performance-based alternatives.

(iv) ***Resistance to Penetrability:***

Generally, concrete penetrability requirements in SANS 10100-2 are typically prescriptive. For instance, the standard requires concretes exposed to freeze/thaw attack to have entrained air and conform to air content limits provided. Nevertheless, air content and air-void spacing factor of hardened concrete that characterize concrete for durability are not measured. In this regard, standardized test methods should be employed either for pre-qualifying or for quality control in order to characterize concrete for durability.

SANS 10100-2 recommends EN 206-1 or BS 8500 to provide resistance against chloride and carbonation ingress. For other aggressive agents such as abrasion, chemical, and acid attack, the standard recommends local South African specifications. In terms of sulfates which may occur naturally in some soils and ground water, an improvement should be made by classifying the attack mechanisms into classes with appropriate limits depending upon the degree of severity.

(v) *Quality Control:*

SANS 10100-2 evaluates the performance of concrete based on pre-qualification tests through measuring parameters such as slump, temperature and air content. Though these parameters play significant roles, many of them are difficult or impossible to verify in practice and therefore there is no assurance that the pre-qualified mixture will be similar to the as-built structure. For instance, the standard allows water to be added when slump is below specified value provided that w/b is not exceeded. Nevertheless, added water is not always inspected or measured, while w/b which is stated as a control measure for penetrability cannot be easily verified in the field. In this context, test methods which have a direct relationship with in-place performance of concrete are imperative in order to ensure adequate durability.

Minimum specified compressive strength is used to characterize concrete for durability in SANS 10100-2. However, compressive strength is controlled by w/b, which has a poor relationship over a broad range of binders that accounts for penetrability. Moreover, a single parameter is insufficient to characterize the behaviour of concrete which is influenced by complex physical and chemical mechanisms. Thus measurements beyond compressive strength are imperative so as to adequately characterize concrete for penetrability with respect to exposure conditions and therefore ensure durability.

SANS 10100-2 also specifies cover depth requirements for different conditions of exposure like any other design standard, though in different way. Cover depth for some exposure conditions decreases with increase in compressive strength. This conflicts with durability requirements since compressive strength and penetrability do not directly correlate. Experience demonstrates that the greater the cover, the longer the distance for the deleterious substances to travel before they can depassivate the reinforcing steel. In this context, change is required to balance between higher compressive strength (which requires higher material quality) and cover depth so as to obtain a practical solution which is economical.

Although SANS 10100-2 emphasises proper construction practices with regard to concrete production, workmanship and curing methods, the effectiveness of many of these practices are not evaluated in real practice. For instance, though cover depth is normally specified and checked, there are no clear guidelines that measure or verify cover depth against various transport mechanisms. For this reason, in-place concrete always shows higher scatter and variability, which interferes with the achievement of durability requirements.

Specific clauses that relate to resistance of hardened concrete to penetrability are summarised in Table 7.2 with respect to the recommended measures that may bring the standard up-to-date following international trends.

Table 7.2: Clauses relate to concrete penetrability resistance in SANS 10100-2

Clause	Measured parameter	Details	Remarks
Fluid penetrability durability requirements			
6.2.3	Freezing and thawing	Total air-content limits for various size of aggregates are given in Table 2 for normal-density concrete while section 6.2.3.2 gives limits for low-density concrete	**Should be modified based on improved exposure classes developed in the international standards* **Suitable performance test methods should be undertaken to measure relevant transport properties*
6.2.5	Exposure to salt-laden air	Detailed guidance to deal with the exposure condition is obtained in the EN 206-1 or BS 8500	
Fluid penetrability related to specific durability requirements			
6.2.4	Aggressive chemicals	Common cement concrete with pH ≤ 5.5 is not recommended. Suitable coatings can be used in extreme conditions.	Effectively dealt with local specifications or specialist literature on subject matter **Performance test methods e.g. permeability tests may be applied*
6.2.6	Exposure to corrosive fumes		
Quality control			
6.2.1.4	Detailing	Shape and design of exposed structural members should promote good drainage and prevent standing pools	Emphasises on proper construction methods
7.3.3	Tempering and control of mixing water	Water may be added to attain specified slump, provided that w/cm is not exceeded to affect strength and durability	**Performance tests e.g. permeability test may be used as a control measure*
8.2	Cover to the reinforcing steel	Minimum cover to the reinforcement for different conditions of exposure is given in Table 3	**Should be revised based on modified exposure classes*
10.8	Protection and curing of concrete	Recommends minimum curing period in Table 4. Minimum curing period should be established on basis of tests for critical durability performance.	Curing method presented are sufficient to ensure durable concrete

Notes:

1. * Should be amended to include performance-based alternatives.

7.3 Actions Plan for Alternative Specifications in Revised SANS 10100-2

Requirements for concrete to withstand environmental actions either in prescriptive or on performance basis depends on DSL. Though DSL dictates the performance of structural elements it also determines the durability criteria. As the first step towards durability specifications, it is suggested to include a reference guideline for the service life period in SANS 10100-2, though this is often determined by the client. This guide will govern the client with regard to the possible design life specifications. Typical examples are shown in Table 7.3 as presented in different guideline and design standards such as EN 1990 (2002).

Table 7.3: Recommended service life for different structure types (per EN 1990, 2002)

Design Service Life Category	**Indicative Design Service Life (years)**	**Examples of Structures**	**Design Methodology (specifications)**
1	10	Temporary	Prescriptive
2	10 to 25	Replaceable Structural Parts	
3	15 to 30	Agricultural and Similar Structures	
4	50	Building and Other Common Structures	Prescriptive/hybrid
5	100	Monumental Building Structures, Bridges and other Civil Engineering Structures	Performance-based

Conditions of exposure differ significantly from region to region and so selection of proper environmental action is vital prior to design of any RC structure. It is noted that the rate of deterioration in RC structures depends also on climatic conditions during the service life of the structure (Stewart et al., 2011). As the second step towards durability specifications, it is suggested to include a map of South Africa in SANS 10100-2 that indicates different climatic zones. In addition to give the specifier a thorough knowledge with regard to the risks involved in a certain climatic zone, the map will form the ground for preparing durability requirements. Figure 7.1 is a map of South Africa showing different climatic zones (*i.e.* macro or meso-climates).

