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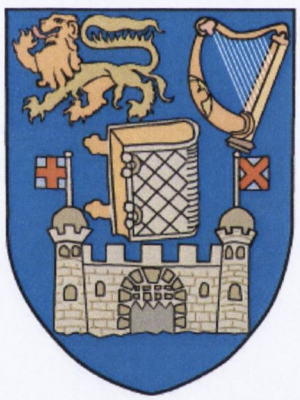
ASSESSMENT OF THE MARINE DURABILITY OF SELF-COMPACTING CONCRETE USING EXPERIMENTAL TESTING AND PROBABILISTIC MODELLING

A Thesis submitted to the University of Dublin in fulfilment of the requirements for the Degree of Doctor of Philosophy in the Faculty of Engineering

By

Paraic C. Ryan
BE, MSc
August 2013



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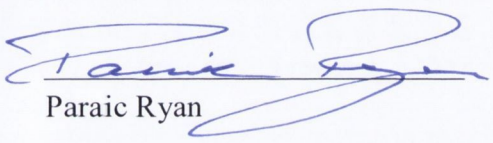

Paraic Ryan

August 2013

SUMMARY

Chloride induced corrosion is recognised as the main cause of reinforced concrete deterioration for structures located in marine environments. The primary aim of this thesis is to examine the marine durability of a number of Self-Compacting Concrete (SCC) options in the initiation phase of chloride induced corrosion, through laboratory testing, and subsequent probabilistic modelling. It is recognised in the literature that very limited information is available concerning chloride penetration in these increasingly popular SCCs. To date, only a small number of short term laboratory investigations have been published which explore the marine durability of different SCC options. Furthermore, due to the relatively recent development of the SCC technology, no long term information on the marine durability of SCC exists at present. Based these gaps in the existing literature, and shortcomings relating to popular laboratory testing methods used to assess marine durability, the following three core PhD research questions were developed in this thesis;

Research Question 1: How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

Research Question 3: How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques?

There are two parts to Research Question 1; a) how do different SCC options compare to one another, and b) how do the performances of SCCs of traditional vibrated concretes compare? Both parts were addressed in this PhD through examining chloride transport properties obtained from 1 year natural chloride migration tests carried out in salt fog chambers. This test apparatus incorporates wetting and drying cycles during exposure. For part a) of Research Question 1 the following five SCC options were examined; Ordinary Portland Cement (OPC), OPC + Ground Granulated Blast-Furnace Slag (GGBS), OPC + silane surface treatment and OPC + Pulverised Fuel Ash (PFA) with, and without, the *k* value efficiency factor applied. The OPC + GGBS SCC was found to be the most durable option with a chloride resistance performance of between 1.9 and 3.5 times that of the OPC SCC, depending on the extent of drying in the chloride exposure condition.

Chapter 5 of this thesis addressed Research Question 1 part b) through comparison of SCCs and VCs for three difference concrete types, namely OPC, OPC + GGBS and OPC + PFA with *k*-value applied, in a form of benchmarking process.The traditional vibrated concretes were found to be

more resistant to chloride ingress for the three concrete types assessed. The difference in performance was most pronounced for the OPC concretes, with the vibrated concrete having a chloride resistance 72% better than that of the equivalent SCC. For the OPC + GGBS concretes, however, the difference between the vibrated concrete and SCC performance was minimal, at just 16%. It should be noted that the VC specimens were well compacted in this study.

To address Research Question 2 the effect of exposure condition on the performance of OPC + GGBS SCC and OPC + PFA SCC, relative to OPC SCC, was examined across three sets of exposure conditions. It was found that the relative performance of the SCCs containing GGBS and PFA was notably reduced with increased levels of drying in the exposure condition. This is a particularly interesting finding in light of the fact that much of accelerated testing published in the literature to date is carried out in fully submerged conditions, while the critical chloride induced corrosion zones in real structures occur in areas which are subject to cyclic wetting and drying.

Research Question 3 was addressed in this PhD through probabilistic modelling incorporating the experimental testing results. The analysis indicated that, when considering timeframes applicable to service life, there is a substantial increase in the relative performance of the SCCs containing PFA and GGBS, when compared to the 1 year experimental testing. The OPC + GGBS SCC option experienced the greatest increase in relative performance, with OPC + GGBS SCC performing 3.25 times better than the OPC SCC for the experimental testing, and 12.5 times better than the OPC SCC in the calibrated probabilistic analysis. The performance of the OPC + PFA SCCs approximately doubled when compared to OPC SCC, from the 1 year experimental results to predicted long term performance. The improved performances of the SCC containing PFA and GGBS were primarily due to the increased influence of time dependant diffusion over service life timeframes. The calibration of the experimental results with the data from Ferrycarrig Bridge was not found to have a notable effect on the relative performance of the SCCs containing PFA or GGBS. But, the calibration did result in large increases in the predicted T_i values for all the SCCs.

Finally, a probabilistic marine durability case study assessment for Ferrycarrig Bridge was carried out in this thesis. This assessment facilitated comparison of the OPC with increased cover SCC option with the other SCC options. The case study also allowed probability based predictions to be made as to the likely time to initiation of corrosion in the Ferrycarrig Bridge crosshead beams. The predicted long term performance of OPC SCC with an additional 20mm cover was approximately 2.1 to 2.6 times better than OPC SCC with standard cover. The predicted times to initiation of corrosion at Ferrycarrig bridge were 60 years for the OPC crosshead beam and 140 years for OPC with increased cover crosshead. The analysis suggested that corrosion will not occur in the GGBS crosshead over the service life of the structure. It should be noted, however, that the COV values associated with these long term predictions were high, ranging from 0.67 to 0.91.

Dedicated to the memory of my beloved Father,
whose courage and strength in his two year fight with cancer has taught me that no battle in life is too tough, no challenge too great.

In Iothlainn Dé go gCastar Sinn

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NOMENCLATURE

C (x,t)	Percentage chloride content at depth x and time t
C_s	Surface chloride content,
C_{cr}	Critical chloride content
C_i	Initial chloride content in the concrete
C_d	Cover depth
Cal_1	Calibration factor 2
Cal_2	Calibration factor 1
CDF	Cumulative distribution function
CI	Corrosion Inhibitor
Conf. Int.	Confidence interval associated with the mean value estimate
COV	Coefficient of variation
cu	Cube specimens
D	Diffusion coefficient (general or unspecified)
D_{app}	Apparent diffusion coefficient
D_r	Instantaneous diffusion coefficient at reference time t_r
erf	The error function,
GGBS	Ground granulated blast-furnace slag
k	k value efficiency factor specified in EN 206-1:2000
LOI	Loss of Ignition
m	time dependent diffusion coefficient
N	number of samples in the sample set
NRA	National roads authority
OPC	Ordinary Portland cement
PDF	Probability distribution function
PFA	Pulverised fuel ash
RC	Reinforced concrete
RCMT	Rapid Chloride Migration Test
RCMT	Rapid Chloride Penetration Test
RH	Relative humidity
RM	Relative Merits (used when comparing OPC concretes to alternative concrete options such as OPC + GGBS and OPC + PFA)
RP	Relative performance (used when comparing SCC to VC)

s	Standard deviation associated with the sample set
SCC	Self Compacting Concrete
SCM	Supplementary Cementing Material
SFC	Salt fog chamber
sl	Slab specimens
t	Exposure duration or time
t_c	Time at which the diffusion coefficient becomes constant and t is the time being considered
t_{ex}	Age of the concrete at the time when samples were initially exposed to chlorides.
t_s	Test statistic obtained from the t-distribution table
T_i,	Time to initiation of corrosion.
t_r	Reference time for instantaneous diffusion coefficient
VC	Vibrated concrete
w/b	water/binder ratio
W_t	Weight at time, t, in the soaking tank
W_{dry}	Dry weight of the specimen
x	Depth below concrete surface
$\bar{x}$	Mean value estimate of the true mean based on the sample set
α	The confidence with which the interval is concerned i.e. an α value of 0.05 corresponds to a 95% confidence interval.
μ	Mean value

Concrete Types

OPC option = Ordinary Portland Cement Concrete

OPC-S = Ordinary Portland Cement Concrete with silane surface treatment

OPC-CI = Ordinary Portland cement concrete with mixed in corrosion inhibitor

OPC + GGBS = Concrete with ground granulated blast-furnace slag used as a direct replacement for Ordinary Portland Cement Concrete

OPC + PFA = Concrete with pulverised fuel ash used as a direct replacement for Ordinary Portland Cement Concrete

OPC + PFA (k) = Concrete with pulverised fuel ash used as a replacement for Ordinary Portland Cement Concrete with the k value applied in accordance with EN 206-1:2000

SCC = Self compacting concrete

VC = Vibrated concrete

1. INTRODUCTION

1.1 INTRODUCTION

Reinforced concrete is the predominant construction form used in buildings, bridges, power plants, wharves, and other infrastructure in the US, Europe, China and around the world (Stewart et al. 2012). These Reinforced Concrete (RC) structures are subject to deterioration with age, especially when located in aggressive environments, resulting in substantial annual maintenance costs. For instance, the annual worldwide cost of deterioration due to corrosion alone is estimated to be $1.8 trillion (Schmitt et al. 2009). In the United States it is estimated that 24.9% of the bridge infrastructure network has deteriorated to the point of being either structurally deficient or functionally obsolete, with a $20.8 billion annual investment required to eliminate the investment backlog for the United States bridges by 2028 (ASCE 2013). In the United Kingdom it was estimated in 2001 that the annual cost of bridge maintenance for the national road bridge network was €225 million (BRIME Project et al. 2001). Given the scale of the direct and indirect costs associated with RC infrastructure deterioration, it is reasonable to conclude that any increase in the durability performance of concrete structures will result in substantial reductions in annual maintenance costs in the future.

It is not surprising then, that large volumes of research have been carried out to date to reduce the susceptibility of RC to the primary causes of deterioration. One of the main causes of concrete deterioration is chloride induced reinforcement corrosion (Val and Trapper 2008). This deterioration mechanism is recognised to be the primary cause of degradation of concrete for marine structures, and structures subject to de-icing salts (Bastidas-Arteaga et al. 2011; Stipanovic Oslakovic et al. 2010; Yuan et al. 2009). Considerable research efforts in this area over the past number of decades have led to an acute awareness of the issue of chloride induced corrosion, resulting in changes to International design codes (Centre Europeen de Normalisation CEN 2000), and the development of durability based design practices (CEB-FIB 2006). Research efforts have also led to the development of new methods to increase the marine durability of concrete through the use of corrosion inhibitors (El-Hacha et al. 2011; Elsener and Concrete 2001; Luo and Schutter 2008; Page et al. 2000; Soylev et al. 2007; Soylev and Richardson 2008), chloride resistant concrete surface treatments (Basheer et al. 1998; de Vries and Polder 1995; McCarter 1996; Medeiros and Helene 2009; Schueremans et al. 2007), and perhaps most importantly, through the use of Supplementary Cementing Materials (SCMs) as partial replacements for Ordinary Portland Cement (OPC) (Dhir et al. 1996; Dhir et al. 1997; Dias et al. 2003; Mangat and Molloy 1991; McPolin et al. 2005; Pack et al. 2010; Stanish and Thomas 2003; Thomas and Matthews 2004; Thomas et al. 2008).

Despite the considerable amount of research carried out to date to develop sustainable, low maintenance RC solutions, a number of significant shortcomings in the understanding of the marine durability of concrete still remain. Firstly, it is noted that the concrete industry is dynamic in nature, with new concrete types constantly being developed. One of the most significant technology developments in this area in decades is the introduction of Self-Compacting Concrete (SCC) (Sonebi and Nanukuttan 2009). This concrete, which consolidates under its own weight without a need for compactive effort, was developed in the mid-1980s in Japan, and introduced in Europe in the mid-1990s (EFNARC et al. 2005). Its growth in popularity has been termed a 'quiet revolution' in the concrete construction process (Gesoğlu and Güneyisi 2011). However, despite the widespread acceptance of SCC in the construction industry (Hwang and Khayat 2009), little is known about the marine durability characteristics of this relatively new concrete type, especially for SCC containing SCMs utilised as partial cement replacements (Nehdi et al. 2004). In 2008 this shortcoming in existing knowledge was re-iterated by a publication authored by the RILEM Technical Committee for SCC (Rilem Technical Committee et al. 2008). This paper pointed out that fundamental differences in the mix designs for SCC and vibrated concrete (VC), and the absence of vibration in SCC, could lead to differences in the pore structure and durability behaviour. The RILEM Technical Committee paper also stated that, due to the very limited information available in the literature concerning chloride penetration in SCC, this concrete type is being used without a general and fundamental insight into the intrinsic durability of the material itself. Other authors have highlighted the lack of studies which compare the marine durability of SCCs to equivalent VCs (Sonebi and Nanukuttan 2009; Zhu and Bartos 2003; Zong et al. 2011). The work in this thesis seeks to address these issues by examining the marine durability of a number of SCCs and VCs in the initiation phase of chloride induced corrosion. The work herein does not consider the propagation phase of reinforcement corrosion. The basis for the SCCs examined is derived from the repair of Ferrycarrig Bridge, which was repaired using five different SCC options in 2007, as will be discussed in detail in Chapter 2.

Another shortcoming in the existing literature relates to the manner in which the marine performance of different concrete types are evaluated and compared. Due to the slow nature of chloride ion ingress much of the information pertaining to the comparison of different concretes is derived from accelerated testing (Castro et al. 1997; Li 2001). As will be highlighted in Chapter 2, the majority of this accelerated testing is carried out in fully saturated conditions. However, it is recognised in the literature that the critical chloride induced corrosion zones in RC marine structures occur in the splash, spray and tidal zones, where the concrete is subject to periodic wetting and drying (Neville 2011; Ye et al. 2012). Consequently, much of the information available relating to the relative performance of different concretes, (i.e. OPC and concretes containing

SCMs) is obtained in an exposure condition which is, in general, not critical in the real marine environment. As will be discussed in Chapter 2, there is merit to testing in fully submerged conditions, however, there is also a need to compare different concretes types across a range of saturation conditions. However, as pointed out by Ben Fraj et al. (2012), experimental studies examining chloride ingress in non-saturated conditions are not well developed in the literature.

The final shortcoming in the literature discussed in this section is also related to the slow nature of chloride ion ingress in the real marine environment. This slow deterioration process means that changes in the concrete characteristics over time will have a notable influence on the durability of marine structures. However, as stated above much of the information relating to comparisons of different concrete types is obtained from short term accelerated tests. In the case of SCCs there is currently no long term marine durability performance data available due to the recent development of this concrete technology. A relatively small number of long term studies do exist in the literature for vibrated concretes (Mohammed et al. 2002; Pack et al. 2010; Schueremans et al. 2007; Thomas and Matthews 2004; Thomas et al. 2008). However, to understand time dependant effects for different concrete types, chloride profiles must be obtained at a number of different times throughout the exposure period (Thomas and Bamforth 1999). The large investment of time and finances required to obtain information on effects such as time dependant diffusion means there is a lack of information relating to these parameters for different concrete types (Nokken et al. 2006).

Consequently, there is considerable uncertainty surrounding the extrapolation of short term test data to timeframes applicable to structure service life. However, as pointed out by Van den Heede (2012), short term test results alone are insufficient, as they give little indication as to the potential life span of different concretes. Thus there is a need to carry out modelling which recognises high levels of uncertainty. The most powerful tool for this type of analysis is probabilistic assessment (Ferreira 2010). Yet, studies in the literature using probabilistic methods to compare the performance of different concrete types over time periods applicable to service life are limited in the existing literature (Bentz 2003; Ferreira 2004; Ferreira 2010; McNally and Sheils 2012; Schueremans et al. 2007; Stewart et al. 2012; Van den Heede et al. 2012). Consequently, it was stated by Van den Heede (2012) that sufficient knowledge on the actual service life and sustainability of different concrete types is lacking.

Three core PhD research questions were developed based on the three significant shortcomings outlined above. These research questions are presented in the next sections, together with the means by which the research questions are addressed in this thesis.

1.2 CORE PhD RESEARCH QUESTIONS

The core research questions for the PhD sought to partially address the significant shortcomings outlined in the previous section concerning; a) performance of SCCs, b) the lack of information relating to differences in likely long-term performance of concretes, specifically SCCs, and the c) the effect of wetting and drying on relative performance of concretes. As previously mentioned these areas are explored herein in the context of the initiation phase of chloride induced reinforcement corrosion. The research questions, and the approaches used to investigate these research questions in this PhD, are listed below;

Research Question 1: How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

This research question was addressed in this PhD in two parts as follows the:

a) A comparison was carried out between the chloride transport properties of a number of different SCC options through natural chloride migration laboratory testing which incorporated wetting and drying cycles
b) The same experimental techniques were used to compare SCC to the more established traditional vibrated concrete type for both OPC concretes and concretes with SCMs used as partial cement replacements

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

This research question was addressed herein by examining the effect of three different chloride exposure conditions on the relative performance of OPC SCC and SCCs containing SCMs utilised as partial cement replacements.

Research Question 3: How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

This research question was addressed in this PhD through the following;

a) Probabilistic modelling was used, together with results of natural chloride migration laboratory tests, to compare the performance of different SCC types and the performance of SCCs and VCs, over timeframes applicable to service life
b) The probabilistic framework was utilised to investigate the effect of calibration of experimental test results on the relative performance using data from the real marine environment
c) Ferrycarrig Bridge was used as a case study to illustrate the implementation of the probabilistic framework and calibration methods by predicting time to initiation of corrosion in each of the marine structure's SCC crosshead beams

1.3 THESIS LAYOUT

The Thesis is organised into eight chapters and two appendices. Chapter 1 presents an introduction to the problem of chloride induced reinforced corrosion and an overview of some of the important shortcomings in existing knowledge in this area. The key research questions are also presented together with the manner in which the research questions were addressed in the PhD. Finally in Chapter 1, the thesis outline is presented.

Chapter 2 provides essential background information relating to Ferrycarrig Bridge and the accelerated testing methods used in the literature to investigate the resistance of concretes to chloride ingress. The Ferrycarrig Bridge discussion provides context for the research carried out herein, while the discussion of accelerated testing gives a general understanding of this testing type, which forms the main investigative tool used in the PhD research.

Chapter 3 presents a detailed and focused review of the existing literature in the areas related to the three key research areas presented in the previous Section 1.1. The shortcomings in the existing literature are presented and the way in which these shortcoming helped form the research questions themselves is discussed. This highly focused approach adopted for the literature review ensures detailed discussion is only presented in the areas where the PhD research constitutes a contribution to existing knowledge.

Chapter 4 of the thesis presents the first phase of experimental testing. The primary aim of this work was to assess part a) of Research Question 1 through examination of the chloride ingress resistance of five SCCs. This was carried out utilising two salt fog chambers to examine 27 concrete samples comprising of both 300 x 300 x 120 mm slab specimens and 100mm cube

specimens. The methodology and details of the 1 year Phase 1 testing programme are presented in the chapter. Subsequently, the test results are discussed in detail, with the findings from each chamber presented separately, before being compared to assess the effects of testing on different scales. Finally, in Chapter 4, the results are compared to the results in the literature and the areas requiring further investigation in a second phase of testing are presented.

Chapter 5 of this thesis presents a second, more specific and comprehensive phase of experimental testing. The work in this chapter is aimed at addressing both Research Question 1 and 2. Again this is done through experimental testing utilising a salt fog chamber in addition to absorption tests, drying test and workability tests. This testing allows the chloride resistance of SCCs to be compared to VCs and facilitates the examination of the effect of exposure condition on the relative performance of different SCCs. All results are compared to results published in the existing literature where possible.

Chapter 6 presents the probabilistic model, which is used in Chapter 7 to assess different concrete types over timescales appreciable to service life through utilisation of the Phase 1 and Phase 2 results as model input parameters. The first section of Chapter 6 establishes the theoretical basis for the probabilistic model through consideration of a number of theoretical models from the literature. Subsequently, the details of the selected model are discussed, and an issue is highlighted with the use of the proposed model in a probabilistic framework. The model is revised using a theoretically grounded adaption to resolve the issue. The model parameters which are adopted from the literature are then discussed, and a probabilistic sensitivity analysis is carried out to quantify the influence of each parameter on the model output.

In Chapter 7 Research Question 3 is addressed through presentation and detailed discussion of the probabilistic model output arising from incorporating the Phase 1 and Phase 2 test results into the model developed in Chapter 6. Firstly the probabilistic modelling results of the Phase 2 SCCs are presented. This analysis assesses the SCC options over service life timeframes and the nature of the probabilistic model output is discussed. The model parameters which contribute most to overall model uncertainty are also examined at this point using a dorm sensitivity analysis. Subsequently, the calibration of the accelerated test results, using the Ferrycarrig Bridge 2007 data, is presented in the probabilistic framework. The predicted long term performance of SCC and VCs are then compared to assess if the patterns identified in 1 year experimental testing in Chapter 5, still hold firm for service life durations. A similar type of investigation is also carried out to further investigate the effect of exposure conditions over longer exposure durations. Finally in Chapter 7,

the probabilistic model and calibrated results are used in a form of case study to make predictions as to the time to initiation of corrosion for the Ferrycarrig Bridge crosshead beams.

Chapter 8 concludes the thesis by outlining how the work presented in this thesis has addressed the three core research questions presented in this chapter. In addition recommendations for future work in this research area are presented.

2. FERRYCARRIG BRIDGE & ACCELERATED EXPERIMENTAL TEST METHODS

2.1 INTRODUCTION

Chloride induced reinforcement corrosion is recognised as global durability problem (Stewart et al. 2012), with the annual cost of corrosion worldwide estimated to exceed $1.8 trillion (Schmitt et al. 2009). Consequently, there is a large volume of published literature relating to chloride induced reinforcement corrosion. It was thus necessary to adopt a highly focused approach to the literature review in Chapter 3. As discussed in Chapter 1, this approach involves presenting a detailed discussion of the existing literature relevant to the three core PhD research questions only. However, there is also a need to provide some background information and set the context for the research in this PhD. This Chapter presents two areas which, rather than falling directly under one of the core research questions, instead providing necessary background information and set the context for the PhD research. The first of these areas is Ferrycarrig Bridge. As will be discussed in Section 2.2, the 2007 repair of this bridge saw the use of five different SCC durability options on the bridge's seven crosshead beams. These different SCC options provide the basis for the SCCs examined in this PhD. The other area discussed in this Chapter is accelerated chloride ingress testing. This section provides a discussion on the most popular forms of accelerated laboratory based testing utilised in the literature to assess the chloride resistance of different concretes. Laboratory based testing is the primary means of performance assessment utilised in this thesis and as such, an understanding of the different testing methods is important to appreciate the value of the tests carried out herein. Section 2.3 provides background on the different testing options available and discusses of the advantages and disadvantages of each test.

2.2 FERRYCARRIG BRIDGE

Ferrycarrig Bridge was constructed in 1980 and forms part of the N11 Dublin – Wexford national primary route. The bridge is located in the River Slaney tidal estuary on the South-East coast of Ireland, as can be seen from Figure 2.1. The 125.6m long structure consists of 8 equal spans of precast, prestressed beams with a reinforced in-situ concrete infill deck. The bridge beams are supported at the seven intermediate support locations by crosshead beams. These crosshead beams are supported by two separate walls which encase steel tubular piles which are driven to rock. The bridge is continuous over all intermediate supports, with the exception of the middle support, where an expansion joint was provided in the deck. A photograph of the bridge is shown below in Figure 2.2 with crosshead 1, 4 and 7 labelled.

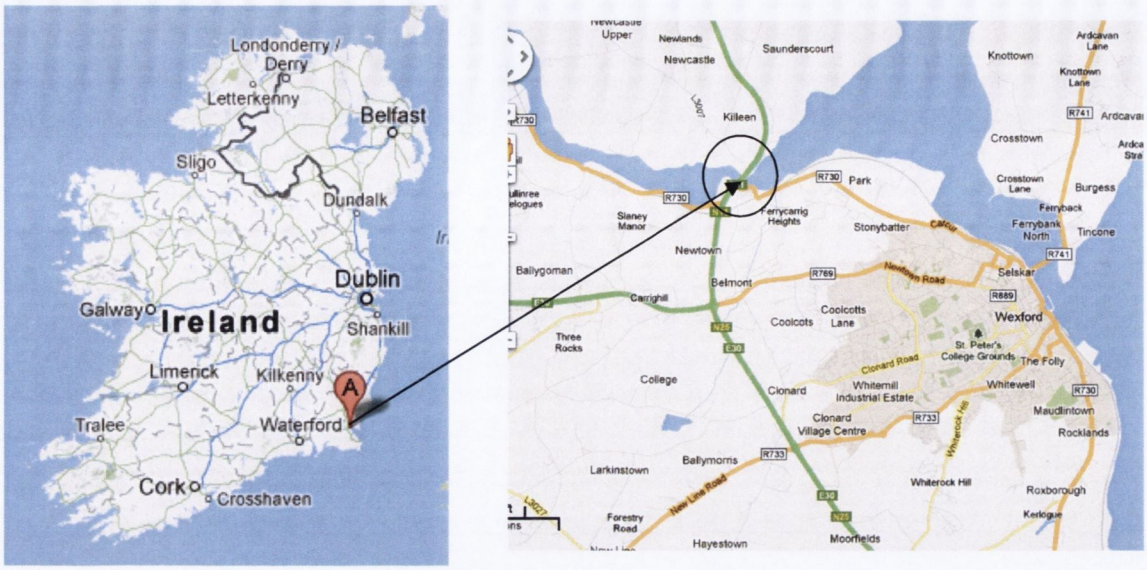


Figure 2.1 Location of Ferrycarrig bridge (Google Maps 2013)

Figure 2.2 Ferrycarrig Bridge with crossheads 1,4 and 7 labelled

In 2002 extensive cracking was identified on the Ferrycarrig Bridge crosshead beams and South abutment. In 2004 a special inspection and structural assessment of Ferrycarrig Bridge was commissioned by the Irish National Roads Authority (NRA) to determine the cause and extent of the cracking, and to investigate the severity of the deterioration. It was found that the observed cracking in the pier crosshead beams was due to a lack of reinforcement to resist the serviceability limit state stresses (i.e. shrinkage, thermal, creep etc.), and the insufficient provision of reinforcement to resist torsional moments. Based on these findings a decision was made to repair and strengthen the structure (National Roads Authority 2005). The repair works were carried out over seven months from July 2007 to January 2008.

In the context of the chloride induced corrosion it is noted that Ferrycarrig Bridge is located in the River Slaney tidal estuary, where there are high concentrations of both wind borne and river borne chlorides (National Roads Authority 2005). Consequently, information relating to chloride induced corrosion was collected during the 2004 special bridge inspection, in the form of a chloride content survey. The survey considered chloride content at various locations on the bridge at three depths, 5 - 30mm, 30 - 55mm and 55 - 80mm. The highest levels of chlorides were observed in the crosshead beams. It was, however, found that even in the areas where the chloride concentrations were highest the risk of corrosion was not high (National Roads Authority 2005). This was confirmed following visual inspection of the reinforcement during the special inspection, which revealed either no corrosion, or very minor surface corrosion, with no significant section loss observed after 24 years exposure.

A far more detailed chloride content analysis of the Ferrycarrig Bridge crosshead beams was carried out by Kenshel as part of a PhD thesis in 2007 (Kenshel 2009). This work utilised concrete cores which were extracted before the bridge's crosshead beams underwent substantial repairs. Details of this investigation, designed to assess spatial variability of chloride ingress across a marine structure, were published by O'Connor and Kenshel (2013). The study carried out by O'Connor and Kenshel will be discussed in more detail in Chapter 7, when the results are used to calibrate the accelerated test data obtained in this thesis. It is noted at this point, however, that the findings of Kenshel (2009) indicated that the chloride levels in the crosshead beams after 27 years of exposure were moderately high with a mean surface chloride content of 0.218% by weight of concrete and a mean apparent diffusion coefficient value of 10.25mm^2/yr obtained from 42 concrete cores. Kenshel found the maximum and minimum chloride contents at the 27.5mm - 35.0mm depth interval were, 0.132% and 0.01% by weight of concrete, respectively. These findings, combined with the fact that no evidence of severe corrosion was identified in 2004 during the special inspection, would indicate that the crosshead beams consisted of durable concrete and the Ferrycarrig Bridge exposure condition could be termed as a moderately severe marine environment.

2.2.1 Repair of Ferrycarrig Bridge

As established above, the special inspection of Ferrycarrig Bridge revealed that the deterioration which necessitated extensive maintenance was not caused by chloride induced reinforcement corrosion. However, the nature of the repair works carried out in 2007 afforded the Irish National Roads Authority a unique opportunity to study the global problem of reinforcement corrosion. It was therefore decided to employ one of five different Self-Compacting Concrete (SCC) durability

options for each of Ferrycarrig Bridge's seven crosshead beams. This initiative was designed to allow the SCC durability options to be studied and compared at a single marine location, as part of a real bridge structure.

For each of the crosshead beams hydro-demolition was used to remove all concrete cover to a depth of approximately 50mm beyond the existing reinforcement. Additional steel was put in place to strengthen the crosshead beams before the SCC was slowly pumped from the bottom of the crosshead beam formwork. The five different SCC durability options utilised for the seven Ferrycarrig Bridge crosshead beams are detailed below.

- Crosshead 1 – CEM I - Ordinary Portland Cement: This SCC durability option was selected to act as the control for the study. A standard 50mm cover was selected in accordance with the requirements of EN 1992-1-1:2004 for a bridge beam in a marine splash zone (Centre Europeen de Normalisation CEN 2004).

- Crosshead 2 – OPC with Increased Cover: For this crosshead the standard 50mm cover was increased to 70mm. This is seen as an enhanced durability option as increasing the cover increases the distance that externally sourced chloride ions need to migrate to reach the reinforcement.

- Crosshead 3 – OPC with a Silane Surface Treatment: Two coats of monomeric alkyl (isobutyl) trialkoxy-silane, commonly known as silane, were applied to all the surfaces of this OPC crosshead beam as an enhanced durability measure. This enhanced durability option is classified as a hydrophobic surface treatment which aims to reduce the capillary absorption of water and water borne contaminants. This is achieved by reducing the contact angle of the concrete through the reaction of the silane silicon atoms with the silicates in the concrete pore wall surface (Bertolini et al. 2004).

- Crosshead 4 – OPC + 60% Ground Granulated Blast-Furnace Slag (GGBS): For this crosshead the binder component of the mix comprised of 60% GGBS and 40% CEM I OPC. The use of GGBS as a partial cement replacement has been found by a large number of researchers to increase the marine durability of reinforced concrete (Loser et al. 2010; McNally and Sheils 2012; McPolin et al. 2005; Melchers and Li 2009; Pack et al. 2010; Thomas et al. 2008). This slag material is a waste product formed in the manufacture of pig iron, with about 300kg of slag produced for each 1,000 kg of pig iron. It must be noted that crosshead 4 is located at the expansion joint in the structure and is thus likely to be exposed

to notably different exposure conditions than the other crosshead beams. It was found by Kenshel (2009) that the crosshead beam under the expansion joint was subject to a less severe exposure condition due to the effects of chloride washout resulting from the expansion joint leakage.

- Crosshead 5 – OPC + Admixed Corrosion Inhibitor: The corrosion inhibitor used on Ferrycarrig Bridge was Ferrogard 901, produced by the Sika Group Ltd. This product comprises of a combination of organic and inorganic corrosion inhibitors and is added to the fresh concrete mix. The product is designed to reduce both the anodic and cathodic reactions of the electrochemical corrosion process. In addition the product is designed to form a film on the steel surface which delays the onset of corrosion (Sika Group Ltd. 2006).

- Crosshead 6 – 60% GGBS + 40% OPC (Identical to Crosshead 4): Crosshead 6 allows performance assessment at, and away from the expansion joint, as well as facilitating a fair comparison between the GGBS SCC option and the other durability options.

- Crosshead 7 – OPC (Identical to Crosshead 1). This provides an opportunity to study the spatial variability of chloride ingress across the structure.

During the repair of the crosshead beams six of the seven crossheads were instrumented to facilitate remote monitoring of the relative efficiencies of the different SCC durability options. The instrumentation scheme design developed by C-Probe Ltd. called for the installation of three different types of probes, each of which is listed below with a brief description of their functionality:

- Corrosion potential probes: Provide a reference potential for the assessment of the risk of corrosion of steel reinforcement in concrete structures.

- Chloride-ion penetration depth probe: Monitor the penetration of chloride ions at two depths in the concrete cover, the first sensor point is 20mm from the depth of reinforcement, the second sensor is located at the level of the reinforcement. This is a prototype probe developed by C-Probe Ltd.

- Corrosion rate probes: Allow the assessment of instantaneous corrosion rates through the utilisation of the linear polarisation resistance measurement, a technique which serves as a suitable non-destructive method for determination of corrosion rate (Pradhan and Bhattacharjee 2009).

In total ten probes were embedded in each of the instrumented crosshead beams. Temperature and humidity sensors were also installed at three locations on the bridge. Due to the slow nature of chloride ingress, no meaningful results are available from the probes to date. Consequently, the presentation and discussion of Ferrycarrig Bridge probe system results is outside the scope of this PhD.

The SCCs used in the repair of Ferrycarrig Bridge were designed for use in aggressive marine environments and are thus resistant to the ingress of chlorides. Therefore, it is likely to be 20 to 30 years before information on the relative performance of the various SCC options will be obtained from Ferrycarrig Bridge. As mentioned in the thesis Introduction, and discussed in detail in Chapter 3, it is important to obtain information relating to the durability of SCC options in the short to medium term, while these concrete options are being widely used in the construction industry. This need constitutes a key part of the overall context of this PhD research. The marine performance of a number of the Ferrycarrig Bridge SCC durability options are investigated in the short to medium term in this PhD thesis through two phases of accelerated laboratory based testing. The next section of this chapter provides a discussion on the most popular accelerated laboratory testing methods used in the literature, and presents the reason for using salt fog chamber testing in this PhD research to assess the durability characteristics of the Ferrycarrig Bridge SCC options. It is also noted at this point that in addition to the Ferrycarrig Bridge SCCs an SCC using Pulverised Fuel Ash (PFA) as a partial cement replacement was investigated in the accelerated laboratory testing herein due to the documented advantages of OPC + PFA concretes in marine environments (McPolin et al. 2005; Mohammed et al. 2002; Stanish and Thomas 2003; Thomas and Bamforth 1999). This will be discussed in more detail in Chapter 4.

2.3 ACCELERATED EXPERIMENTAL TESTING

1.1.1 Introduction

The primary means of gathering information relating to marine durability in this thesis is accelerated laboratory testing. As stated by Castro (1997) the very slow nature of the chloride ingress process in concrete means it is almost impossible to evaluate all types of concrete

combinations in the real marine environment. Consequently, much of the information relating to marine durability is obtained by some form of accelerated lab based testing (Li 2001). This section will provide a discussion on the various forms of accelerated laboratory testing methods which are employed in the literature to examine the ingress of chlorides into concrete in the initiation phase of reinforcement corrosion. According to the widely accepted two phase model proposed by Tutti (1979), this initiation phase commences upon exposure of the concrete to a chloride laden environment, and ends at the point when the critical chloride content is reached i.e. when reinforcement corrosion commences. The second phase in this model is the corrosion propagation phase. As previously mentioned, the sole focus of the work in this thesis will be on the initiation phase of chloride induced reinforcement corrosion.