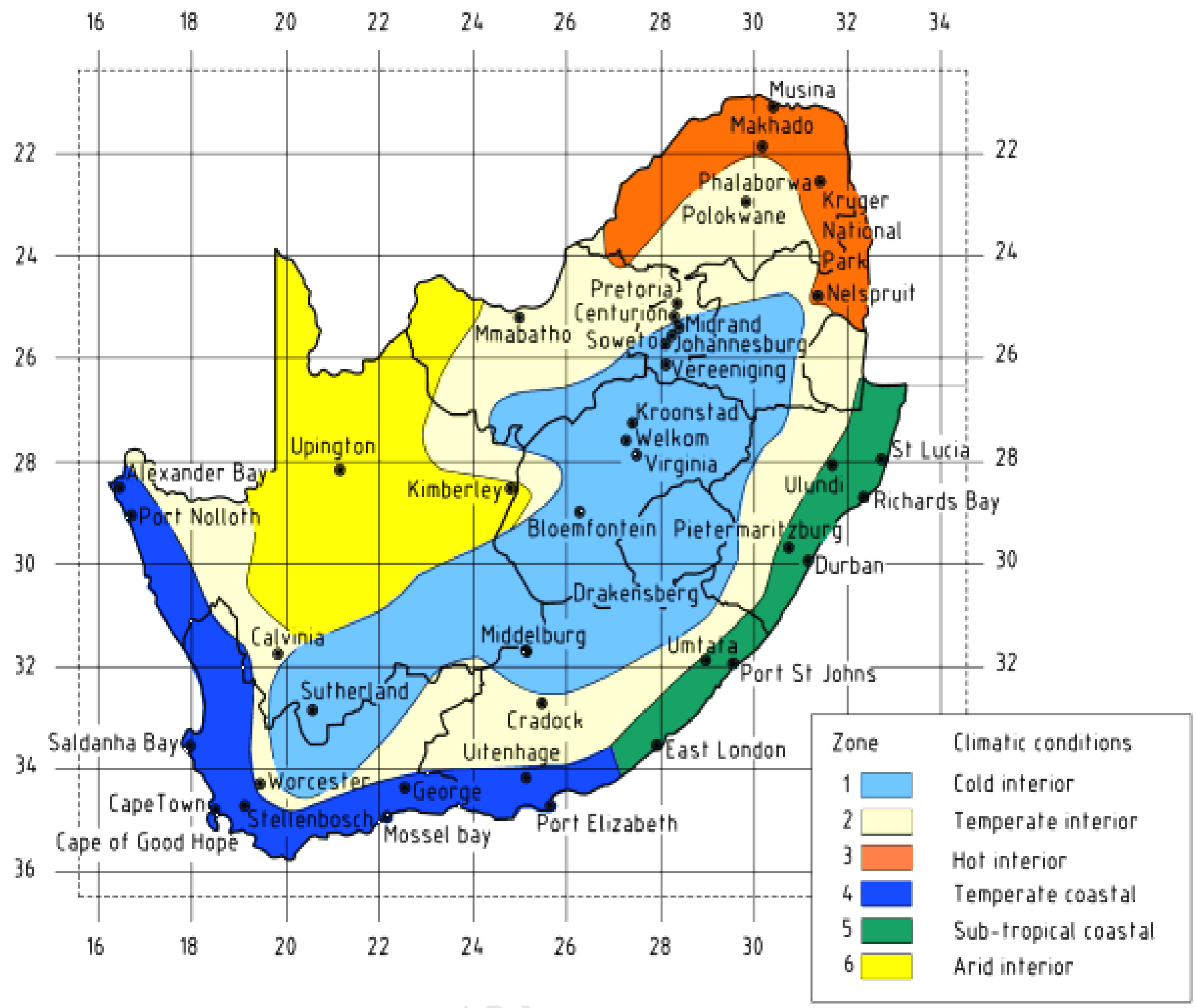

Figure 7.1: Climatic zones for South Africa
(www.insulpro.co.za/TIASA climate_zones_south_africa.htm)

Micro or meso-climates solely do not determine the causes of deterioration in RC structures. Only micro-climate classifications (*i.e.* immediate environment surrounding a structure) can give sufficient durability requirements (Haque et al., 2007). Suggestions have been made to adopt exposure conditions developed in EN 206-1(Alexander et al., 2008; SANS 10100-2 2009), in particular for carbonation and marine conditions. However, other exposure conditions such as freeze/thaw, sulfate, and chlorides from sources other than sea water that may impair concrete durability should also be considered. The following section illustrates the proposed exposure classifications based on EN 206-1.

7.3.1 Proposed Exposure Classifications in Revised SANS 10100-2

This section presents proposed exposure classifications in revised SANS 10100-2, which is a modified version of Table 1 in EN 206-1. Additional classes for concrete requiring low permeability is considered to cover all concretes exposed to continuous contact with water e.g. water-retaining structures. This class is not defined in EN 206-1 but it is recommended in section 6.2.1.2 of SANS 10100-2.

Chloride from sources other than sea water, temperature variations and fluctuations in the levels of moisture may also induce corrosion and result in deterioration. It is therefore suggested to consider all exposure conditions as defined in EN 206-1 with slight modification to suit South African conditions. Modifications have been made for critical exposure classes e.g. chloride exposure classes from sea water and possible freeze/thaw exposure in the interior parts of the country, i.e. cold climate such as Lesotho. For sulfate exposure from soil and/ or ground water, modifications have been made by classifying the degree of severity into classes while in EN 206-1 the exposure is given in general form of chemical attack.

For general application, the proposed exposure conditions may be classified into six main categories based on different degradation mechanisms as shown below:

- ‘XC’ for concrete exposed to carbonation-induced corrosion,
- ‘XS’ for concrete exposed to chlorides-induced corrosion from sea water,
- ‘XD’ for concrete exposed to chlorides-induced corrosion other than sea water,
- ‘XF[#]’ for concrete exposed to freezing and thawing with or without de-icing salts,
- ‘XA[+]’ for concrete exposed to sulfate attack, and
- ‘XP’ for concrete requiring low permeability.

These categories are further sub-divided into 19 sub-classes as shown in Table 7.4. Moreover, it would be appropriate to include a class ‘X0’ i.e. “no risk of attack” in order to reduce confusion regarding the possibility of a particular degradation mechanism.

Classifications of environmental actions from macro to micro-climates will reduce conflicts when one needs to select correct exposure class. In this case, a structure can be specified according to its location while at the same time each member can be also specified with respect to a particular exposure class. Figure 7.2 provide a typical example of different exposure classes for individual members of RC structure.

Table 7.4: Proposed exposure classifications in revised SANS 10100-2 (per EN 206-1: 2000)