Before a discussion can be presented on the testing methods employed to examine the ingress of chlorides into concrete it is important to first understand the actual physical processes by which chlorides move into concrete structures in the marine environment. Bertolini (2004) stated that the movement of chlorides from external sources into the concrete takes place via three main transport mechanisms as follows:

1. Diffusion
2. Absorption
3. Permeation

The movement of chlorides by diffusion is considered to be the most dominant chloride transport mechanism, especially in the saturated inner layers of the concrete (Nanukuttan et al. 2009; Thomas and Bamforth 1999). This transport process involves chloride movement via ionic diffusion through pore water under a concentration gradient. This process is far more effective in saturated pores than in partially saturated pores (Bertolini et al. 2004). There are two classifications of diffusion; steady state diffusion and non-steady state diffusion. Steady state diffusion pertains to Fick's first law of diffusion and is relevant when chlorides are passing through concrete at a constant flux. This will be discussed further in the context of the rapid chloride migration test in Section 2.3.2. As pointed out by Bertolini (2004), however, this steady state condition rarely arises in concrete structures. Consequently, non-steady state diffusion is generally encountered in concrete structures. Non-steady state diffusion is generally represented by Ficks's second law, presented as Equation 2.1.

$$C(x,t) = C_s\left[1 - erf\left(\frac{x}{2\sqrt{t.D}}\right)\right] \qquad \textbf{Equation 2.1}$$

where C(x,t) is the percentage chloride content at depth x and time t, C_s is the surface chloride content, erf is the error function, x is the depth below the concrete surface being considered, D is the diffusion coefficient and t is time. As will be discussed in Section 2.3.1 Fick's second law is often used in the literature to mathematically represent chloride profiles obtained from experimental testing under natural chloride migration conditions (McPolin et al. 2005; Nokken et al. 2006). The diffusion coefficient obtained from this process is termed the apparent diffusion coefficient, D_{app}.

The transport of chlorides by absorption takes place when water borne chlorides ingress into void spaces in the concrete due to capillary suction. As stated by Richardson (2002) and Neville (2011), this transport mechanism may be significant in concrete which is subject to cyclic wetting and drying. The absorption action depends on surface tension, viscosity and density of the liquid, the radius of the pore and the angle of contact between the liquid and the pore walls (Bertolini et al. 2004). It is noted that, by definition, the D_{app} values obtained from the tidal, splash and spray zones of real structures incorporate the chloride transport due to both absorption and diffusion (Saassouh and Lounis 2012).

The transport of chlorides due to permeation occurs due to pressure gradients. This chloride transport mechanism is limited to situations where concrete elements are subject to a hydrostatic head i.e. groundwater flow (Richardson 2002). This situation is uncommon in the marine environment and is not considered in this thesis.

Having established the physical means by which chlorides ingress into concrete, the testing methods used to examine the resistance of concretes to chloride ingress may now be discussed. The test types will be discussed under two separate headings. Firstly, methods which utilise natural chloride migration to examine concrete will be presented. These tests, although accelerated with regard to the levels of chloride exposure, do not accelerate the movement of chlorides by other means such as electrical current. This results in the natural migration of chlorides into the concrete, in the same general manner as in the real marine environment. The second group of tests which will be discussed employ an electric current so that the movement of chlorides into concrete is highly accelerated to a time frame of a number of hours to a number of days. A brief description of each test discussed will be provided, after which, the advantages and disadvantages of the test in question will be discussed. The primary reference for the information detailed in this section is a

report produced by Stanish et al. (1997). This report is a literature review of the most popular methods for examining the chloride penetration resistance of concrete. Other more recent sources of information relating to the suitability of each test method are also referred to in the discussion.

2.3.1 Natural Chloride Migration/Diffusion Tests

Bulk Diffusion Test

The bulk diffusion test is used to obtain values for the apparent diffusion coefficient, D_{app} and the surface chloride content, C_s, from concrete samples which are fully submerged for the duration of the exposure period. The test involves submerging pre-saturated concrete specimens in a 16.5% chloride solution for a period of generally greater than 3 months (Nordtest Method 1995). The concrete specimen being tested is sealed on all bar one face during the exposure period. Once the exposure period is complete chloride profiles are developed from dust samples and values for C_s and D_{app} are obtained (Stanish et al. 1997). The bulk diffusion test is standardised in NT Build 443 (Nordtest Method 1995) and ASTM C 1556 (ASTM 2011).

The main advantage of the bulk diffusion test is that it examines chloride movement under natural conditions without the influence of an electric current. This, however, also results in the disadvantage of long test durations. Exposure periods greater than three months are generally required for concrete types deemed appropriate for use in marine environments in accordance with international standards (water/cement ratio < 0.5, cement content > 300kg/m^3) (Centre Europeen de Normalisation CEN 2000). Another disadvantage of the test is the fact it replicates the fully submerged condition and thus neglects the influence of absorption on the chloride migration in concrete, which can have a significant influence on chloride movement in concrete subject to wetting and drying (Hong and Hooton 1999; Neville 2011).

Salt Ponding Test

The salt ponding test is standardised in AASHTO T259 (American Association of State Highway and Transportation Officials 1980). According to this standard three identical concrete slab specimens, which are sealed on the side faces only, are ponded to a depth of 13mm on the top face with a 3% salt solution for a period of 90 days. Prior to testing the slabs are moist cured for 14 days and then stored in a drying room at 50% Relative Humidity (RH) for 28 days. The procedure dictates that the chloride content is then examined at two to three progressive depths (McGrath and Hooton 1999).

Although the test examines chloride resistance under natural chloride migration conditions a number of problems with the test procedure have been pointed out by researchers (McGrath and Hooton 1999; Stanish et al. 1997). Firstly, the sample is dried for 28 days at 50% RH prior to exposure. Consequently, initial absorption may play a key role in the test, which is designed to examine pure diffusion through the ponding of concrete samples for 90 days without drying cycles. Secondly, the fact that the bottom face of the test specimen is not sealed results in chlorides being drawn into the sample due to a process known as wicking caused by a moisture differential between the top and bottom face (Stanish et al. 1997). In addition, as pointed out by McGrath and Hooton (1999), obtaining chloride contents at only two to three depths can give a false impression of the depth of chloride penetration, particularly in high quality concretes. Finally, it is noted that the exposed face is subject to a submerged condition for the duration of the testing period, again neglecting the role of absorption which may be encountered in the tidal, splash and tidal zones commonly identified as most severe exposure zones for marine structures (Ye et al. 2012).

It should also be noted that some researchers have used variations of this salt ponding test to examine chloride resistance of concrete specimens subject to wetting and drying cycles. The specimens used in these tests range from slab specimens, as per the specimen dimensions in AASHTO T259 (McPolin et al. 2005), to loaded concrete beams (Ababneh et al. 2012).

Salt Spray/Fog Test

The experimental testing conducted in this thesis utilises salt fog chamber testing. Consequently, the salt fog chamber operation and testing procedure are discussed in detail in Chapter 4. A brief description of this form of testing will, however, be provided at this point to facilitate comparison with the other test methods in this chapter.

Salt spray/fog chamber testing mimics the splash and spray zones of concrete structures by subjecting concrete specimens to alternating salt spray/fog cycles in a purpose built environmental chamber. This allows specimens to be exposed to wetting and drying cycles in a uniform and controlled manner when the appropriate equipment is utilised. Such environmental chambers can be split into two classifications; salt fog chambers and salt spray chambers. The naming convention is based upon the means by which the samples are exposed to the salt solution. The salt spray chamber test utilises water pressure to spray drops of chloride solution onto the surface of the test samples during the wet periods and is referred to by some researchers as a marine shower system (Mangat and Gurusamy 1987). Testing carried out in these spray chambers can be affected by spatial variability due to uneven dispersion of salt solution droplets within the chamber. The

development of the salt fog chambers, which appear in concrete durability literature 12 years after the salt spray chambers (Asrar et al. 1999; Mangat and Gurusamy 1987), sought to address this spatial variability issue through atomisation of the saline solution using compressed air (Asrar et al. 1999). This solution results in a saline mist or fog being emitted during wetting cycles. This fog fills the entire chamber resulting in an even dispersion of saline solution throughout the chamber. As noted above salt fog chambers are utilised in the experimental testing described in this thesis. The high cost of these chambers means studies in the literature which examine chloride ingress of different concretes using salt fog chambers are limited to (Asrar et al. 1999; Konin et al. 1998; Wang et al. 2011). Studies carrying out such investigations using salt spray chambers are slightly more prevalent, although still rare in the literature (Dhir et al. 1994; Leung et al. 2008; Li 2000; Li 2001; Mangat and Gurusamy 1987; Mangat and Gurusamy 1987; Mangat and Molloy 1991; Mangat and Molloy 1995; Zhang and Jin 2011).

There is no standardised procedure for testing concrete in salt spray/fog tests. EN 9227: 2006 does provide a standard for testing the corrosive resistance of metals and metal coatings using salt spray tests (Centre Europeen de Normalisation CEN 2006). The test apparatus' utilised in this thesis were designed and constructed in accordance with this standard EN9227 (Weiss Technik). However, the actual test procedures outlined in the standard are not suitable for examining the chloride resistance of concrete specimens i.e. salt solution sprays for entirety of EN 9227:2006 test for a prescribed test period as low as 2 hours. In brief, the procedure used herein involved exposure of concrete specimens, sealed on all bar one moulded face, to periodic wetting and drying cycles. The wetting cycles entailed exposure of the samples to a 5% NaCl solution fog which fills the chamber during testing. As will be discussed in detail in Chapters 4 and 5, different wetting and drying cycle orientations were utilised in the two phases of testing. After the test period was complete the dust collection and analysis system prescribed in NT Build 443 for analysis of the bulk diffusion test samples was adhered to (Nordtest Method 1995).

As with the bulk diffusion tests and the salt ponding test, one of the primary advantages of the salt spray/fog test is that it examines the natural movement of chlorides in concrete. As mentioned above, however, such methods necessitate substantial testing durations. In addition the sample analysis in accordance with NT Build 443 is time consuming and labour intensive. The salt fog chamber test procedure utilised herein differs from the bulk diffusion test and AASHTO salt ponding test in that it incorporates wetting and drying cycles into the exposure period. This means that the absorption component of chloride ion ingress is not neglected in the testing. As will be discussed in detail in Chapter 3, this is an advantage in the light of a) the recognised importance of the wetting and drying in the splash zone (Neville 2011), b) the very slow rate of corrosion in fully

submerged concrete elements when compared to elements in the splash and tidal zone (Hussain 2011), and c) the fact that the majority of experimental testing in the published literature was carried out in fully submerged conditions.

The provision of wetting and drying cycles does, however, have three notable disadvantages. Firstly, it is very difficult to correlate the level of drying in the salt spray/fog chamber with the level of drying in the real marine environment. Thus the role of absorption in the chloride migration process may be greater than, or less than, the real marine environment for a given test set-up. It is noted, however, that a range of exposure conditions are experienced across a marine structure as pointed out by Zhang and Jin (2011). Secondly, it is difficult to standardise the test with different levels of drying occurring in different laboratories due to variation in the apparatus used. These differing levels of drying will lead to variation in results from laboratory to laboratory when the same concrete is tested as will be illustrated in Chapter 4 and 5. The final disadvantage relates to spatial variability within the test space. In salt spray chambers specimens are sprayed with salt using pressurised spray nozzles. Logic dictates that this procedure is subject to spatial variability due to uneven distribution of chloride solution droplets throughout the chamber. However, the more advanced salt fog chamber system (used in this thesis) addresses this issue through the delivery of the chloride solution in a fine mist of fog. This fog system is described in detail in Chapter 4.

Cyclical Immersion Tests

Researchers have also used a range of test set-ups which involve the cyclical immersion of samples in chloride solution to mimic the splash and tidal zones in the marine environment. Two such studies, which are discussed in detail in the next chapter, were carried out by Audenaert et al. (2005), and Ben Fraj et al. (2012). As will be discussed Audenaert et al. used a test set-up where concrete disc were rotated, with only a portion of the disc specimen immersed in a salt solution at a given time. The authors did not seal the sides of the samples meaning the small scale of the specimens would have had considerable effect on results due to multidirectional flow. Ben Fraj et al. exposed samples to cyclic wetting and drying by periodically pumping a salt solution in and out of a climactic chamber. A similar approach was adopted by Ye et al. (2012), who pumped salt solution into plastic containers where small beam specimens were located. Ye et al. also failed to seal the sides of the samples meaning a considerable size effect would have been encountered. It is noted that these is no established international standard for these forms of tests. Therefore the applicability and accuracy of the testing depends on the procedure adopted in each case. The advantages and disadvantages of the procedures of testing programs which are relevant to the

research carried out herein will be discussed in Chapter 3, on a case by case basis. At this point, however, it is noted that such tests have the advantages of incorporating absorption into the exposure period while examining the chloride resistance of concrete under natural chloride migration conditions.

2.3.2 Accelerated Testing Utilising Electrical Migration Techniques

Steady State Rapid Chloride Migration Test (RCMT)

This test developed by Andrade (1993) utilises an electrical current to accelerate the movement of chlorides through a disc shaped concrete specimen. The test examines the migration coefficient in the steady-state of chloride migration. This means that the migration coefficient is obtained when the rate of movement of chlorides is constant, after the point when the chlorides from the chloride reservoir have passed through the concrete. The steady state diffusion period is represented by the straight portion of the curve in Figure 2.3, which shows the chloride concentration downstream of the concrete sample.

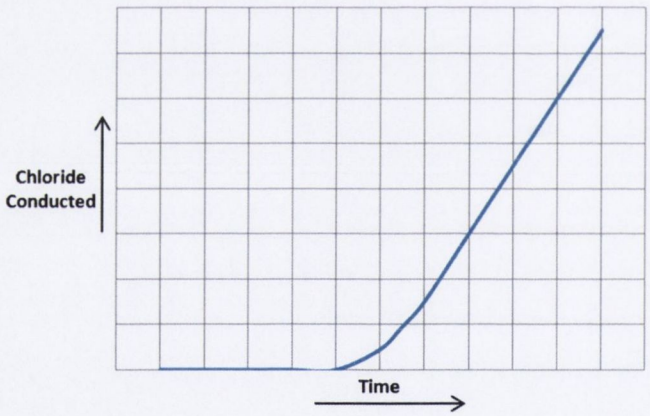


Figure 2.3 A typical Migration plot of downstream chloride concentration versus time

The experimental test set-up is shown below in a diagram taken from Stanish et al. (1997). As can be seen from the figure, the concrete disc specimen is placed in the middle of a two chamber cell. At the start of the test the cathode chamber contains chloride ions but the anode chamber is chloride free. A voltage is then applied to drive the chloride ions through the concrete. The concentration of chlorides in the downstream chamber is monitored throughout the test. The Nernst-Planck equation is then used to obtain chloride migration coefficients (Andrade 1993).

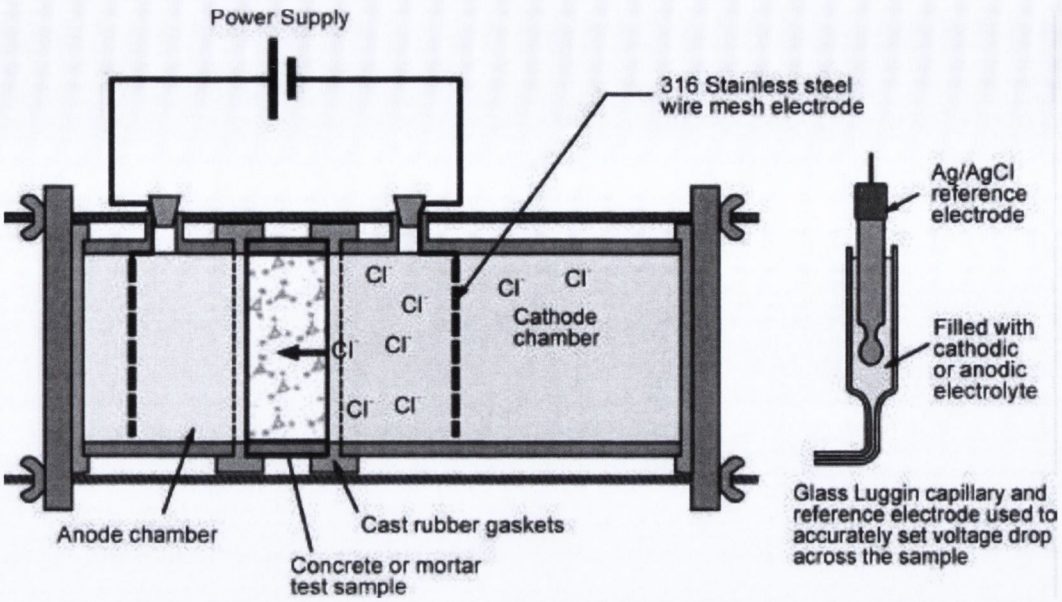


Figure 2.4 Steady state chloride migration test set-up (Stanish et al. 1997)

The primary advantage associated with this test is the fact that results can be obtained over a matter of days, as compared to a number of months for the natural chloride migration testing. However, as pointed out by Stanish et al. (1997), there are a number of drawbacks associated with this form of testing. Firstly, if the test is carried out using a high voltage the results can be affected by rises in temperature of the concrete specimen due to the application of the driving voltage. These rises in temperature due to voltage are higher for poorer quality concretes making these concrete appear worse than they actually are. Secondly, the inclusion of conductive materials in the concrete will short circuit the cell with the current being carried by the conductor. Also, if the test specimen is not thick enough, relative to the aggregate size, a weak transition zone may exist around the aggregate which extends most of the way through the sample. The test examines concrete in a fully submerged state neglecting the influence of absorption. As will be discussed in the context of the work of Hussain (2011) later in this thesis, corrosion does not in general pose a significant threat to the durability of submerged concrete due to a lack of the oxygen required to cause corrosion at the reinforcement depth. Finally, logic dictates that there is considerable uncertainty associated with the acceleration of a process that takes a number of years, to a period of a number of days. For example, the formation of Friedal's salts caused by chloride binding during the natural migration of chlorides through concrete reduces the diffusion coefficient of concrete (Luping and Gulikers 2007). However, acceleration of the chloride migration process to a number of days means these Friedel's Salts do not form and the resulting increase in chloride resistance is not accounted for (Spiesz et al. 2012).

Non-Steady State RCMT

The non-steady state RCMT developed by Tang and Nilsson (1993), is similar to the chloride migration test but instead examines chloride migration in the non-steady state. The test has been standardised in NT Build 492 and has become increasingly popular in recent years as a result of the use of the output of the test in the Duracrete service life prediction model (DuraCrete 1998; Spiesz et al. 2012). The test set-up is shown below in Figure 2.5. As can be seen from the figure a two cell system is used, as with the steady state RCMT. The primary difference with this test is that the chloride concentration in the downstream cell is not monitored. Instead the sample is removed from the test apparatus after a given test period and the specimen is axially split. A colorimetric indicator, in the form of silver nitrate solution, is then sprayed on the split face to establish the depth of chloride penetration. The chloride migration coefficient is then calculated based the Nernst-Planck equation (Nordtest Method 1999).

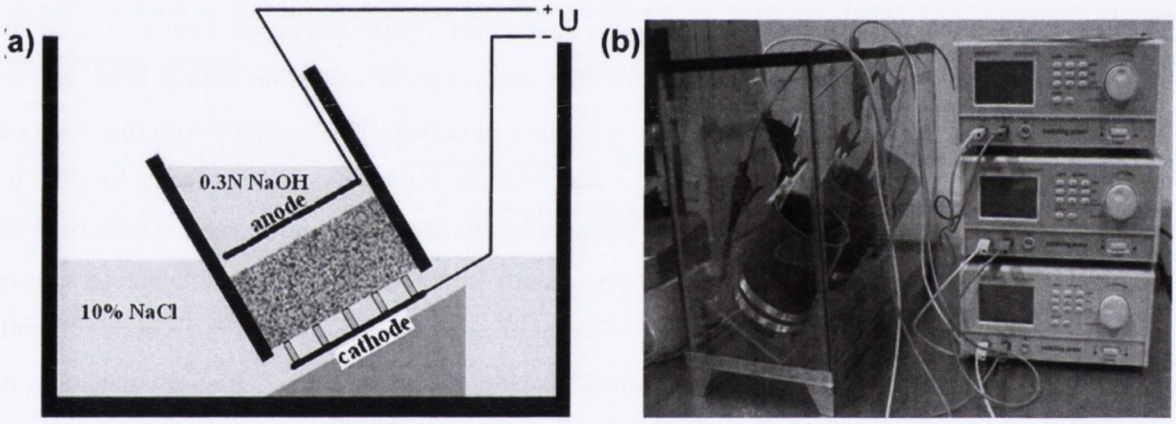


Figure 2.5 Non-steady state RCMT set-up (Spiesz et al. 2012)

The primary advantage of the RCMT is that the test duration is short, with results obtained in a matter of days. The test is, however, subject to the most of the disadvantages associated with the steady state RCMT. In addition a number of other issues associated with the calculated chloride migration coefficient, using the Nernst-Plank equation, were pointed out by Spiesz et al. (2012). These issues relate to assumptions regarding a) the shape of the free chloride profiles at the end of the test, b) linear chloride binding isotherm and c) equilibrium between free and bound chlorides. Spiesz et al. (2012) proposed theoretical solutions to these problems, but there remain a high level of uncertainty associated with the acceleration of the chloride migration process and the method of determination of the chloride migration coefficient.

Rapid Chloride Permeability Test (RCPT)

The final test which will be discussed is the rapid chloride permeability test. This test is formalised in standards ASTM C 1202 (2012) and AASHTO T227 (1990) and is perhaps the most popular test method in the existing literature (Gesoğlu and Güneyisi 2011; Gesoglu et al. 2009; Hassan et al. 2009; Kasemchaisiri and Tangtermsiriku 2008; Kou and Poon 2009; Nayak et al. 2011; Nehdi et al. 2004; Panesar and Shindman 2012; Pathak and Siddique 2012; Sahmaran et al. 2009; Uysal et al. 2012). As pointed out by Stanish et al. (1997) the naming of the test is inaccurate, as it is not chloride permeability which is being measured but rather ionic movement. The test set-up can be seen below in Figure 2.6. The procedure involves the application of a 60V DC voltage for 6 hours with the total charge passed over the test period recorded. This total charge value is used to rate the chloride ion permeability as high, moderate, low, very low or negligible.

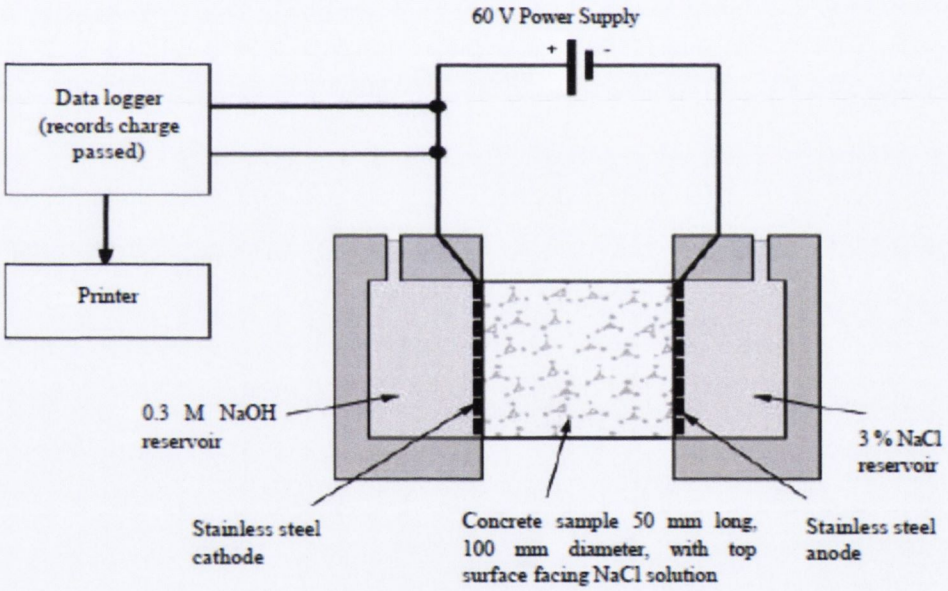


Figure 2.6 RCPT set-up (Stanish et al. 1997)

Although the test allows results to be obtained after 6 hours there are many criticisms of the RCPT. Stanish et al. (1997) pointed out that a) the test measures not just the passage of chloride ions, but rather the passage of all ions in the pore solution b) the measurements are made before steady state migration is achieved c) the high voltage applied leads to an increase in sample temperature, especially for low quality concretes and d) the test relies on the conductivity of the concrete being in some way related to the chloride ion permeability. In addition it was found by Shi (1998) that the replacement of OPC with SCM, can reduce the electrical conductivity of concrete by more than 90% due to the change in the chemical composition of the pore solution. This change is unrelated

to the transport of chloride ions in concrete. Thus Shi et al. found that it is incorrect to use the charge passed as specified in the RCPT to evaluate the chloride permeability of concretes containing SCMs'. Scanlon and Sherman (1995) also alluded to problems with the RCPT stating that the RCPT is not suitable for use with concretes containing silica fume, fly ash, or high-range water-reducing admixtures.

Accelerated Testing Discussion

Overall it is clear that there are advantages and disadvantages associated with each type of chloride ingress test. Tests which are accelerated using an electrical current are removed from reality, but, allow results to be obtained over short test periods, and without labour intensive sample analysis. Test which do not utilise an electric current to accelerate the process are closely aligned with the exposure encountered by real marine structures, however, exposure durations and sample analysis hours can be substantial, especially for concrete mixes which comply with the marine environment requirements specified in International Standards (Centre Europeen de Normalisation CEN 2000).

Castellote and Andrande (Castellote et al. 2006) published details of an investigation into the most appropriate test method for testing and modelling chloride penetration in concrete. 27 different laboratories contributed to the RILEM Technical Committee led programme, which examined 13 different test methods for 4 different concretes. All chloride ingress testing was examined in fully submerged concrete and thus salt spray/fog chambers were not assessed. However, a number of other natural chloride migration tests were utilised which included the salt ponding test and the bulk diffusion test described above. The tests were assessed under four headings as follows:

1. Trueness (variation of mean and skew)
2. Precision (repeatability and reproducibility)
3. Relevance (relevance of diffusion coefficients and method of acceleration to real environment)
4. Convenience (need for grinding, need for chloride ion analysis, difficulty in handling)

It is noted that the salt ponding test and the bulk diffusion test were used as the reference tests for the trueness assessment reiterating the connection between natural chloride migration experiments and real marine exposure. These tests were scored highest in terms of trueness. The ponding test was also scored highest in terms of relevance together with a resistivity test developed by Andrade and a chloride migration test developed by Castellote. The RCPT was ranked highest in terms of precision and convenience. Not surprisingly the bulk diffusion test and the salt ponding test were

ranked last for convenience. Overall the best ranked test method was a resistivity based test developed by Andrade et al. (Andrade et al. 2001). However, when considering this result in the context of the work carried out in this thesis the recommendations of another RILEM Technical Committee must be considered. This RILEM technical committee (No. 205) was responsible for investigating the marine durability of SCC (Rilem Technical Committee et al. 2008). With regard to the testing of SCC the committee stated that even though the microstructure of SCC is different from VC there is so far no indication that the test methods developed for evaluating chloride penetration in conventional concrete are not applicable to SCC. But, it was also stated by the committee that care should be taken in interpreting accelerated test results, especially when using indirect methods based on the principle of electrical resistivity, such as ASTM C 1202 or AASHTO T277. Given this warning it would seem most appropriate to return to first principles when examining SCC, and utilise a test method which examines SCC under exposure conditions which are closest to the real marine environment. This was the basis for the use of the natural chloride migration type test in this thesis. The basis for the examination of SCC in wetting and drying cycles will be discussed in Section 3.3.2 of Chapter 3.

2.4 CONCLUSIONS

The aim of this chapter was to provide necessary background information and context for the research described in this thesis. The research context has been provided by the discussion of Ferrycarrig Bridge and the durability options utilised in the repair of this structure in 2007. Further context and background information has been provided by the presentation of the most popular experimental methods used in the literature to investigate chloride resistance of different concretes. This discussion indicated that the use of a test method which employs natural chloride migration is the most appropriate for the assessment of SCCs.

3. LITERATURE REVIEW

3.1 INTRODUCTION

The previous chapter has provided background information necessary to contextualise and understand the general research area of this PhD. This chapter provides a detailed and focused discussion of the existing literature relevant to the three core research questions outlined in the thesis introduction. This discussion will illustrate how the work presented in this PhD thesis constitutes a novel and significant contribution to existing knowledge. The existing literature in the following three areas is presented;

- Self-compacting concrete
- Effect of exposure conditions on relative performance of concretes
- Probabilistic deterioration modelling

The literature in each of the three areas is discussed in detail with gaps in the existing literature highlighted. Subsequently, the means by which the work described in this thesis addresses these gaps will be discussed.

3.2 SELF-COMPACTING CONCRETE

3.2.1 Introduction

As mentioned in the introduction Self-Compacting Concrete (SCC) is a relatively new form of concrete technology having been developed in Japan in the mid 1980's. It is thought that SCC was used in Europe for the first time in the mid 1990's when it was employed in civil works for transportation networks in Sweden (EFNARC et al. 2005). EN 206-9-2010 defines SCC as concrete that is able to flow and compact under its own weight, fill formwork with reinforcement, ducts box houses etc. whilst maintaining homogeneity (Centre Europeen de Normalisation CEN 2010)

The self-compacting effect is achieved through utilising a different material composition to that used in conventionally Vibrated Concrete (VC). The nature of the material composition utilised to achieve the SCC does, however, differ widely in the available literature. A European guideline document for SCC was developed in 2005 by a project group of five European Federations concerned with the promotion of advanced materials and systems for the supply and use of concrete (EFNARC et al. 2005). This document provides advice on constituent materials, their

control, and their interaction in SCC. It is, however, stated in the guidelines that there are a number of different approaches to the design of SCC mixes. Consequently, no specific method is recommended by the guidelines and the reader is referred to a list of relevant papers for details on SCC mix design. EN 206-9:2010 also provides guidance relating to SCC, however, no guidance is provided in this standard relating to mix proportions. Instead this code specifies the fresh concrete requirements which a SCC must achieve (Centre Europeen de Normalisation CEN 2010).

In general, however, SCC mix designs have a reduced coarse aggregate content and an increased fine aggregate content with a super plasticiser also generally used to increase workability (Su et al. 2001). The powder content is higher in SCC than traditionally vibrated concrete so as to prevent the segregation of the SCC mix. This can be achieved through utilising a larger volume of OPC cement (Gesoğlu and Güneyisi 2011; Uysal et al. 2012) or a combination of OPC and mineral admixtures such as PFA, GGBS or limestone powder (Audenaert et al. 2010; Corinaldesi and Moriconi 2004; Sahmaran et al. 2009). Alternatively some researchers have employed a slightly increased powder content together with a Viscosity Modifying Admixture (VMA) to prevent mix segregation (Hwang and Khayat 2009; Sonebi and Nanukuttan 2009). It can thus be said that there are a variety of approaches adopted to achieve a SCC.

The work in this thesis is not concerned with the examination or development of various mix design approaches in terms of SCC workability. The SCC mixes, for the first phase of testing in particular, were dictated by the SCC mix designs employed in the repair of Ferrycarrig Bridge in 2007. The work described in this PhD thesis is instead concerned with the examination of the marine durability of different SCCs. Consequently, this section presents the SCC marine durability literature published to date, followed by a brief discussion on gaps in the existing literature and how these gaps relate to the research questions posed in Chapter 1. Finally in this section, the manner by which the work described in this thesis helps to reduce existing gaps in the SCC literature by addressing the key research questions is presented.

3.2.2 SCC Marine Durability Literature

A publication authored by the RILEM Technical Committee in 2008 in the Materials and Structures Journal alluded to the fact that the fundamental differences in the mix designs for SCC and VC, and the absence of vibration in SCC, could lead to differences in the pore structure, ionic compositions in the pore solution, chloride binding behaviour and durability behaviour. This paper also stated that, due to the very limited information available in the literature concerning chloride penetration in SCC, this concrete type is being used without a general and fundamental insight into

the intrinsic durability of the material itself (Rilem Technical Committee et al. 2008). Consequently, there is an urgent need for research into a number of different areas of SCC. For instance Sonebi and Nanukuttan (2009) have called for studies comparing the chloride transport properties of SCC to VC while Nehdi et al. (2004) alluded to the need for studies investigating durability consequences associated with the replacement of OPC with supplementary cementing materials in SCC.

In recent years a number of studies have been published in the literature which examine the chloride resistance of SCC (Audenaert et al. 2010; Gesoglu et al. 2009; Hwang and Khayat 2009; Kasemchaisiri and Tangtermsiriku 2008; Kou and Poon 2009; Panesar and Shindman 2012; Pathak and Siddique 2012; Sahmaran et al. 2009; Uysal and Yilmaz 2011; Uysal et al. 2012; Zhao et al. 2012). The majority of these recent publications, however, examine the resistance of SCC to chloride ingress utilising the Rapid Chloride Permeability Test (RCPT). There are, however, two problems associated with the use of this test to compare the relative performance of OPC SCC and SCCs containing supplementary cementitious materials. Firstly, as discussed in the previous chapter the test is an indirect measure of chloride ingress resistance based on concrete resistivity which only allows a concretes chloride penetrability to be ranked in one of five categories; negligible, very low, low, moderate and high (ASTM 2012). Consequently, the qualitative nature of the test it is not appropriate for the evaluation of the relative performance of durability characteristics.

In addition, as stated in Chapter 2, Shi (1998) found that it is incorrect to use the RCPT to evaluate the durability of concretes containing SCMs while Scanlon and Sherman (1995) stated that the RCPT is not suitable for use with concretes containing silica fume, fly ash, or high-range water-reducing admixtures (used in all SCCs). Given the unsuitability of the RCPT for assessing SCC, and concrete in general which contains SCM, the SCC publications which examine chloride ingress resistance solely utilising this method are noted by way of reference but not discussed in detail herein (Gesoğlu and Güneyisi 2011; Gesoglu et al. 2009; Guneyisi et al. 2011; Hossain and Lachemi 2010; Kasemchaisiri and Tangtermsiriku 2008; Kou and Poon 2009; Nayak et al. ; Nehdi et al. 2004; Panesar and Shindman 2012; Pathak and Siddique 2012; Sahmaran et al. 2009; Uysal et al. 2012).