Class designation	**Description of the environment**	**Informative examples where exposure classes may occur**
X0	No risk of attack	Applies to all exposure categories where there is no risk of attack *e.g.* reinforced concrete in a very dry condition
Corrosion induced by carbonation (Reinforced Concrete exposed to air and moisture)		
XC1	Permanently dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact *e.g.* many foundations
XC3	Moderate humidity (60% - 80%)	Concrete inside buildings with moderate or high air humidity. External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure Class XC2
Corrosion induced by chlorides from sea water (Reinforced Concrete in contact with chlorides from sea water or air carrying salt originating from sea water)		
XS1	Exposed to airborne salt but not in direct contact with sea water	Reinforced concrete surfaces near to or on the coast
XS2a*	Permanently submerged	Reinforced concrete surfaces completely submerged and remaining saturated, *e.g.* concrete below mid-tide level [A)]
XS2b*	XS2a + exposed to abrasion	As above, but with heavy wave action where abrasion occurs
XS3a*	Tidal, splash and spray zones	Reinforced concrete surfaces in intertidal, splash, or spray zones [B)]
XS3b*	XS3a + exposed to abrasion	As above, but with heavy wave action where abrasion occurs
Corrosion induced by chlorides other than from sea water (Reinforced Concrete in contact with water containing chlorides, including de-icing salts, from other sources)		
XD1	Moderate humidity	Concrete structures exposed to airborne chlorides Parts of structures exposed to slightly chloride conditions
XD2	Wet, rarely dry	Reinforced concrete surfaces totally immersed in water containing chlorides [A)]
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides *e.g.* pavements and car park slabs
Freeze/thaw attack with or without de-icing agents (For concrete exposed to significant attack by freeze/thaw cycles whilst wet)		
XF1[#]	Moderate freeze/thaw attack	Concrete members exposed to freeze/thaw cycles and occasional exposure to rain or water
XF2[#]	Severe freeze/thaw attack	Concrete exposed to freeze/thaw cycles and in continuous contact with accumulated water *e.g.* horizontal surfaces
FX3[#]	Very severe freeze/thaw attack	Concrete members exposed to frequent splashing with water containing de-icing agents and exposed to freezing
Sulfate attack (For concrete exposed to sulfate attack from natural soils and ground water according to Table 7.4)		
XA1[+]	Moderate sulfate attack	Natural soils and ground water
XA2[+]	Severe sulfate attack	
XA3[+]	Very severe sulfate attack	
Permeability resistance		
XP1	Exposure to water or moisture	Reinforced concrete surfaces requiring low permeability *e.g.* members in continuous contact with water

Notes:

1. [*] Additional sub-clauses for South African coastal conditions.

2. [A)] Concrete members where one surface is immersed in water containing chlorides and another is exposed to air are more potential, especially where the dry side is at a high ambient temperature. Specialist advice should be sought where appropriate, to develop a specification that is appropriate to the actual conditions likely to be encountered.

3. [B)] Exposure XS3a* covers a range of conditions. The most extreme conditions are in the spray zone. The least extreme is in the tidal zone where conditions can be similar to those in XS2a*. It is recommended to take into account the most extreme conditions within this class.

4. [#] Combination of exposure classes in EN 206-1, i.e. XF1[#] represents XFI and XF2 (because of similarities in terms of material compositions and proportions) while XF2[#] and XF3[#] reflects XF3 and XF4 respectively.

5. [+] Sulfate attack from natural soil and ground water.

Figure 7.2: Typical example of exposure classes for elements of a RC structure

7.3.2 Prescriptive and performance Approaches in Revised SANS 10100-2

In order to bring SANS 10100-2 up-to-date following international trends, two types of specifications are considered i.e. prescriptive and performance specifications. These specifications are shortly described below.

7.3.2.1 Prescriptive Specifications in Revised SANS 10100-2

It should be noted that performance specifications are not intended to completely replace current prescriptive specifications in SANS 10100-2, but rather an alternative approach to be considered. Moreover, performance specifications may not be suitable for every project at every location and therefore they apply if they suit particular requirements. Depending on the importance of the structure, it may be reasonable to assume that prescriptive specifications are suitable for common structures with limited service life. Moreover, in cases where test methods are unavailable, expensive, take time or any specific shortcomings with a particular test method for evaluating performance, prescriptive specifications may govern. Furthermore, prescriptive specifications have shown to comply well with specific durability problems such as ASR, abrasion, aggressive chemical, etc. In this context, prescriptive specifications should be retained in the standard with greater emphasis on performance through innovation.

Steps have been undertaken to widen prescriptive specifications in SANS 10100-2. Among the critical step is to widen the limits on concrete compositions and proportions to align with the expanded exposure classes described in section 7.3.1. The following section illustrates proposed limiting values for concrete compositions and properties in the revised SANS 10100-2.

7.3.2.1.1 Proposed Limiting Values for Concrete Compositions and Properties in Revised SANS 10100-2

This section presents proposed limiting values for concrete compositions and properties in revised SANS 10100-2 in terms of maximum w/c, minimum cement content, and minimum strength class based on the requirements presented in EN 206-1. As can be shown in Table 7.5, air content limits for freeze/thaw exposure are based on the requirements given in SANS 10100-2, which are almost similar to the requirements in EN 206-1. In terms of cover depth requirements, Alexander et al. (2008) recommend minimum cover of 30 mm for carbonation and 50 mm for sea water exposure. Following these minimum requirements, cover depth requirements are proposed based on the comparison between EN 1992-1-1 (2004) and SANS 10100-2 as EN 206-1 does not provide cover depth requirements.

Types of cements or SCMs are not included in Table 7.5, but their application should comply with the South African specifications recommended in SANS 10100-2. Chloride and sulfate limits for fresh concrete mix are given in the table in form of clauses.

An upper limit of 550 kg/m^3 of total cementitious binder content given in SANS 10100-2 seem to be very high and therefore a limit of 450 kg/m^3 is suggested. It should be noted that the minimum cement content limits for each exposure class in EN 206-1 covers only Portland cement. However, flexibility for cement plus addition is allowed in EN 206-1. EN 206-1 also requires water/(cement + addition) to have less than the maximum value for a particular exposure class. This complies with water/binder requirements presented in SANS 10100-2, e.g., for extreme freeze/thaw attack. In this regard, it may be reasonable to propose similar requirements for other exposure classes as presented in EN 206-1.

For “no risk” class i.e. ‘X0’, minimum strength class C12/15, maximum w/c of 0.65, minimum cement content of 260 kg/m^3, and minimum cover depth of 20 mm are suggested as recommended in EN 206-1.

The recommended limiting values in Table 7.5 are based on 50 year service life assumption developed in EN 206-1. However, various limiting values such as compressive strength and cover depth requirements may need some validation through laboratory work in order to develop limits that suit South African conditions.

Table 7.5: Limiting values for concrete compositions and properties in revised SANS 10100-2 (per EN 206-1: 2000)

Exposure category	**Exposure class**	**Maximum w/c ratio#**	**Minimum cement content kg/m^{3}#**	**Air content for 14-20 mm aggregate size, (%)**	**Minimum strength class (MPa) at 28-day**	**Minimum nominal cover (mm)**	**Additional requirements****
XC carbonation	XC1	0.65	260	-	25	30	
	XC2	0.60	280	-	30	30	
	XC3	0.55	280	-	37	40	
	XC4	0.50	300	-	37	45	
XS chloride from sea water	XS1	0.50	300	-	37	60	Total acid-soluble chloride content in fresh concrete should not exceed the values in clause 6.1.7
	XS2a*	0.45	320	-	45	60	
	XS2b*	0.45	320	-	45	60	
	XS3a*	0.45	340	-	45	60	
	XS3b*	0.45	340	-	45	60	
XD chloride other than sea water	XD1	0.55	300	-	37	60	
	XD2	0.55	300	-	37	60	
	XD3	0.45	320	-	45	60	
XF Freeze/thaw attack	XF1$^{\#}$	0.55	300	4 - 5	37	50	
	XF2$^{\#}$	0.50	320	4 - 5	37	50	
	FX3$^{\#}$	0.45	340	4 - 5	37	50	
XA^{+} Sulfate attack	XA1^{+}	0.55	300	-	37	50	Total acid-soluble sulfate content should not exceed values in clause 6.1.8
	XA2$^{+A)}$	0.50	320	-	37	50	
	XA3$^{+A)}$	0.45	360	-	45	50	
XP penetrability	XP1	0.50	300	-	37	40	

Notes:

1. $^{\#}$ Water/(cement + addition) ratio shall not exceed the given value for a given class of exposure while for total cement content plus addition, at least equal minimum value for a particular exposure class shall be obtained. Upper limit of 450 kg/m^3 of total cementitious binder content shall be adhered with.

2. $^{A)}$ When sulfate leads to exposure Classes XA2^{+} and XA3^{+}, it is essential to use sulfate-resisting cement. Where cement is classified with respect to sulfate resistance, moderate sulfate-resisting cement shall be used in exposure Class XA2^{+} and high sulfate-resisting cement shall be used in exposure Class XA3^{+}.

3. For no risk class i.e. 'X0', minimum strength class of 15 MPa is recommended with a maximum water/cement ratio of 0.65, minimum cement content of 260 kg/m^3, and minimum cover to the reinforcing steel of 20 mm.

4. **Chloride and sulfate contents that is contributed from concrete constituents including water, aggregates, cementitious materials, and admixtures shall be determined in accordance with SANS 1083 and SANS 5213.

7.3.2.2 Performance Specifications in Revised SANS 10100-2

SANS 10100-2 should consider implementation of performance specifications including hybrid and PBS based on DI tests, particularly for CCI and OPI. In this way, compressive strength, cover depth and DI values should be specified as the basic requirements for evaluating performance. Since PBS are normally linked with SLMs, its implementation in SANS 10100-2 may not be an easy task as more field data from periodic monitoring of structures in actual environment is still required. For projects conducted in South Africa, positive results have been obtained with DI values when used with SLMs e.g. UCT Model. It is therefore reasonable to predict that performance-based alternatives are currently underway to be included in SANS 10100-2, as an integrated approach, when enough confidence is gained with DI values and SLMs. In this context, hybrid specifications may be considered as an efficient and relatively simple way to increase the service life of RC structures prior to fully implementation of PBS in SANS 10100-2. The following section illustrates proposed hybrid specifications to be considered in the revised standard.

7.3.2.2.1 Proposed Hybrid Specifications in Revised SANS 10100-2

This section presents proposed hybrid specifications based on DI values for OPI and CCI for exposure in carbonating and chloride from sea water conditions. For other exposure conditions including freeze/thaw, sulfate, and chloride from sources other than sea water, pre-qualification tests may be used and the acceptance of concrete will based on compliance with the compressive strength requirements. Currently, there are no standardized test methods for evaluating resistance of concrete to freeze/thaw and sulfate attacks in South Africa. In this way, it may be reasonable to adopt relevant test methods in additional to complying with the requirements presented in section 7.3.2.1.1. This test methods include ASTM C457 *"Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete"* for resistance to freeze/thaw attack and ASTM C1012 *"Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution"* for measuring expansion due to sulfate attack. They may be suitable for characterizing concrete for durability based on sufficient acceptance criteria.

Table 7.6 suggests DI values as recommended by Alexander et al. (2008) for common RC structures requiring 50 years service life. As shown in the table, only critical classes for carbonation i.e. XC3 and XC4 are considered while for less critical classes i.e. XC1 and XC2 a good quality concrete and a cover depth of 30 mm is suffice to provide protection. Moreover, as shown in the table, both prescriptive requirements and performance recommendations are included. However, either prescriptive or performance may be selected but not both. In selecting performance requirements, specified compressive strength, cover depth, and DI values should satisfy the recommended values as the basis for acceptance.

Table 7.6: Proposed hybrid specifications in new revised SANS 10100-2

Exposure class	**Prescriptive alternative for resistance to fluid penetrability**			**Performance alternative for resistance to fluid penetrability Durability Index - 50 years service life (common structures)**			
	Maximum w/c ratio	**Minimum strength class (MPa)**	**Minimum nominal cover (mm)**				
Exposure to chlorides from sea water				Maximum Chloride Conductivity (mS/cm)[a]			
				Typical Binder Blends			
				70:30 CEM1:FA	50:50 CEM1:GGBS	50:50 CEM1:GGCS	90:10 CEM1:CSF
XS1	0.50	37	60	3.00	3.50	4.00	1.20
XS2a*	0.45	45	60	2.45	2.60	3.25	0.85
XS2b*	0.45	45	60	1.35	1.60	1.95	0.45
XS3a*	0.45	45	60	1.35	1.60	1.95	0.45
XS3b*	0.45	45	60	1.10	1.25	1.55	0.35
Exposure to carbonation				Minimum oxygen permeability index(log scale)[b]			
XC3	0.55	37	30	9.7			
XC4	0.50	37	50	8.7			

Notes:

1. The requirements in Table 7.6 shall apply for either prescriptive or performance, but not both.
2. [a] The maximum values that shall not be exceeded in the as-built structure, tested on sample removed at 28 days.
3. [b] The minimum OPI value that shall be achieved in the as-built structure, tested on sample removed at 28 days.

The DI values presented in Table 7.6 are based on the actual deterioration rates monitored over periods of up to 10 years (Alexander et al., 2008) and have shown successful performance. Moreover, such values have been adopted by SANRAL and have been used in several projects for quality control.

Under performance specifications, it is also recommended to undertake extensive testing to ensure that all cover depth meet specified requirements. This may be achieved when a performance concrete quality control is carried out and a document demonstrating compliance with the specified durability requirements is provided.

7.4 Durability Specifications Guideline in South Africa

Concrete durability specifications in South Africa require a change of mindset with respect to roles and responsibilities which is often assumed in the standard or in the industry in general. For instance, in SANS 10100-2, roles and responsibilities are not clearly defined and therefore risk normally devolves upon the concrete producer and/or contractor. Since each stakeholder involved may approach a project differently, there should be clear definitions of the roles and responsibilities with clear risk distribution of each stakeholder.

The general concept of PBS with respect to roles and responsibilities has been described in several publications (Bickley et al., 2006b; CSA A23.1/23.2, 2009; Carino et al., 2010). Based on the experience from these publications, this concept is modified to provide a general guideline towards durability specifications suitable for South Africa that include prescriptive, hybrid and PBS. This guideline is described in steps as shown below and summarized in Figure 7.3.

Stage 1: Specifications: Consists of the client's team which includes the client, the specifier, and the client's testing agency. The primary role of the client is to define the project in measurable quantitative terms i.e. the exposure conditions, DSL, functionality and the level of performance. If necessary, the client may also specify other requirements such as colour, texture, aesthetics, maintenance, etc. The client should recognize that different projects in different environments do not require similar specifications.