The other forms of tests used in the literature to assess the chloride transportation properties of SCC are natural chloride migration tests, non-steady state Rapid Chloride Migration Tests (RCMT) and the steady state RCMTs. In terms of data from real structures utilising SCC, it is recognised that practical long term data on SCC is very limited (Rilem Technical Committee et al. 2008). The

author could not in fact find any information on the performance of SCC from real structures in the literature. This is likely to be due to the slow nature of chloride ion ingress in the real marine environment and the relatively recent development of SCC technology.

There is also a shortage of publications in the literature which examine the marine durability of SCC utilising natural chloride migration tests, with a small number of these publications examining the actual chloride transport characteristics C_s and D_{app}. The first of these publications which will be discussed is a paper by Loser et al. (2010) which describes the assessment of a number of SCCs using bulk diffusion tests in accordance with ASTM C 1556 (ASTM 2011). The natural chloride migration methods utilised by Loser et al. (2010), combined with the extensive nature of the experimental programme carried out, make this paper the most relevant SCC publication in the literature to the experimental work described in this thesis. The results of the experimental programme facilitate examination of the relative performance of VC and SCC which utilise the following binder types; OPC, OPC + 20% PFA and a CEM III/A (36-64% GGBS). The paper also provides non-steady state RCMT results in addition to the bulk diffusion test results for each of the SCC and VC mixes examined. This facilitates an examination into the appropriateness of the RCMT to the assessment of SCC, through comparison with a test which allows chloride migration to occur under natural conditions (i.e. without the use of an electrical current to accelerate the process).

Table 3.1 below presents the migration coefficients obtained from the RCMTs and the diffusion coefficients obtained from the bulk diffusion tests for both the SCC and the VC. The figures in the table are in mm^2/yr (to convert m^2/s multiply mm^2/yr value by $3.17x10^{-12}$). The water/binder (w/b) ratio for each mix is given in brackets in the mix title. The relative merits for the concretes are presented in the body of the table in brackets. These figures describe the performance of the concretes containing SCMs relative to the performance of the control OPC mixes. Taking the OPC + 20% PFA SCC RCMT relative merit as an example, this relative merit is calculated by dividing the OPC SCC migration coefficient by the OPC + PFA SCC migration coefficient. These relative values facilitate comparison of the concretes containing SCMs across the two types of testing utilised.

Table 3.1: Loser et al. (2010) results from bulk diffusion tests and RCMTs

Binder Material	VC (0.35) Bulk Diffusion	VC (0.35) RCMT	VC (0.45) Bulk Diffusion	VC (0.45) RCMT	SCC (0.40) Bulk Diffusion	SCC (0.40) RCMT
CEM I OPC	50.5 (1.0)	41.0 (1.0)	91.5 (1.0)	176.6 (1.0)	69.4 (1.0)	138.8 (1.0)
CEM I OPC + 20% PFA	44.2 (1.1)	56.8 (0.7)	78.8 (1.2)	170.3 (1.0)	53.6 (1.3)	94.6 (1.5)
CEM III/A (OPC + 36-64% GGBS)	28.4 (1.8)	6.3 (6.5)	34.7 (2.6)	31.5 (5.6)	18.9 (3.7)	9.4 (14.6)

From Table 3.1 it would appear as though there may be issues with utilising the RCMT to examine the relative performance of concretes containing GGBS, especially SCCs. In general it can be noted from the table that the migration coefficient obtained from the RCMT was higher than the diffusion coefficient obtained from the bulk diffusion tests for the OPC concretes and OPC + PFA concretes. This pattern is, therefore, reversed for the concretes containing GGBS. Consequently, the relative performance of the GGBS concretes is much higher for the RCMT than the bulk diffusion test. This problem is particularly evident for the SCC mix, where the relative merit value obtained for the GGBS was four times greater for the RCMT than the bulk diffusion test. This is cause for concern in the context of the warning issued in the 2008 RILEM publication which stated that care must be taken when assessing SCC with techniques designed for traditional vibrated concretes (Rilem Technical Committee et al. 2008). The RCMT was designed in 1993 for VC (Tang and Nilsson 1993). It stands to reason, however, that the study of chloride movement under natural conditions without the application of a current should be applicable to all forms of building materials as this form of testing is as close to real marine exposure as is practically possible within reasonable laboratory time constraints. It is important to be aware of the possible limitations of the RCMT in assessing SCC highlighted by the work of Loser et al. (2010), as it further highlights the need for SCC studies utilising natural chloride migration techniques. RCMT will, however, still be presented and discussed in this literature review due to the fact that a) there is an insufficient number of natural chloride migration tests studies and real marine environment SCC data available to date and b) the patterns identified by Loser et al. results relating to the appropriateness of the RCMT require verification, but other studies examining SCC using RCMT and natural chloride migration tests have not been carried out in the literature to date.

Considering only the bulk diffusion test results in Table 3.1 it can also be noted that Loser et al. found the CEM III/A SCC to result in the smallest diffusion coefficient of the SCCs with the OPC+ 20% PFA also resulting in an improvement over the OPC control SCC. Comparing the SCCs and VCs across the different concrete types cannot be done fairly based on the Loser et al. results in Table 3.1 due to the fact that no VC was tested with the same w/b ratios as the SCCs.

Audenaert et al. (2005) also published a paper on natural chloride migration in SCCs at the 1st International Symposium on Design, Performance and Use of Self-Consolidating Concrete. The paper examined SCC and VC for both CEM I concrete and CEM III-A concrete. The test method adopted in the experimental study involved cyclic wetting and drying of concrete discs through slow rotation of the discs, which were partially immersed in a salt solution. Figure 3.1 below shows the Audenaert et al. (2005) test set-up.

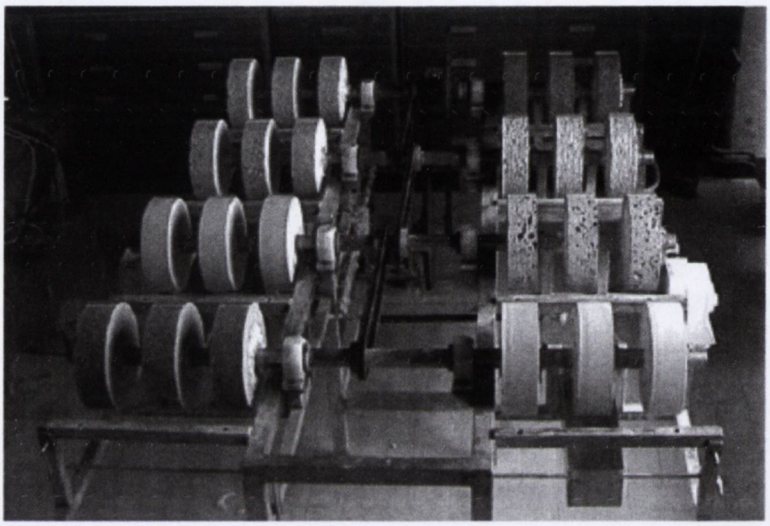


Figure 3.1 Audenaert et al cyclic exposure test set-up (Audenaert et al. 2005)

The experimental program examined SCC and VC with a w/b ratio of 0.46, but, limestone powder was used for the SCCs together with a superplasticiser to achieve the desired workability. The limestone filler was used as an aggregate replacement or filler rather than a partial cement replacement. Thus the binder contribution of the limestone powder was not accounted for in the VC and SCC comparison. The exact GGBS content of the CEM III/A cement was not known but was in the region of 34-65% as per EN206-1:2000. All samples were cured for 28 days before exposure and were subject to the wetting and drying cycles for up to 36 weeks. After the exposure period the samples were broken open and sprayed with $AgNO_3$ to obtain the approximate depth of the chloride front penetration. No chloride profiles were obtained from the samples and thus no values of the apparent diffusion coefficient or the surface chloride content were obtained. It must also be noted that the samples were not coated on any face with sealant paint and thus the chlorides were free to ingress from all faces of the sample. This will lead to the size of the sample influencing results.

The short nature of the Audenaert et al. conference paper means that the results were not discussed in great detail. It was found, however, that the use of the CEM III/A binder resulted in an improved

performance for both the SCC and the VC with the improvement being greater in the SCC concrete. This is in agreement with the Loser et al. (2010) bulk diffusion test findings presented above. Audenaert et al. also found that the chloride penetration depth in the SCC was less than the chloride penetration depths in the VC for both the CEM I concretes and CEM III/A concretes. This may, however, have been due to the increased binder content of the SCCs which contained 240kg/m^3 of limestone powder which was not included in the w/b ratios quoted by the author.

In 2008 Yazici et al. utilised a freeze-thaw natural diffusion test to investigate the chloride related durability consequences of using PFA in high volumes as a partial cement replacement in SCC. The test involved submersion of concrete samples in a 10% NaCl solution at 20°C for 24 hours followed by placement of the samples in a -25°C environment for 24 hours. The samples underwent 90 freeze thaw cycles before they were cracked open and sprayed with $AgNO_3$ to measure the approximate depth of chloride penetration. The SCC mixes examined had a w/b ratio of 0.28 and a total binder content of 600kg/m^3 with PFA used as a direct replacement for cement in proportions of 30%, 40%, 50% and 60%. No chloride profiles were obtained from the samples meaning no C_s values or D_{app} values could be calculated. The depth of penetration of the chloride front obtained by Yazci et al. for the various SCC mixes is presented below in Figure 3.2.

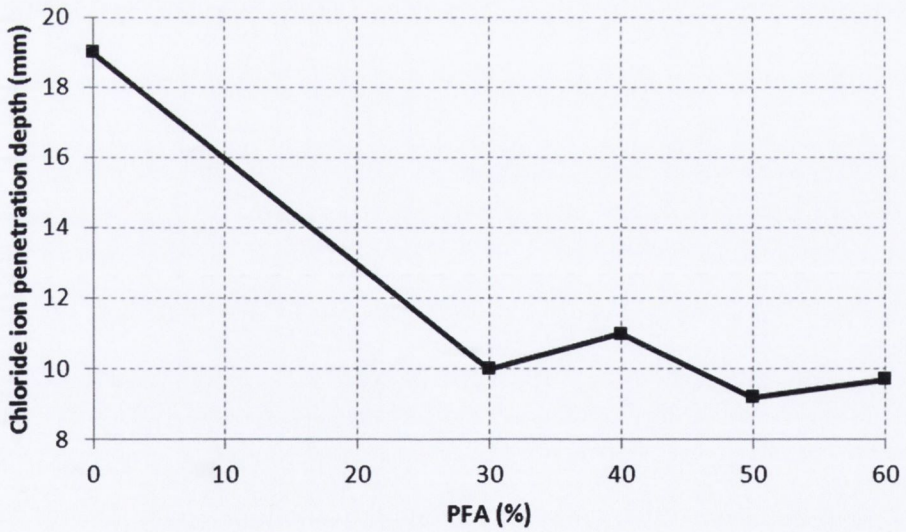


Figure 3.2 Chloride penetration observed in concretes with different PFA contents (Yazici 2008)

As can be seen from the figure Yazici et al. found that a dramatic decrease in the penetration depth of the chlorides was identified for the replacement of OPC with 30-60% PFA. It can also be noted from the Yazici et al. plot that increasing the percentage of PFA beyond 30% OPC replacement

does not seem to have a significant effect on the resistance to chloride ingress under freeze thaw cycles.

The three SCC studies discussed above constitute the extent of the relevant and available literature which compares different SCCs using natural chloride migration techniques. The relevant publications in the literature which utilise the non-steady state migration RCMT (Nordtest Method 1999) to examine SCCs will now be discussed. The first paper in this area which will be discussed is a publication by De Schutter and Audenaert (2004). The authors utilised the RCMT to compare the marine durability of SCC and VC for both CEM I and CEM III/A concretes utilising identical binder contents and w/b ratios (0.46) for CEM I and CEM III-A. The authors do not provide details as to whether any powder filler or superplasticiser was used to obtain the desired SCC workability.

The VC CEM I and CEM III/A migration coefficients obtained by De Schutter and Audenaert were 448 mm^2/yr and 369 mm^2/yr, respectively. Lower migration coefficients were obtained for the SCCs, with the CEM I and CEM III/A values equal to 363 mm^2/yr and 145 mm^2/yr, respectively. It is interesting to note that these results indicate a greater relative performance of the CEM III/A concrete in the SCC concrete. The CEM III/A relative merits obtained by dividing the CEM I migration coefficient by the CEM III/A migration coefficient for the VCs and the SCCs are 1.21 and 2.5 respectively. Again it is unknown what the exact GGBS content of the CEM III binder is as it may range from 34-65%.

In 2010 Audenaert was again a co-author of a paper which compared the performance of SCC to traditional VC using the non-steady state RCMT (Audenaert et al. 2010). This paper was focused on exploring the time dependency of the chloride migration coefficient for both SCC and VC. The authors examined CEM I concrete and CEM III/A concrete for both SCC and VC. Powder filler material and superplasticiser was used in the SCC mix to obtain the desired workability. For 15 of the 16 SCC mixes examined this filler consisted of limestone powder filler. PFA was used as a filler material in one of the mixes. The use of PFA in this form does not constitute cement replacement with PFA, as was investigated by Loser et al. (Loser et al. 2010). The study by Audenaert et al. instead utilises the PFA as an aggregate replacement material meaning the cement content in the mix is not reduced to reflect the addition of PFA. This distinction between the use of PFA as a filler material and the use of PFA as a cement replacement is an important distinction in the literature. There are a number of studies which utilise the RCPT to investigate the use of PFA and GGBS as direct cement replacement in the SCC literature (Gesoglu et al. 2009; Guneyisi et al. 2011; Nehdi et al. 2004; Sahmaran et al. 2009; Uysal et al. 2012). As previously discussed, however, the RCPT is not suitable for examining the quantitative relative performance of

alternative SCC options. As will be presented in this literature review, there are few publications available which use more appropriate testing methods, to investigate the use of GGBS, and in particular PFA, as OPC replacements in SCC.

It is noted then when comparing the CEM I and CEM III/A VCs and SCCs, the performance of the SCCs is likely to be enhanced in part by the limestone powder or PFA aggregate replacement. Ignoring the binder contribution of the filler materials the cement content of 360kg/m^3 and a constant w/c ratio of 0.46 was used for both the SCC and VC mixes. Audenaert et al. present the results in the form of the chloride migration coefficient at a time of 1 year and a corresponding time dependant diffusion coefficient *m*. This time dependant diffusion coefficient will be discussed in detail in Chapter 6 in the context of model development. The CEM I and CEM III/A chloride migration coefficients obtained for the SCC were 175 mm^2/yr and 52 mm^2/yr, respectively. The corresponding values obtained for the CEM I and CEM III/A VC were 199 mm^2/yr and 120 mm^2/yr. As might be expected the SCC which utilised PFA as a filler material resulted the much lower migration coefficient of 4.7 mm^2/yr.

It can also be noted that for both the CEM I concretes and the CEM III/A concretes the SCCs have a lower migration coefficient. Again this may be due, in part, to the effect of the limestone powder in the SCC concretes. It can also be noted from the results that the relative merit value for the CEM III/A concrete is 3.34. The corresponding value for the VC is 1.66. These findings are in agreement with the De Schutter and Audenaert (2004) publication discussed above where the migration coefficient was found to be greater for the VC than for the SCC and the relative merit value for the CEM III concrete was found to be greater for the SCC than the VC, with values of 2.5 and 1.2 obtained for SCC and VC respectively (De Schutter and Audenaert 2004).

In 2003 Zhu and Baratos also compared the chloride resistance of SCC and VC using the non-steady state RCMT (Nordtest Method 1999). The experimental programme examined C40 and C60 grade concretes for both VC and SCC. Two types of VC were examined, one with OPC, and one with PFA utilised as a partial cement replacement. The three SCC types were examined; OPC with a VMA, OPC with a limestone filler and OPC with PFA. The nature of the replacement of OPC with PFA is different in the C60 concrete and C40 concrete mixes. The use of PFA in the C60 mix is close to uniform replacement of OPC by weight for both the VC and the SCC and thus the C60 strength grade results will be discussed herein. For the VC C60 mix the PFA is utilised as a 20% direct replacement for OPC cement with both the OPC and OPC + PFA containing 465kg/m^3 binder. For the C60 SCC mix the OPC + PFA concrete contains 20% PFA and total binder content of 510kg/m^3 while the plain OPC SCC mix contains 475kg/m^3 of binder. The w/b ratio must,

however, also be considered. The mix designs utilised by Zhu and Baratos result in a very similar water/OPC ratio for all the C60 strength concretes. Consequently, while the PFA was used as a cement replacement in terms of total cement content, the lowering of the water content in the PFA mix designs means that the PFA was used as a filler material or aggregate replacement when considering w/b ratio. As the w/b ratio has a greater influence on durability than the total binder content the role of PFA in this study can be considered to be as a filler material.

The chloride migration coefficients obtained by Zhu and Baratos (2003) for the OPC and OPC + 20% PFA SCCs were 511 mm^2/yr and 199 mm^2/yr, respectively. The corresponding values obtained for the OPC and OPC + PFA VCs were 297 mm^2/yr and 104 mm^2/yr, respectively. It was thus found that the OPC + PFA concrete results in lower migration coefficients than the OPC concrete for both the SCC and the VC. This performance is, however, likely to be due, at least in part, to the reduced w/b ratio in the OPC + PFA. Thus the change in marine durability due to the use of PFA binder as a replacement for OPC binder cannot be directly quantified on an equal binder level from this study. It must be noted in the context of the *k*-value previously discussed that there is some merit to this. Yet, the k-value of 0.4 specified in EN 206 is not utilised in this study. Instead the concretes are compared with equal water/OPC ratios meaning 0% of the PFA content contributes to the w/b ratio.