The specifier should establish the required specifications i.e. prescriptive specifications, hybrid specifications or performance-based specifications. In prescriptive specifications, the specifier may be governed with the limiting values for concrete compositions and properties presented in the standard. In case of performance, different scenarios imply and therefore the specifier needs to specify parameters in broader perspective. When PBS is considered, DI values for CCI and OPI are required to be incorporated in relevant SLMs for estimating DSL. In addition to satisfying DI values, obtained DSL may also be used as a criterion for evaluating the performance of concrete for durability. Durability indicators such as chloride permeability, electrical resistivity, absorption, etc., may not follow similar scenario since there are no SLMs currently available. In this case, hybrid specifications that rely on durability indicator may be considered for evaluating performance. Moreover, hybrid specifications may also be considered when there are specific shortcomings with SLMs under certain exposure conditions.

Generally, the specifier needs to specify the following: (a) service environment with respect to exposure classes, (b) standardized test methods and the parameters to be measured, (c) surrogate tests in case the means of assessing performance using specified test methods are impractical, (d) specify when and where the tests should be performed, (e) SLMs for estimating DSL of RC structure, and (f) construction specifications for the contractor.

Regardless of the type of specification, structural safety, constructability, and durability, are the key elements to be considered. Moreover, cost considerations, availability of local laboratories to carry out the tests to the desired precision as well as the importance of the structure should govern the specifier.

The client or the client's testing agency should specify the frequency of the test methods and the range for acceptability. Testing variability should be taken into consideration in order to establish adequate acceptance criteria. Generally, client's testing agency and the specifier should define the acceptance criteria, consequences for non-conformity and recommend appropriate measures to be taken. A bonus-penalty system may also be introduced at this stage to encourage the production of a high quality product that satisfies the client.

The client's team should establish a qualification system that evaluates the requirements for quality control. This system is responsible for evaluating the eligibility of the concrete producer and/or contractor based on their previous projects prior to award of a contract.

Stage 2: Construction and Testing: This stage includes team work between concrete producer and contractor. Although concrete producer and/or contractor work together to ensure the specified end results are achieved, each has their own responsibility as described below.

The primary role of the concrete producer is to prepare a mix design (*i.e.* select materials and their proportions) with the fresh properties required by the specifications. Environmental conditions such as temperature should be considered and then develop means to handle slump loss, setting time, and strength requirements. A series of pre-qualification tests such as slump, air content and temperature should be conducted. A quality control plan should be drawn up that ensures concrete delivered is not different from the pre-qualified concrete. Finally, the concrete producer should prepare a submittal document that demonstrates the compliance with the specified requirements in stage 1.

The contractor should determine the means and methods required to take the fresh concrete from the delivery point to the hardened state. In this regard, the contractor is responsible for placing, finishing, and curing concrete to meet the desired end results. Moreover, the contractor should conduct a quality control plan that verifies the concrete will satisfy the intended requirements.

The concrete producer and/or contractor should perform field tests (*i.e.* strength and durability tests) to evaluate concrete with respect to the type of specification. In case of prescriptive specifications, strength tests may be the only criterion for evaluating durability requirements. In terms of performance specifications, durability tests should be performed to measure relevant durability parameters set forth at *stage 1*. Generally, the concrete producer and/or contractor should prepare a submittal document at this stage that demonstrates the compliance with the performance requirements.

Stage 3: Verification: The client should be able to verify during the handover period that the durability requirements has been or will be satisfied based on criteria set forth at *stage 1*. This is where the consequences of compliance or non-compliance occur, which may sometimes involve bonus or penalty, particularly in performance specifications. At this stage, the contractor may be subjected to full payment sometimes with bonus upon full acceptance, reduction of payment upon conditional acceptance or penalty upon rejection. The payment method depends upon the criteria set forth at *stage 1*.

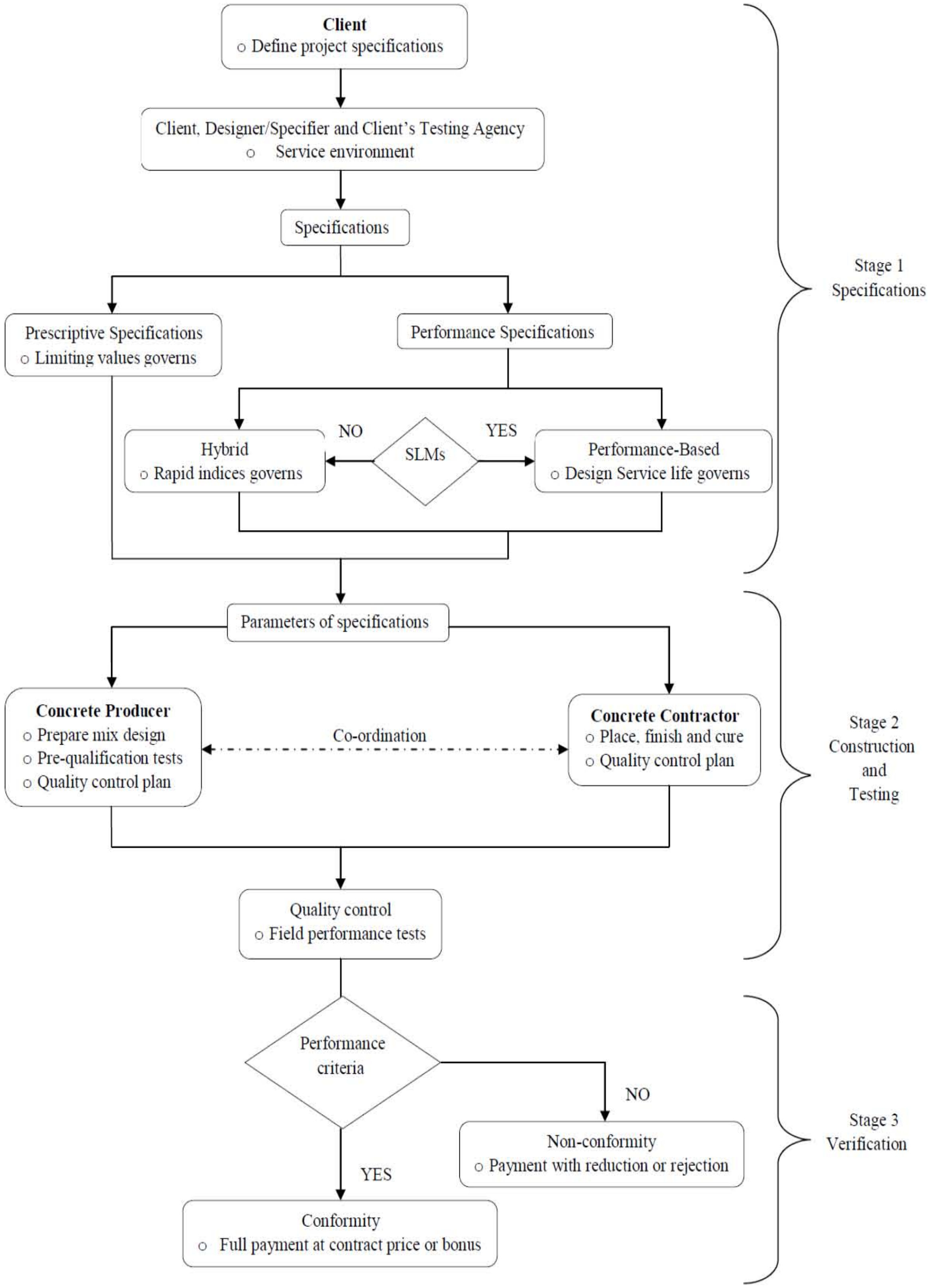

Figure 7.3: Flow chart illustrating durability specifications guideline in South Africa

Generally, committed and knowledgeable stakeholders are required in order to implement this guideline properly, in particularly for performance. Though the client's primary interest is to get the greatest possible return on the investment during the structure's life time, she should recognize the choice of requirements as they have a direct relationship with a particular type of specification. On the other hand the specifier has a final decision on durability specifications. Such decision should take into account a variety of inputs that satisfy the client's interests. In this context, the specifier must recognize the impacts of the specified parameters when specifying concrete for durability. The concrete producer and/or contractor must recognize their roles in producing, handling, placing, and curing concrete in order to satisfy the intended durability. An important point to consider is that each stakeholder should maintain frequent communication and a co-operative relationship in order to achieve the desired durability. Moreover, each stakeholder should be held accountable only for those quality characteristics under their control.

CHAPTER 8: CONCLUSIONS and RECOMMENDATIONS

8.1 GENERAL CONCLUSIONS

Concrete is versatile, economical, strong and durable, thus making it an ideal construction material for most infrastructure. So far, concrete dominates most types of infrastructure such as buildings, long-span bridges, bored tunnels, concrete pressure vessels etc. Its performance is vital for underpinning the nation's essential services and economic activities. Despite the remarkable work of concrete in terms of performance, it is now facing additional challenges with respect to durability. Recently, the world has experience severe premature deterioration in much important infrastructure that requires extensive repair and maintenance to restore serviceability. Though the consequences of premature deterioration threaten private and public sector budgets, they also have a great impact on natural resources and human safety in general.

South Africa like most other countries around the world relies on concrete infrastructure for the social and economic well-being. Many concrete structures in South Africa have performed satisfactorily to satisfy their intended serviceability, with the exception of some cases where structures indicated premature signs of deterioration. The predominant cause of premature deterioration for most structures in South Africa is related to corrosion of reinforcing steel due to either carbonation or chloride ingress. Other causes such as ASR, sulfate attack, freeze/thaw attack, chemical attack, abrasion, etc., though less common, are still a critical means of material deterioration. Macro and micro climates, material properties and proportions, and construction methods, all play vital roles in the performance of concrete structures. Some of these parameters are related to standards of practice currently used in construction industry in South Africa that fail to address properly durability requirements.

This dissertation conducted an in-depth review on the durability provisions for RC structures in AS 3600 (2001) and AS 1379 (2007), ACI 318 (2008), CSA A23.1/23.2 (2009), EN 206-1 (2000), IS 456 (2000), and SANS 10100-2 (2009). The review indicated that durability requirements in most design standards are based on prescriptive specifications that account for material properties and proportions and construction methods. Commonly specified parameters include limiting values for w/cm, compressive strength and cover depth with respect to exposure conditions. These limits are chosen based primarily on judgement to ensure intended performance. Though such limits may be sufficient for common RC structures, it is different for special structures requiring long-term performance particularly in aggressive environments.

There are several shortcomings with current prescriptive specification in the design standards when longer service life is required. For instance, though the specifications may encompass requirements for, inter alia, minimum compressive strength, maximum w/cm and cover depth, desired concrete performance is not necessarily described. These specifications do not account for material and construction variability, and are unable to ensure the intended performance. Several requirements e.g. maximum w/cm and minimum water content are difficult or impossible to measure or verify in practice. Further, these requirements often stifle innovation. Broadly, these specifications have limited application in today's concrete construction industry when considering long-term performance of concrete structures.

An alternative to prescriptive specifications that may ensure long-term performance particularly in aggressive environments is performance specifications. These specifications are intended to control variability of materials and construction methods through measuring relevant properties that account for durability. Performance specifications are considered relevant for solving many durability problems that threaten the performance of RC structures. They are becoming more common in the industry as the behaviour of concrete during its life can better be quantified prior to construction. These specifications generally impose few or no restrictions on the concrete constituents, proportioning, or construction methods, but rather promote innovations. In these specifications, the concrete producer and/or contractor can apply alternative techniques to produce durable concrete that meets the client's requirements.

While prescriptive specifications are still the norms, in many design standards including SANS, there are a few design standards that have successfully implemented performance specifications. CSA A23.1/23.2 and AS 1379, are examples of such design standards that specify performance. The specifications contained in these design standards are termed as 'hybrid specifications' as they still present some prescriptive requirements, including guidance in order to arrive at the required performance. Moreover, the design standards recommend test methods and acceptance criteria for evaluating performance. For instance, CSA A23.1/23.2 recommends RCPT in accordance with ASTM C1202 for critical chloride exposure, ASTM C457 for freeze/thaw exposure, and ASTM C1012 for sulfate exposure, each with suitable limits for acceptance.

Currently, there is no performance-based standard for concrete durability in South African specifications. What is needed is an integrated approach that links durability specifications, including durability indicators from relevant test methods, and durability design through SLMs in order to estimate DSL of RC structures. Development and implementation of such standards are not an easy task as there are still more challenges, including (a) lack of reliable, consistent and standardized test methods which can evaluate concrete performance in time or routinely, (b) lack of adequate SLMs which can capture the main aspects involved in deterioration including environmental-specific factors, and (c) lack of experience in developing sufficient performance requirements with appropriate acceptance criteria.

In summary, durability specifications should follow an iterative progress as shown in Figure 8.1. Such progress involves materials selection and optimization, safety and serviceability, and performance requirements for the service life required. The principle routes for undertaking durability specifications are either prescriptive or performance-based approaches, or more commonly a combination of the two i.e. “hybrid”.