In terms of the comparison of VC and the SCC it is noted from the results that the migration coefficients obtained for the SCC were higher than the migration coefficients found for the VC for both the OPC concrete and the OPC + PFA concretes. This contradicts the findings of the Audenaert et al. (2010) and De Schutter and Audenarert (2004) studies which found that for a given w/c ratio the SCC OPC concrete had a lower migration coefficient than the equivalent OPC VC. It should be noted that the SCC investigated by Zhu and Bartos does not utilise a limestone filler. Thus the Zhu and Baratos results indicate that the improved chloride resistance properties for SCC identified by Audenaert et al. (2010) and De Schutter and Audenaert (2004) may have been due to the binder contribution of the limestone powder used in their SCC mixes. This issue will be discussed further in Chapter 5 of this thesis.

In 2009 Sonebi and Nanukuttan published a very similar study to that carried out by Zhu and Baratos (2003). The mix designs used by the two sets of authors for the high strength concretes are almost identical. Table 3.2 below shows the mix design details utilised by Sonebi and Nanukuttan (2009) for the OPC and OPC + PFA concretes with the mix design details utilised by Zhu and Baratos (2003) also shown in the table in brackets.

Table 3.2 OPC and PFA mix design details from Sonebi and Nankatuun (2009) with Zhu and Baratos (2003) mix design details shown in brackets in the table

Mix Constituents	VC OPC (kg/m3)	VC OPC + PFA (kg/m3)	SCC OPC (kg/m3)	VC OPC + PFA (kg/m3)
Water	196 (195)	170 (169)	196 (196)	180 (179)
OPC	460 (465)	375 (375)	475 (475)	410 (410)
PFA	0 (0)	95 (95)	0 (0)	115 (100)
Total Binder	465 (465)	470 (470)	475 (475)	525 (525)
Sand	700 (620)	720 (625)	990 (930)	990 (930)
Coarse Agg.	1100 (1085)	1100 (1100)	760 (750)	760 (750)
w/b	0.43 (0.42)	0.36 (0.36)	0.41 (0.41)	0.34 (0.34)
w/c	0.43 (0.42)	0.45 (0.45)	0.41 (0.41)	0.34 (0.34)

The nature of the testing utilised in the two studies was, however, different. Zhu and Baratos (2003) utilised the traditional non steady state RCMT test set-up developed by Tang and Nielson (1993) and formalised in NT BUILD 492 (Nordtest Method 1999). Sonebi and Nanukuttan (2009) on the other hand used an in place chloride migration test to determine the chloride migration coefficient. This device, developed at Queens University Belfast (Nanukuttan et al. 2006), is stated to be similar to the principles of the steady state RCMT developed by Andrade et al. (1993), but it is designed to allow the chloride migration coefficient to be obtained from the face of the concrete without core extraction.

When examining the two sets of results it must also be noted that Sonebi and Nanukuttan (2009) conducted their tests at a concrete age of 5 months. The age of testing is not stated in the Zhu and Baratos publication, but, the testing age is likely to have been shorter than 5 months as it is stated in the paper that only 7 days of specimen curing could be afforded to the specimens before test preconditioning due to the time limitations of the study. Table 3.3 presents the results of the Sonebi and Nanukuttan study for the concretes containing OPC and OPC + PFA for both VC and SCC. The Zhu and Baratos (2003) migration coefficient results are also shown in the table in brackets to facilitate comparison.

Table 3.3 Sonebi and Nanukuttan (2009) test results with Zhu and Baratos (2003) results shown in brackets

Concrete Mix	OPC (mm^2/yr)	OPC + 20% PFA (mm^2/yr)	PFA Relative Merit
VC	66 (297)	63 (104)	1.05 (2.8)
SCC	142 (511)	35 (199)	4.1 (2.6)

Firstly considering the comparison of SCC and VC, it can be noted that for the Sonebi and Nanukuttan results the migration coefficient for the OPC SCC is greater than the migration coefficient for the VC. This in is agreement with the findings of Zhu and Baratos (2003). When considering the OPC + PFA concrete however Sonebi and Nanukuttan found that the SCC has a lower migration coefficient than the VC. Zhu and Baratos identified the opposite pattern as can be seen from the figures in the brackets in Table 3.3. The most likely explanation for the difference in findings is the different test apparatus adopted for the two studies. The variation associated with the new test method utilised in the Sonebi and Nanukuttan study is not reported in their publication. Zhu and Baratos however present the migration coefficient variation which ranges from 0.8% to 18%. The time of testing may also have had an effect on the migration coefficient although the patterns observed in Table 3.3 are not consistent with the expected effects of the time dependant migration coefficient assuming that the Sonebi and Nanukuttan samples were tested at a greater age.

When comparing the OPC and OPC + PFA VC mixes Sonebi and Nanukuttan found that the OPC + PFA migration coefficient was only slightly smaller than the OPC migration coefficient. A greater difference was detected by Zhu and Baratos. For the SCC both papers found that the OPC + PFA resulted in a smaller migration coefficient than the OPC concrete. In the case of Sonebi and Nanukuttan the migration coefficient was smaller by a factor of 4.1 while the Zhu and Baratos found the OPC + PFA migration coefficient to be smaller by a factor of 2.6 as can be seen from Table 3.3.

In interpreting their results Sonebi and Nanukuttan note that migration coefficients in their study were affected by the nature of the supplementary materials, especially with the SCCs. The authors also state that as expected the migration coefficient for the high strength SCC in their study was lower than the migration coefficient for the medium strength SCC due to the higher binder content of the high strength SCC. This discussion fails to allude to the fact that, just as the higher binder content will affect the medium and high strength OPC cements, the higher binder content in the OPC + PFA SCC resulting from the use of PFA as a filler type material, will affect the OPC and OPC + PFA comparison. Thus the statement by the authors that the cause of the improved performance in the OPC + PFA SCC is due to the nature of the PFA material is not entirely accurate. This point highlights the need for studies which utilise appropriate test methods to assess PFA as a direct replacement for OPC to facilitate analysis of the actual effect of the PFA material properties on SCC rather than a combined effect of material properties and reduced w/b ratio. As previously sated however, with the exception of the Loser et al. (2010) study, such an SCC comparison does not exist in the published literature for a test method other than the RCPT.

The steady state migration coefficient of SCC, which Sonebi and Nanukuttan measured, has been investigated by a small number of other researchers. Assie et al. (2007) utilised an adaption of the traditional steady state RCMT (Andrade et al. 1997) discussed in Chapter 2 to investigate the chloride transport in SCC. This adaption was developed by Truc et al. (2000) to allow the chloride diffusion coefficient to be calculated from the non-steady state RCMT set-up. Assie at al. used this test to investigate the marine durability of VC and SCC however, the use of PFA or GGBS, either as cement replacements or filler materials, was not investigated in the study. Three OPC SCC strength classes were assessed; C20/25, C40/50 and C60/75. EN 206-1:2000 specifies a minimum reinforced concrete strength class of C30/37 for any form of chloride exposure. Consequently the C20/25 results are not discussed herein. CEM I cement was used for the C40/50 and C60/75 strength with equal cement content used in the VC and SCC mixes at each strength class. In order to achieve the desired workability in the SCC, limestone filler was used in combination with increased water content.

For the concrete mixes used Assie et al. found that the effective diffusion coefficient obtained from the RCMTs were similar for SCC and VC for both C40/50 and C60/75 strength classes. Effective diffusion coefficient of 99 mm^2/yr and 92 mm^2/yr were obtained for the C40/50 VC and SCC respectively. The C60/75 strength class yielded effective diffusion coefficient of 56 mm^2/yr and 92 mm^2/yr for the VC and SCC, respectively.

Hwang and Khayat (2009) also published a paper which utilised the steady state RCMT developed by Truc et al. to derive the diffusion coefficient for a number of SCC mixes. The experimental programme was aimed at evaluating the influence of mixture parameters on the durability characteristics of SCC mixes typically used for infrastructure repair applications in Canada. The study explored the effect of utilising different admixtures, binder types and w/b ratios on the durability of SCC. Three VC mixes were also examined. The Canadian binder types used by Hwang and Khayat are different from the binder combinations discussed for the other SCC studies presented herein. In addition an Air Entrained Admixture (AEA) was utilised in all the Hwang and Khayat mixes to increase resistance of the concrete to freezing and thawing, a durability concern in Canada. The binders used by the authors in the SCC were as follows: OPC + 30% PFA +5% silica fume, OPC + 25% PFA + 5% silica fume, OPC + 25% GGBS + 5% silica fume and OPC + PFA + GGBS + silica fume in unknown proportions. The supplementary materials were used as direct replacements for OPC in the mixes. No powder filler material was utilised in the SCC mixes. Two water contents were used to facilitate examination of SCC at a w/b ratio of 0.36 and 0.42. The SCC with 0.36 w/b ratio was constructed with the OPC + 30% PFA + 5% silica fume only. The

remainder of the SCC mixes were constructed using the 0.42 w/b ratio. Two VC mixes were constructed using OPC + 25 % PFA + 5 % silica fume. These had w/b ratios of 0.35 and 0.42. Unusually, the VC with the 0.42 w/b ratio had the same coarse and fine aggregate proportions as the SCC mixes while the other VC had higher coarse/fine aggregate proportions consistent with typical VC mixes. No OPC control concretes were tested for VC or SCC in the experimental programme.

In total Hwang and Khayat tested 12 different concrete mixes with a variety of different admixtures used. The large amount of combinations means that comparison between SCCs with different binder types and comparison between VC and SCC is somewhat difficult. Table 3.4 however presents details of the comparable mixes together with the diffusion coefficient results obtained for each mix. It should be noted that Hwang and Khayat also performed RCPTs. The results of these tests are also presented below to illustrate the difficulty in ascertaining relative performance of various concretes using RCPT results. Both the RCMT and the RCPT were conducted at 56 days. No details are provided in the paper as to whether duplicate sample testing was carried out.

Table 3.4 Diffusion coefficient and RCPT results obtained by Hwang and Khayat (2009)

Mix No.	Mix Name	VC or SCC	Binder Type	w/b Ratio	Diffusion Coefficient (mm^2/yr)	RCPT Result (Coulombs)
1	RC-1	VC	OPC + 25% PFA + 5% SF	0.42	32	630
2	42-C3-B1	SCC	OPC + 25% PFA + 5% SF	0.42	79	505
3	42-C3-B2	SCC	OPC + 25% GGBS + 5% SF	0.42	107	905
4	42-C3-B3	SCC	OPC + PFA + GGBS + SF (UP)	0.42	110	185
5	35-C3-BC	SCC	OPC + 30% PFA, 5% SF	0.35	66	315
6	RC-2	VC	OPC + 25% PFA + 5% SF	0.35	82	240
7	RC-3	VC	OPC + 8 % SF	0.40	73	6000

SF: Silica Fume, UP: Unknown Proportions

The results in Table 3.4 will first be used to compare SCC to VC. Comparing mix 1 and mix 2, which both have the same w/b ratio and binder type, it can be noted that the SCC results in a larger diffusion coefficient than the VC. The same pattern is observed when comparing mix 5 and 6, which have the same w/b ratio but a slightly different binder composition. This is in agreement with the findings of Zhu and Bartos while contradicting the findings of Audenaert et al. (2010) and De Schutter and Audenaert (2004). This trend is further indication that the improved performance in the Audenaert et al. (2010) and De Schutter and Audenaert (2004) for SCC is due to the influence of the limestone powder filler which supplies an additional binder contribution.

Mixes 2, 3 and 4 can be used to examine the effect of binder on diffusion coefficient in SCC with constant w/b ratio and binder content. As can be seen from the table the OPC + 25% PFA + 5% SF and the OPC + 25% GGBS + 5% SF have almost identical diffusion coefficients. The blended cement with unknown proportions of SF, PFA, GGBS and OPC results in the lowest diffusion coefficient however as the binder proportions are unknown this finding is of little use. Finally in terms of binder content it can be noted that the OPC + 8% silica fume VC, which is the closest to a control OPC mix, results in the largest diffusion coefficient of all the mixes shown in Table 3.4.

The results obtained by Hwang and Khayat presented in Table 3.4 above also illustrate the difficulties associated with using the RCPT to determine the relative performance of concretes. Examining the results for mixes 1-3 it can be noted that mixes 2 and 3 were found by Hwang and Khayat to have almost identical diffusion coefficients however if the RCPT results are used as a measure of relative performance mix 3 appears to be almost twice as resistant to chloride migration as mix 2. Similarly upon examination of the mix 1 and mix 2 results the diffusion coefficients indicate that mix 1 is approximately 30% more resistant to chloride ingress than mix 2. The RCPT results however indicate that mix 2 is more resistant to the ingress of chlorides by approximately 20%. Finally comparing, RC-1 and RC-3 it can be seen that the RCPT results indicate that RC-1 is approximately 10 times more resistant to chloride ingress than RC-3 while the diffusion coefficient results indicate that RC-1 is approximately 2 times more resistant to chloride ingress than RC-3. Although this lack of agreement is not alluded to by the authors the comparison of the RCPT results and the RCMT results are important as they highlight a problem with one of the forms of testing. As previously stated the RCPT, as per ASTM C1202, is not intended to facilitate evaluation of the quantitative relative performance of concrete based on Coulomb readings and has been found to be unsuitable for comparing concretes with SCMs to OPC concretes (Shi et al. 1998). It is however used by many authors to do so in the investigation of SCC (Guneyisi et al. 2011; Nayak et al. 2011; Pathak and Siddique 2012; Uysal et al. 2012).

3.2.3 SCC Literature Summary

The comprehensive literature review of the existing work in the area of marine durability testing of SCC has revealed the following:

There is a need to assess the chloride resistance characteristics and chloride transport properties of SCC using natural chloride migration testing and site based evaluation. To date there are very few studies which utilise natural chloride migration tests to analyse either chloride resistance

characteristics (Audenaert et al. 2005; Yazici 2008) or chloride transport properties (Loser et al. 2010) of different SCCs. This need is intensified by the call of the RILEM committee to exercise caution when using testing techniques which have been developed for VC. The concern surrounding the use of electrical based accelerated methods for SCC has been justified by the findings of Loser et al. (2010), whose results indicated that the non-steady state RCMT largely overestimated the relative performance of GGBS SCC. In addition the problems associated with the RCPT were highlighted by the work of Hwang and Khayat (2009), whose results illustrated the inability of the RCPT to examine the relative performance of concretes containing SCMs, an issue comprehensively explored by Shi (1998) and Scanlon and Sherman (1995). Natural chloride migration testing brings chloride transport to its simplest form meaning the relatively unexplored area of SCC can be investigated without the unknown influences of external electrical forces and highly accelerated testing durations.

When assessing the marine durability performance of SCC it is important to also assess the durability performance of VC for an equivalent VC mix. The need for SCC and VC marine durability comparison has been highlighted by a number of researchers in the literature (Sonebi and Nanukuttan 2009; Zhu and Bartos 2003; Zong et al. 2011). From the presentation of the available literature herein it is clear that there is a shortage of studies examining SCC and VC with equal w/b ratios utilising appropriate testing techniques (Audenaert et al. 2005; Audenaert et al. 2010; De Schutter and Audenaert 2004; Hwang and Khayat 2009; Zhu and Bartos 2003). There is also a distinction within this area between studies which do, and do not, utilise a filler material in the SCC mixes. Consequently, only very limited studies are available which utilise an appropriate testing method to consider VC and SCC, with supplementary cementing materials used as an OPC replacement rather than as a filler material (Hwang and Khayat 2009; Loser et al. 2010; Zhu and Bartos 2003). Two of these studies utilise testing accelerated by electrical current (RCMT), while the other study by Loser et al. compares SCCs and VCs with differing w/b ratios.

Only one relevant publication could be identified in the literature which studied the chloride resistance characteristics of SCC using a test method which incorporates a level of drying in the exposure regime (Audenaert et al. 2005). This study did not, however, explore the chloride transport characteristics of the SCCs. All other studies of SCC which explored chloride transport characteristics examined the process in a fully saturated state either without a current applied (ASTM 2011) or with a current applied (ASTM 2012; Nordtest Method 1999). This constitutes a considerable gap in the existing literature when it is considered that corrosion does not in general cause problems in submerged concrete due to a lack of available oxygen as proven experimentally by Hussain (2011).

Limited studies also exist which compare the use of PFA as a partial cement replacement for OPC in SCC to a control OPC SCC with consistent w/b ratio using a test other than the RCPT (Loser et al. 2010; Yazici 2008), with the Yazici et al. study examining this area using a freeze thaw chloride test. It is also noted that the effect of silane surface treatment on the transport properties of SCC have not been assessed in the available literature to date.

These important gaps in the existing literature resulted in the development of Research Question 1, previously presented in Chapter 1 but listed again below for clarity sake;

Research Question 1: How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

This question was addressed experimentally in this PhD by firstly comparing the performance of a number of different SCC options. These options included OPC SCC, OPC + PFA SCC with PFA used as a cement replacement, OPC + GGBS SCC with GGBS used as a cement replacement, and OPC with Silane surface treatment. This comparison was carried out through conducting natural chloride migration laboratory testing which incorporated wetting and drying cycles using a salt fog chamber. This work is described primarily in Chapter 4, but also in Chapter 5.

Secondly, in order to fully understand the performance of SCCs a comparison must be made with the more established traditional vibrated concretes as a form of benchmark. Consequently, salt fog chamber tests was also utilised to compare the chloride transport properties of SCC and vibrated concrete for both OPC concretes and concretes with GGBS and PFA used as partial cement replacements. This comparison was carried out without the use of filler materials i.e. PFA and GGBS used as direct cement replacements. This work is discussed in Chapter 5 of this thesis.