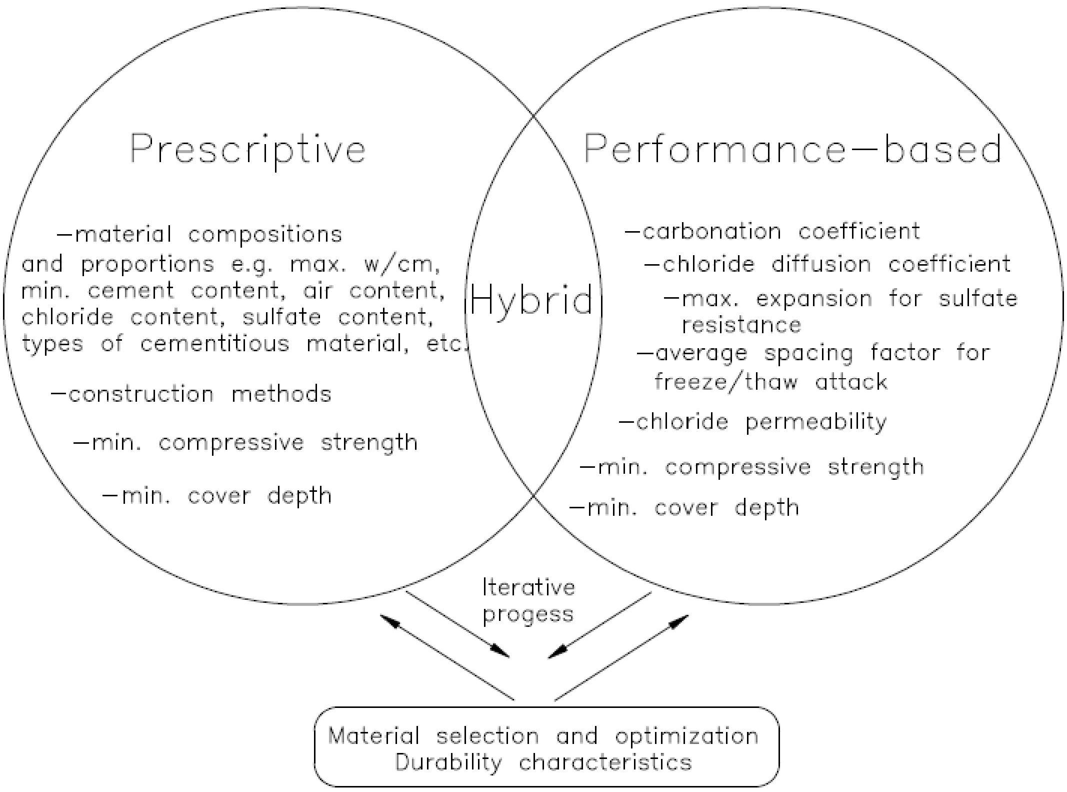

Figure 8.1: Iterative progress of concrete durability specifications

A critique of the durability provisions in SANS revealed several shortcomings with respect to long-term performance. Several recommendations required to bring SANS up-to-date following international trend were presented. Similar approach used in CSA A23.1/23.2 has been considered in SANS, where both prescriptive and performance requirements were presented. For prescriptive specifications, EN 206-1 is considered in terms of exposure classifications and limiting values for material compositions and proportions. In terms of performance, DI tests for CCI and OPI have been proposed in SANS 10100-2 for evaluating performance of concrete. It should be emphasised that DI values proposed are based on short-term monitoring of structures in actual environmental conditions and therefore suitable under hybrid specifications. More work is still required to produce refined DI values that are linked to SLMs for performance-based durability specifications.

8.2 RECOMMENDATIONS FOR FUTURE WORK

Concrete durability specification is a broad subject and therefore only certain issues that relate to material properties and environmental actions have been presented in this dissertation. It is important to acknowledge that much work on concrete durability has been done worldwide that can be used to improve the design standards. Nevertheless, more work through research and practical experience over time is still required to cover various durability requirements in a broader perspective. Among the issues which may require attention in near future to be included in SANS are shortly outlined and discussed below.

(a) Exposure classes based on proximity to the sea:

It is well-noted that the degree of corrosion damage resulting from chloride ingress varies significantly based on the proximity of the structure to the sea. SANS 10100-2 only considers structures located within 30 km from the sea as severe exposure conditions. This may need some modifications by providing more categories, at least to shorter distances from the coastline. A similar approach presented in AS 3600 (2001) may be adopting SANS 10100-2.

Recommendations with respect to different categories of the structures from the coastline have been given by Haque et al. (2008), specifically for the African continent. Table 8.1 illustrates these categories with possible attack mechanisms. Field tests are necessary in order to find the actual variations of the chloride content with the distance of the structure from the coastline so that a more scientific solution can be achieved. In this way, Table 8.1 may be used as a foundation during the development of durability requirements based on the location of the structures from the sea.

Table 8.1: Classification based on distance from the sea (Haque et al., 2008)

Exposure	**Distance from sea**	**Micro climates**		**Description of attack**
Marine Zone **M**	0-100 m from the shore	M_1	Spray	• Active corrosion due to aerosols and salts
		M_2	Splash/tidal	• Acute chloride-induced corrosion due to sea waves and current abrasion
		M_3	Submerged	• Minimum corrosion risk • Chloride and sulfate decomposition • Biological attack
Coastal Zone **C**	100 m from the shore up to 10 km			• Dampness on structures attracting salts and fungal growth • Chloride build-up from salt spray, soils and ground water • Carbonation due to high relative humidity (55–75%) • Sulfate-rich coastal soils induce sulfate attack
Inland Zone **I**	10-50 km	I_A	Within capillary rise zone	• Attack due to sulfates and chlorides present in soil and groundwater from either natural or industrial sources
		I_B	Above capillary rise zone	• Deterioration due to salt-weathering/ carbonation and/ or dry winds carrying aggressive salts
Low Risk zone **L**	50 km and above			• Occurrence of contamination or attack is low

(b) A concrete corrosivity map of South Africa:

In addition to exposure classes based on the proximity to the sea, it is recommended to develop a distress map for different geographical locations in South Africa particularly for corrosion. This map will provide good guidance with respect to corrosion-prone areas resulting from either carbonation or chloride ion attack. Development of the corrosion map of South Africa will depend upon the data collected from either field experience or laboratory. Availability of this map which may classify the country into classes with appropriate descriptions with respect to the degree of severity will improve the quality of concrete construction with respect to corrosion threats.

(c) Durability Service manual:

Experience has shown that there may always be durability potential deficiency in the long-term even if strict performance requirements are both specified and obtained (Arkog et al., 2006). For instance, the presence of numerous micro-cracks in concrete may still allow penetrability, e.g. chloride ingress. Neither the 28 day chloride diffusivity from laboratory specimens nor the achieved chloride diffusivity in the field accurately reflects the potential chloride diffusivity of a given concrete in the long-term. In this context, it is recommended to have a proper service manual that will be given to the client during handover for future monitoring of the expected service life of the structure (Gjørv, 2011). Among the parameters to be included in the service manual is the "birth certificate", which contains important information that defines the form and condition of the structure after construction. Specific details such as cover depth to the reinforcing steel, concrete permeability, environmental conditions, quality of construction achieved, etc., should be included in the "birth certificate". Such detailed information should act as a basis for monitoring the condition of the structure and for planning maintenance activates during its service life. Generally, inclusion of the service manual would provide the ultimate basis for achieving a more controlled durability and service life of RC structures.