3.3 EFFECT OF EXPOSURE CONDITIONS ON RELATIVE PERFORMANCE OF DIFFERENT CONCRETES

3.3.1 Context of Study in Literature

This section deals with the research published in the literature to date which examines the effect of exposure condition on the ingress of chlorides into concrete. However, before the existing studies

in this area are presented, one must consider why this area needs to be explored. A large amount of studies have been published in the literature to date which compare the marine durability of different types of concrete i.e. OPC, OPC + GGBS, OPC + PFA. However, the majority of this laboratory testing has been carried out with concrete specimens in a fully submerged state (Ampadu et al. 1999; Asrar et al. 1999; Audenaert et al. 2010; Gesoglu et al. 2009; Guneyisi et al. 2011; Guneyisi et al. 2005; Hooton and Titherington 2004; Loser et al. 2010; McNally and Sheils 2012; Nehdi et al. 2004; Pathak and Siddique 2012; Sahmaran et al. 2009; Stanish and Thomas 2003; Yildirim et al. 2011; Zhu and Bartos 2003). As discussed in the previous chapter chloride ingress testing in the submerged state is carried out under two different conditions. One condition, termed natural chloride migration or natural diffusion testing, involves submerging the specimens in a silane solution for the duration of testing and measuring the extent of chloride ingress at the end of the exposure period (American Association of State Highway and Transportation Officials 1980; Barnes and Trottier 2008; Nordtest Method 1995; Stanish and Thomas 2003; Tamimi et al. 2008). The more popular approach however is to examine the concrete in a fully submerged condition with a current applied to accelerate chloride movement in tests such as the RCMT or the RCPT, which were discussed in the previous chapter (Ampadu et al. 1999; Asrar et al. 1999; Audenaert et al. 2010; Guneyisi et al. 2005; Hooton and Titherington 2004; McNally and Sheils 2012; Yildirim et al. 2011).

Chloride ion ingress is studied in a fully submerged condition for two valid reasons. Firstly, and most importantly, fully submerging concrete samples and applying a current to accelerate the process of chloride movement allows results to be obtained in a matter of days. This represents a considerable time and cost saving when it is considered that natural chloride migration test durations are of the order of months. As discussed in the previous chapter however, a number of disadvantages have been identified with the application of a current to accelerate chloride movement in concrete (Rilem Technical Committee et al. 2008; Scanlon and Sherman 1995; Shi et al. 1998; Spiesz et al. 2012; Spiesz and Brouwers 2010; Stanish et al. 1997).

The second reason for testing concrete in a fully saturated condition relates to the moisture condition of the interior of the concrete during real marine exposure. It is recognised that the interior of concrete remains constantly saturated in most marine environments (Neville 1995). This is due to the fact that wetting of concrete occurs very rapidly while the drying of concrete is much slower (Neville 1995). However it is also recognised that wetting and drying cycles, and the relative length of the wetting and drying times, experienced by concrete in the marine environment can have a significant influence on chloride movement (Hong and Hooton 1999; Neville 2011). Testing of concrete in a fully saturated state, without any drying component incorporated, ignores

this fact, focusing solely on the transport of chlorides by diffusion. In addition as pointed out by Neville (1995), and proven experimentally by Hussain (2011), corrosion does not, in general, occur in fully saturated concrete due to a lack of oxygen at the depth of the reinforcement. Thus due to the availability of oxygen at the reinforcement and the role of absorption, leading to rapid chloride movement to a shallow depth, reinforced concrete elements in the splash, spray and tidal zones of marine structures, which are subject to wetting and drying cycles, are most at risk of reinforcement corrosion (Dhir et al. 1994; Neville 1995; Ye et al. 2012).

While testing concrete in a submerged condition allows a standard and uniform test to be developed, the discussion above highlights the fact that there is a need to also examine the marine durability characteristics of concrete in conditions which facilitate some level of concrete drying. This is especially true when considering the relative merits of different concrete types, as relative performance may be effected by the wetting and drying conditions which will be experienced in the critical zones of a reinforced concrete structure in service. For example, an OPC + PFA concrete could perform far better than an OPC concrete in the fully saturated condition, but could be only marginally better than the OPC concrete under exposure conditions with wetting and drying. Thus this OPC + PFA concrete may be optimal for construction in submerged zones but may not be appropriate in the tidal zones.

A plot produced by Zhang and Jin (2011), shown below as Figure 3.3, provides a practical grounding for the study of the effect of varying wetting and drying cycles. The plot shows the relationship between wetting and drying times with altitude for the tidal zone. It is illustrated how, for the tidal zone areas considered by Zhang and Jin, contact time with salt water ranges from 14.9 hours to 1.2 hours per day. Variation will increase moving into the splash zone, and from point to point along the splash zone. Zhang and Jin state that, when considering the changes in exposure, as the point considered is moved away from the low tide level the accumulation of chloride on the surface of the concrete is at first aggravated due to the increasing drying time proportion, and then weakened due to the reduction in contact time with chlorides.

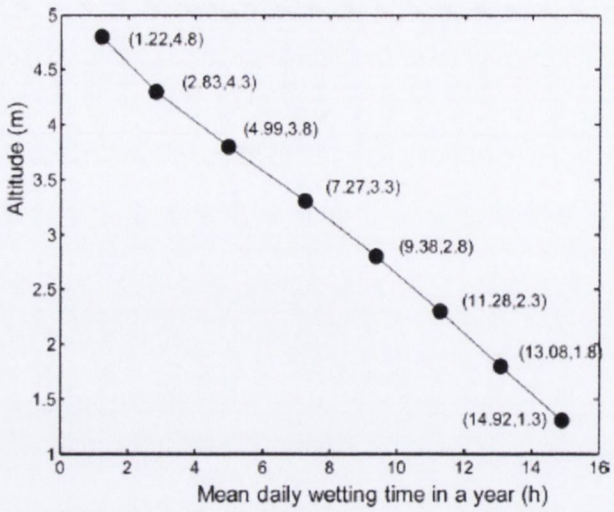


Figure 3.3 Variation of wetting time with position in tidal zone (Zhang and Jin 2011)

The Zhang and Jin (2011) publication did not consider different concretes in their experimental programme which utilised salt spray chamber testing. Only an OPC VC was considered in the study, which was focused on the development of a mathematical model of chloride transport in concrete under wetting and drying conditions. Modelling this effect is useful in the context of service life prediction, as will be discussed in Chapter 6, however it is noted that it is intuitive that different levels of wetting and drying will lead to changes in chloride ingress in a single OPC concrete. However, the effects of exposure condition on relative performance is less intuitive and must be examined experimentally by investigating different concrete types using a series of tests which utilise different wetting and drying cycles. But testing incorporating wetting and drying must be carried out under natural chloride migration conditions. As pointed out by Spiesz and Brouwers (2010) laboratory testing of concrete under such conditions is rare in the literature due to the fact that the associated tests are time consuming, costly and labour intensive. Consequently, as will be shown in the following section, there are a very limited number of studies which consider this important effect of exposure condition on the relative performance of different concrete types.

3.3.2 Publications Examining Effect of Exposure Condition on Relative Performance of Different Concretes

In 2012 Ben Fraj et al. published a study which partially addressed this issue in the context of vibrated concretes by investigating the chloride transport properties of OPC VC and OPC + GGBS VC using a number of different wetting and drying cycles. The testing saw the examination of chloride transport properties across three different relative humidity levels, implemented during drying cycles. These were 90% RH, 75% RH and 50% RH. For each of the relative humidity levels

a wetting and drying cycle of 6 hours drying followed by 6 hours wetting was adopted. Figure 3.4 below shows the test set up utilised by Ben Fraj et al. As can be seen from the figure the wetting cycles were controlled by the depth of water in the immersion tank. The test durations ranged from 1-30 days with chloride profiles, C_s, and D_{app} values obtained from the samples at the end of the test cycles. Three VCs were tested, a C30/37 OPC concrete with a w/b ratio of 0.7, a C60/75 OPC concrete with w/b ratio of 0.48 and an OPC + GGBS C60 concrete with a w/b ratio of 0.48. The use of PFA was not considered in the experimental programme despite studies in the literature suggesting PFA has a poorer sorptivity and absorption performance than GGBS (Guneyisi et al. 2011). It should be noted that the exposure periods utilised by Ben Fraj et al. were relatively short at a maximum of 30 days. In addition the 100mm diameter 50mm high concrete specimens tested in the 50% and 75% RH conditions were oven dried to 50% RH and 75% RH respectively before testing. Given the short duration of the tests, and the dry state of the samples before testing, initial water absorption may have had an impact on the experimental results. This initial absorption effect would result in a) increased rates of ingress over the short exposure period and b) greater ingress with lower relative humidity skewing the results. It is however difficult to know to what extent the results were affected by initial absorption.

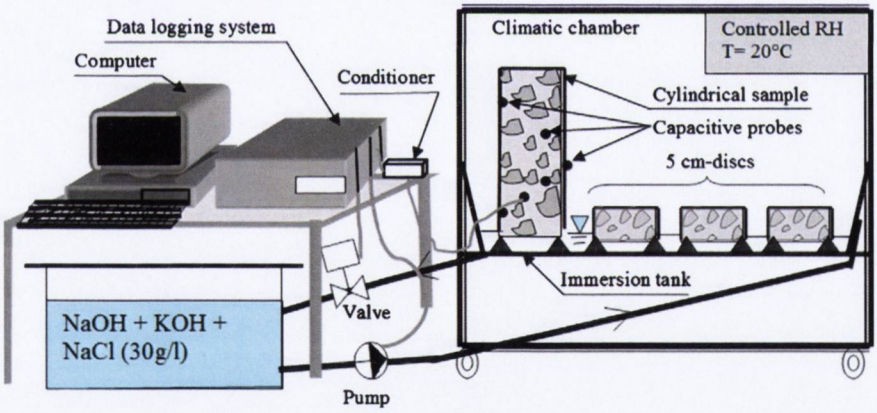


Figure 3.4 Wetting and drying test set-up utilised by Ben Fraj et al. (Ben Fraj et al. 2012)

The results of the experimental study showed that the apparent diffusion coefficient was increased to a greater extent for the GGBS concrete than the OPC concrete with reducing RH during the drying cycle. For the 50% RH condition the OPC diffusion coefficient was found to be 1.8 times that of the OPC diffusion coefficient at 90% RH. However for the OPC + GGBS concrete the diffusion coefficient at 50% RH was 5.7 times that of the OPC + GGBS diffusion coefficient at 90% RH. Consequently, it can be said that the performance of the GGBS concrete relative to the OPC concrete reduced with reducing RH levels during the drying cycles.

Hong and Hooton (1999) also went about examining the effects of cyclic wetting and drying regimes on chloride ingress in different concretes. The study exposed two sets of concretes specimens to two different wetting and drying regimes; a 6 hours wet, 18 hour dry regime (1 day series), and a 6 hour wet, 66 hour dry regime (3 day series). The samples were immersed in containers filled with a 1 molar NaCl solution during the wetting cycles and placed in an enclosed chamber with relatively humidity approximately equal to 50% during the drying cycle. As noted by Ben Fraj et al. (2012) this experimental design was flawed as the RH in the drying chamber was controlled with saturated $CaCl_2$ solution. Three different concrete types were investigated by Hong and Hooton; a 25% GGBS VC with a w/c ratio of 0.4, a 25% GGBS + 8% silica fume VC with a w/c of 0.4 and a 25% GGBS + 8% silica fume VC with a w/c ratio of 0.3. No plain OPC concretes or PFA concretes were investigated by Hong and Hooton.

Hong and Hooton found that for the three concretes examined, 3-day cycles resulted in greater chloride ingress compared with the corresponding number of 1-day cycles. However when comparing the two wetting and drying regimes at similar total test times it was found in general that the 3 day cycles resulted in slightly lower chloride concentrations. This is in agreement with the findings of Zhang and Jin (2011). The authors attribute this to the longer total NaCl contact time in the 1 day series, for a given test duration. To further investigate the role of wetting and drying on chloride ingress Hong and Hooton utilised results from bulk diffusion tests carried out at 120 days in order to compare the ingress as a result of wetting and drying to the predicted ingress as a result of pure diffusion. Figure 3.5 (a) & (b) below presents the comparison of 36 one day cycles with the predicted 36 days pure diffusion, and the 25 three day cycles with the predicted 75 days pure diffusion respectively. Hong and Hooton also plot the chloride profile for the 120 day pure diffusion although this is of little use in the comparison. As can be seen from the plots the wetting and drying cycles result in much higher surface chloride contents due to the effects of absorption. For the 1 day cycle the chloride content is greater than the pure diffusion at each depth plotted. For the 3 day cycles the chloride content is greater than pure diffusion up to the point of 10mm after which the pure diffusion results in greater chloride contents. This indicates that the chloride ingress as a result of wetting and drying can be greater than the ingress due to pure diffusion due to the combination of the absorption at the shallower depths and the diffusion at the deeper depths, which occurs in wetting and drying zones.

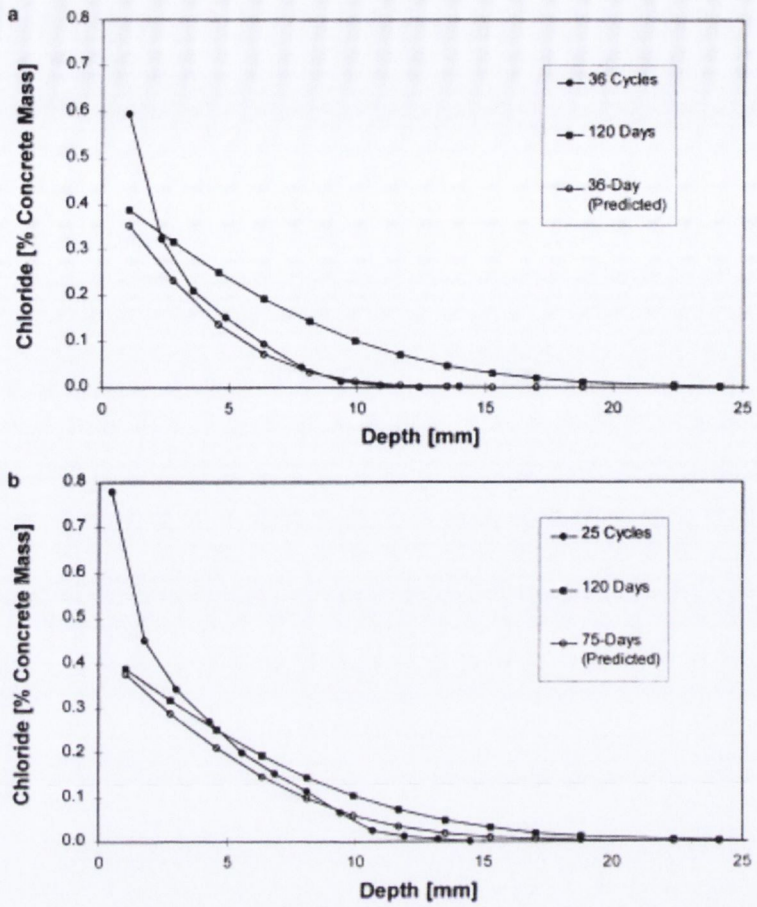


Figure 3.5 Comparison of pure diffusion and wetting and drying chloride migration for 25% GGBS concrete (a) 1 day cycles (b) 3 day cycle (Hong and Hooton 1999)

Examining the effect of drying time on the different concrete types it was found that the increase in chloride ingress, caused by longer drying time, was not the same for all three concretes. Thus the relative performance of the concretes differed across the two exposure conditions. The 25% GGBS concrete was found to be most affected by the change in exposure conditions while the concrete with the 25% slag 8% silica fume and w/c ratio of 0.3 was found to be least affected by the increased drying time.

3.3.3 Effect of Exposure Conditions: Summary

The recent publication by Ben Fraj et al. (2012), and the publication by Hong and Hooton (1999), represent the extent of the literature published in this area to date. Three things are clear from the presentation of these studies. Firstly, there is a very limited amount of research investigating the change in relative performance of different concretes across a range of exposure conditions. For instance, salt spray/salt fog chambers are established testing apparatus whose automated systems make this test method ideal for examining the effects of exposure conditions on the relative

performance of different concretes. However examination of the relevant studies published in the literature which use salt fog/spray chambers to examine chloride transport properties of concrete (Konin et al. 1998; Li 2000; Li 2001; Mangat and Gurusamy 1987; Mangat and Gurusamy 1987; Mangat and Molloy 1995; Wang et al. 2011; Zhang and Jin 2011), only the publication by Zhang and Jin (2011) paper considers different exposure conditions, and this study focuses solely on OPC concrete. This highlights the extent of the gap in the literature in this area.

Secondly, presentation of the existing literature has indicated that the exposure conditions do effect relative performance for VC from OPC concrete to GGBS concrete (Ben Fraj et al. 2012) and from 25% GGBS concrete to 25% GGBS + 8% silica fume (Hong and Hooton 1999).

Finally, the discussion in Section 3.3.1 highlighted the important of considering the magnitude by which concretes such as OPC + GGBS and OPC + PFA out-perform traditional OPC concretes across a range of wetting and drying conditions. This need becomes more pronounced when one considers the fact that the majority of experimental investigations into concretes containing PFA and GGBS has been carried out in saturated conditions.