APPENDIX A

Table A.1: Additional requirements for sulphate attack in CSA A23.1/23.2 (2009)

Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§††	Performance requirements§ Maximum expansion when tested using CSA A3004-C8, %	
						At 6 months	At 12 months††
S-1	Very severe	> 2.9	> 10000	> 2.0	HS or HSb	0.05	0.10
S-2	Severe	0.20-2.0	1500-10000	0.60-2.0	HS or HSb	0.05	0.10
S-3	Moderate	0.10-0.20	150-1500	0.20-0.60	MS, MSb, LH, HS, or HSb	0.10	

Notes:

1. †In accordance with CSA A23.2-3B.
2. ‡In accordance with CSA A23.2-2B.
3. §Where combinations of supplementary cementing materials and Portland or blended hydraulic cements are to be used instead of the cementing materials listed, the performance requirements shall be used to demonstrate equivalent performance against sulphate exposure (see Clauses 4.1.1.6.2, 4.2.1.1, and 4.2.1.3, and 4.2.1.4). Such combinations shall not be designated as blended cements.
4. **Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates. Refer to Clause 4.1.1.6.3.
5. ††If the expansion is greater than 0.05% at 6 months but less than 0.10% at 1 year, the cementing materials combination under test shall be considered to have passed.

Table A.2: Values of minimum cover concrete for RC structures in EN 1992-1-1 (2004)

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure class according to Table 3.11						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

A.3**:** Durability parameters acceptance range: OPI and cover depth (SANRAL, 2010)

Acceptance Criteria	OPI Requirements		Cover Depth Requirements	
	OPI (log scale)[a]	Percentage payment	Overall cover (mm)	Percentage payment
Full acceptance	> 9.70	100%	≥ 85% or < 100% +15 mm	100%
Conditional acceptance [b]	> 8.75 ≤ 9.70	80%	< 85% ≥ 75%	85%
Conditional acceptance [c]	-	-	< 75%	70%
Rejection	< 8.75	Not applicable	< 65%	Not applicable

Notes:

1. [a] OPI is measured in log scale, therefore the difference between the values is quite substantial
2. [b] With reduced payment
3. [c] With remedial measures as approved by engineer and reduced payment

Table A.4: Classification based on distance from the sea (Haque et al., 2007)

Exposure	**Distance from sea**	**Micro climates**		**Description of attack**
Gulf Marine Zone GM	0-100 m within the shore	GM_1	Spray	• Active corrosion due to aerosols and salts
		GM_2	Splash/tidal	• Acute chloride-induced corrosion due to sea waves and current abrasion
		GM_3	Submerged	• Minimum corrosion risk • Chloride and sulfate decomposition • Biological attack
Gulf Coastal zone GC	100 m from the shore up 10 km			• Dampness on structures attracting salts and fungal growth • Chloride build-up from salt spray, soils and ground water • Carbonation due to high relative humidity (55–75%) • Sulfate-rich coastal soils induce sulfate attack
Gulf Inland zone GI	10-50 km	GI_A	Within capillary rise zone	• Attack due to sulfates and chlorides present in soil and groundwater from either natural or industrial sources
		GI_B	Above capillary rise zone	• Deterioration due to salt-weathering/ carbonation and/ or dry winds carrying aggressive salts
Gulf Low Risk zone GL	50 km and above			• Occurrence of contamination or attack is low

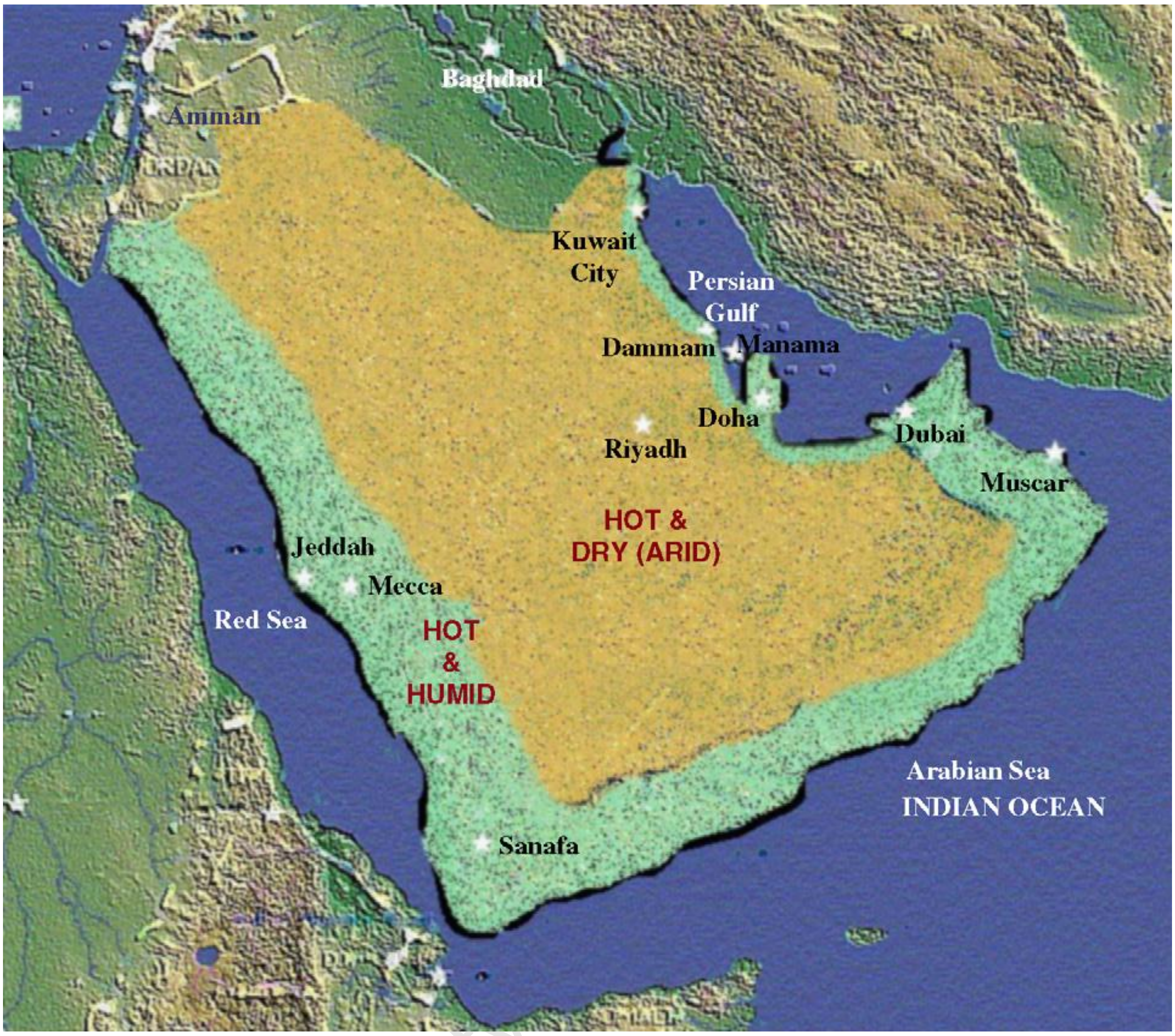

Figure A1: Hot-dry and hot-humid zones of the Arabian Peninsula (Haque et al., 2007)

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