Given these findings the following research question, previously presented in Chapter 1, was developed to build on the existing literature;

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

This research question was addressed in this PhD by examining of the relative performance of OPC SCC, OPC + PFA SCC and OPC + GGBS SCC across three different exposure conditions, each of which have a different level of drying during testing. This experimental work is described in Chapters 4 and 5 with the results and analysis of the comparison presented solely in Chapter 5. In addition, the comparison was also considered in a probabilistic framework in Chapter 7 to investigate if the patterns identified in the experimental study were reduced by time dependant effects over timeframes applicable to the service life of marine structures.

3.4 PROBABILISTIC MODELLING

3.4.1 Introduction

As discussed in the introduction this thesis is concerned with examining the initiation phase of reinforcement corrosion. Thus the modelling carried out herein sets the limit state as the time to initiation of corrosion. This approach of omitting the period of corrosion propagation is often adopted in the service life predictions due to the fact that the corrosion propagation phase commonly last only a few years before significant structural damage is observed (Bertolini et al. 2004; Sagues et al. 2000). The approach has been utilised by a number of researchers in the probabilistic deterioration modelling literature (Song et al. 2009; Stipanovic Oslakovic et al. 2010).

The progression of chlorides into concrete in this initiation phase of corrosion is however highly complex with various physical–chemical interactions among silane solutions, solid phases of concrete and moisture (Climent et al. 2002). The complicated nature of this process, and the variable nature of concrete (essentially a composite material), leads to significant levels of uncertainty when modelling the initiation phase of chloride induced corrosion in RC structures. In addition to this model uncertainty there is also significant uncertainty associated with some of the parameters which dictate chloride induced corrosion, such as the time dependent diffusion coefficient (Nokken et al. 2006) and the critical chloride content, as discussed by Angst et al. (2009).

These relatively high levels of uncertainty, combined with recent developments in computational power, have lead a number of researchers in more recent times to use probabilistic methods to represent chloride-ion ingress and make predictions about the onset of chloride induced corrosion (Bastidas-Arteaga et al. 2009; Lu et al. 2011; O'Connor and Kenshel 2013; So et al. 2009; Stewart and Suo 2009; Suo and Stewart 2009; Vu and Stewart 2000). These probabilistic methods will be used in this thesis to allow the uncertainty associated the initiation phase of the chloride induced corrosion to be incorporated into the predictive modelling. This facilitates quantification and interpretation of this uncertainty in the context of predicted performance of different concretes, leading to a robust study of service life predictions.

The literature relating to probabilistic chloride induced corrosion has examined a wide range of issues over the past number of years. Areas explored include; the improvement in statistical parameters used in probabilistic modelling (Vu and Stewart 2000), the incorporation of spatial variability into probabilistic analysis (O'Connor and Kenshel 2013; Stewart and Suo 2009), probabilistic modelling focusing on the propagation phase of chloride induced corrosion (Stewart

2009; Vu et al. 2005), and the incorporation of sensor and inspection data into probabilistic modelling (Marsh and Frangopol 2008; Rafiq et al. 2004; Zhang and Mahadevan 2000). Attempts have also been made to incorporate more sophisticated corrosion initiation models into probabilistic analysis incorporating the effects unsaturated concrete (Bastidas-Arteaga et al. 2011), weather effects (Bastidas-Arteaga et al. 2010), and the effect of structural cracking (Lu et al. 2011). Such models will be discussed in detail in Chapter 6, where it is shown that the simple Fick's law approach is still the most widely used and accepted probabilistic approach.

The probabilistic modelling in this thesis is not concerned with the incorporation of new theoretical models into a probabilistic framework, or with issues such as spatial variability. Instead, in keeping with the theme of the experimental work carried out herein, probabilistic modelling will be used to explore the service life consequences of utilising different concrete durability options. Given the relatively recent popularity of concretes containing supplementary cementing materials and silane surface treatments, a limited number of studies exist which compare such durability options to the traditional OPC approach over timeframes comparable to service life design periods (Mohammed et al. 2002; Pack et al. 2010; Schueremans et al. 2007; Thomas et al. 2008). As stated in Section 3.2 this is particularly true for SCCs, with no long term marine durability performance data currently available for SCCs. Van den Heede (2012) stated that due to the current focus on comparative durability assessment based on accelerated tests, sufficient knowledge on the actual service life and sustainability of different concretes is lacking. However, as stated in the Introduction of this thesis, the extrapolation of accelerated test data to time periods relevant to infrastructure service life is subject to significant uncertainty. This is particularly true when considering the issue of time dependant diffusion as will be discussed in detail in Chapter 6.

Consequently probabilistic methods, which allow uncertainty to be incorporated into the analysis and quantified, are ideally suited to the study of the effect of the concrete option adopted, on the predicted service life of structures. Existing work in this area is however very limited. The purpose of this section of the literature review is to present the existing research which utilises probabilistic methods to explore the service life consequences of utilising different concrete durability options. A summary discussion will highlight the gaps in this research area and illustrate how these gaps lead to the development of the key research questions of this PhD. Finally the manner by which the work described in Chapter 6 and Chapter 7 of this thesis contributes to the existing literature by addressing the research question posed is discussed.

3.4.2 Use of Probabilistic Modelling for Comparative Studies

One of the first publications in the available literature which utilises probabilistic analysis to examine the marine durability of a number of different concrete options was published by Ferreira (2008). This paper presents a probabilistic durability design approach for reinforced concrete structures which evaluates the performance of a number of design options. These design options are a) cement type used, b) cover depth specified, c) application of a hydrophobe and c) influence of the temperature in the country in which the structure exists (although temperature is not a design option). The theoretical basis for the probabilistic model is a Fickian approach, with the effects of time dependant diffusion and temperature incorporated. The publication is purely concerned with the initiation phase of chloride induced corrosion.

It is noted that a somewhat conservative service life limit state is selected by the authors at 10% probability of corrosion initiation. This 10% probability of corrosion initiation was specified in the CEB-FIB Model Code bulletin 34 (2006), and has been adopted by a number of other authors for probabilistic assessment of existing structures (Song et al. 2009; Stipanovic Oslakovic et al. 2010). A 10% probability of corrosion would seem justifiable carrying out durability design, however using a 10% probability of corrosion initiation to predict the service life of existing structures would seem conservative. In addition, as illustrated by Ryan and O'Connor (2013), assessing the relative performance of different concrete durability options at a single probability of corrosion initiation fails to account for the change in relative performance of concretes across the probability of corrosion range.

Firstly, considering the examination of the cement type, Ferreira utilises the results of tests which were accelerated using an electrical current (RCMT) in accordance with NT BUILD 492 (Ferreira 2004; Nordtest Method 1999). The concretes examined were vibrated concretes which contained CEM I binder, CEM III/B binder (66% to 90% GGBS), CEM II/A-V (6%-20% PFA) binder and CEM I + 10% silica fume binder. The authors adopted m values from the Duracrete model with OPC, PFA and GGBS mean value of 0.37, 0.51 and 0.60 respectively. The m value COVs adopted were between 14% (PFA) and 25% (GGBS). The authors found the cement type to have significant influence on service life. The defined limit state (10% probability of corrosion initiation) was violated for the OPC at 4 years, the PFA at 13 years and the GGBS at 42 years. Thus it was found that the use of CEM III/B concrete resulted in the most durable solution with the GGBS solution resulting in an 85% improvement in the probability of corrosion initiation at 50 years when compared to the OPC option. It should be noted at this point that this probabilistic modelling was based on the results of the RCMT which uses highly accelerated test conditions with an electric current applied to obtain diffusion coefficients over a matter of days. Spiesz et al. (2012) stated that

this test overestimates the diffusion coefficient while the Loser et al. (2010) results, presented in Section 3.2.2, indicated that the test may increase the relative performance of concretes containing GGBS. In addition the model utilised did not account for the historic change in the time dependant diffusion coefficient, a common error pointed out by Visser et al. (2002) and later by Pack et al. (2010).

Ferreira's examination of the effect of the hydrophobe on service life is not based on experimental testing. Consequently, the assessment of the hydrophobe effect by Ferreira is a rough approximation based on the assumption that the hydrophobe is only effective for 7-10 years after application. Performance of the surface coating is then approximated by assuming a gradual decrease of the surface coating permeability from 0 years to 7 years. Few details are provided, however, this suggests that the surface coating is assumed to be impermeable at $t = 0$ and completely ineffective at $t = 7$ years. No references or basis for these assumptions are provided in the publication. Ferreira investigates two scenarios; one with surface treatment applied once, and another with the surface treatment applied a second time after 7 years. The probabilistic analysis showed that the silane delayed degradation for the effective period as would be expected.

The investigation into the effect of the specified cover depth is carried out by Ferreira by varying the x parameter mean value in Fick's second law from 50mm to 90mm. The author states that cover has a considerable effect on probability of corrosion over service life up to a point of 60mm cover, after which there is a decreased effect for a given cover depth increase.

Finally considering temperature, Ferreira found that the probability of corrosion initiation increased significantly from an average temperature of 8°C to 18°C and from 18°C to 25°C. It is thus stated by Ferreira that deigning an RC structure at a reference temperature may result in an under performance of the structure with regards to durability for hotter climates. It is noted that unusually, the same author published an almost identical publication in 2010 in the Journal of Engineering Structures (Ferreira 2010). This 2010 paper presents the same results as the paper discussed above with identical concluding remarks and findings. Consequently, this later publication is noted but not discussed herein.

McNally and Sheils (2012) also used a combination of probabilistic methods and experimental results for different concretes to assess the service life implications of concrete type selection. The authors examined the durability of vibrated concretes constructed with CEM III/B cement and concretes constructed with CEM II cements with GGBS used as a partial cement replacement. The publication examined durability in terms of both chloride ingress and carbonation. As with the

Ferreira publication (2008) discussed above, the chloride related experimental programme consisted of carrying out a series of tests accelerated by electric current (RCMT) in accordance with NT Build 492 (Nordtest Method 1999). McNally and Sheils state that these highly accelerated tests were favoured over natural chloride migration tests due to slow nature of the natural chloride migration tests. Six different CEM II vibrated concretes with w/c ratios of 0.45 and 0.55 were investigated, which had GGBS replacement levels of 0%, 50% and 70%. In addition CEM III/B vibrated concrete, which has a GGBS content of between 66% and 80%, was also tested at the two w/c ratios. Two specimens were tested for each mix.

For the probabilistic modelling of the experimental results the authors incorporated the time dependant diffusion coefficient in accordance with the work of Pack et al. (Pack et al. 2010). The *m* values used by McNally and Sheils were also adopted from the work of Pack et al. As will be discussed in detail in Chapter 6, these *m* value of 0.06 and 0.23, for OPC and GGBS respectively, are low when compared to the other *m* values recorded in the literature. This may be due to the fact that Pack et al. used diffusion data from a number of different bridges of different ages to calculate *m* values. Again this will be discussed further in Chapter 6. It is also interesting to note that McNally and Sheils do not model the time dependent diffusion coefficient value probabilistically. This is unusual when it is considered that the purpose of the probabilistic analysis is to incorporate uncertainty, and the time dependent diffusion coefficient value and the critical chloride content value are perhaps the two parameters which are associated with the greatest uncertainty in the time to initiation of corrosion predictive model. Again the uncertainty associated with the Fick's law parameters are discussed in detail in Chapter 6.

Considering the results of the McNally and Sheils publication a number of interesting findings were made. The results were presented in terms of probability of corrosion initiation at 50 and 100 years. The addition of GGBS at a level of 50% and 70% was found to substantially increase the durability of concrete in terms of chloride ion ingress. For example, the probability of corrosion initiation calculated at 100 years for the CEM II/A-L concrete and the CEM II/A-L concrete with 70% GGBS as a direct cement replacement were 0.28 and 0.02 respectively. McNally and Sheils also found that the addition of GGBS increased the probability of corrosion initiation due to carbonation. However, the probabilities of corrosion initiation due to carbonation at 50 and 100 years in the concretes containing GGBS were still very low. Consequently it is stated by the McNally and Sheils that the pace of the chloride and carbonation ingress mechanisms differ drastically, meaning that any interaction between the two is unlikely and carbonation is thus unlikely to influence the repair and maintenance requirements of a structure subject to both forms of deterioration. Finally, the results of the experimental testing and probabilistic service life

predictions by McNally and Sheils also revealed that the performance of concrete with GGBS was added at the mixer was similar to the performance of an equivalent concrete constructed of cement with GGBS blended in a factory setting.

In another 2012 publication, Van den Heede et al. (2012) utilised a similar approach to that employed by McNally and Sheils (2012) and Ferreira (2008; 2010) in utilising the results of RCMTs (Nordtest Method 1999) in a probabilistic model to compare service life predictions for different VCs. It was stated by Van den Heede et al. state that such an approach is required as the current focus in the literature on comparative durability assessment based on accelerated tests has led to a lack of knowledge relating to the performance of different materials over service life. The papers examines OPC VC, OPC + PFA VC with cement replacement levels of 15% and 50% and OPC + GGBS VC of with cement replacement levels of 50% and 70%. The GGBS is used as a direct cement replacement. The PFA is utilised with the *k*-value efficiency factor as specified in EN 206-2:2000, however the *k*-value is not applied in accordance with EN 206-1:2000 for the OPC + PFA mix with 50% OPC replacement (Centre Europeen de Normalisation CEN 2000). This concrete has an equivalent w/b ratio of 0.50 (actual w/b ratio is 0.35), while all other mixes have equivalent w/b ratios of 0.45. OPC + PFA concretes without the *k*-value were not investigated. The RCMT was carried out at 28 days and 91 days for the OPC + PFA VCs, 28 days and 56 days for the OPC + GGBS VC and at 28 days, 56 days and 91 days for the OPC VCs.

The probabilistic analysis carried out by Van de Heede et al. was conducted in accordance with the FIB Model Code (CEB-FIB 2006). The chloride induced corrosion service life prediction model in this code is based on the Duracrete approach (1998). As pointed out by Luping and Gulikers (2007) this approach is flawed as the expression adopted by the DuraCrete model does not fulfil the underlying differential equation of Fick's second law. In addition Van de Heede et al. adopted an unusual approach to the selection of the critical chloride content and cover depth parameters from the literature. Values for the parameters were selected on the basis of a probabilistic parameter study which examined the effect of choice of values on predicted service life. The critical chloride content and cover depth values for service life assessment for the three concrete types were then based on the parameter values which returned a service life prediction of approximately 40-60 years. This appears to be a crude form of model calibration.

Considering the comparison of the OPC and OPC + PFA service life prediction it is interesting to note that the Van de Heede et al. identified a far greater OPC + PFA relative performance for the 91 day tests than for the 28 day test results. The OPC + PFA probability of failure at 100 years for the 28 day results ranged from approximately 0.006 to 0.012 while the corresponding 91 day tests

resulted in probabilities of failure ranging from approximately 0.001 to 0.002. This indicates that either the tests performed had low precision and repeatability or that the time dependant effects are appropriately incorporated in the probabilistic analysis. This could be due to the failure of the model utilised to take into account the historic change in the diffusion coefficient.

The OPC + GGBS VC investigated are based on a slightly different mix design and are compared to a slightly different OPC VC as a result. However the results presented by Van de Heede et al. indicate that the OPC + GGBS concretes preform far poorer than the OPC + PFA concretes. Given that the OPC + GGBS VCs were found to have similar diffusion coefficients to the OPC + PFA VCs at the 28 day point, this may be attributed to the fact that a time dependant diffusion coefficient of 0.60 was adopted for the OPC + PFA concrete while a value of 0.45 was utilised for the GGBS concrete. The OPC + GGBS probabilities of corrosion initiation at 100 years ranged from approximately 0.090 to 0.098 for the 28 day test results and 0.065 to a value slightly greater than 0.10 (cannot be determined from the plots presented). Again the difference in the 28 day and 56 day test results may be due to the inability of the time dependant model utilised to capture the actual reduction of the diffusion coefficient with time or a lack of repeatability of test results. The OPC VC used for the OPC + GGBS comparison resulted in a 10% probability of corrosion initiation at approximately 40 years. No details of the COV values associated with the predictions are provided in the paper.

Probabilistic methods were also used by Bentz (2003) to compare predicted T_i values for an OPC concrete and a concrete containing 40% PFA as a direct cement replacement. The paper focuses on the development of a linearized analysis which facilitates a probabilistic analysis without the need for Monte Carlo simulation or the application of the First Order Reliability Method (FORM) or the Second Order Reliability Method (SORM). The comparison of OPC concretes and OPC + PFA concretes in the paper is very brief and is used merely as an illustrative tool. The analysis assumes the same D_{app} value and C_s value for OPC and OPC + PFA concretes. This is not in accordance with findings in the materials literature (Dhir et al. 1993; Loser et al. 2010; McPolin et al. 2005). A time dependant diffusion coefficient model is adopted with an *m* value of 0.20 for the OPC and an *m* value of 0.51 used for the OPC + 40% PFA concrete. Bentz found that the 50% probability of corrosion initiation occurred at 6.9 years in the OPC concrete and at 31.1 years in the OPC + PFA concrete. It is noted however that the comparison of the two concretes is not based on measured values of C_s or D_{app} and as such is not a complete comparison of service life predictions. As stated it is instead an illustrative example which is used to test the applicability of a newly developed probabilistic method across two kinds of concretes.

Stewart et al. (2012) utilised probabilistic methods to investigate climate change adaption for corrosion control of concrete infrastructure. The paper first assesses climate change related increases in RC corrosion risks due to chloride ingress and carbonation. The authors then examine the effectiveness of a number of adaption measures which could be employed at the design and construction stages to mitigate the effects of climate changes. The measures investigated are a) increasing cover, b) reducing the diffusion coefficient and increasing strength grade and c) application of surface coatings. Each of these adaption measures was examined across three different emission scenarios and nine urban locations.

When assessing the effect of surface coatings Stewart et al. utilised a model developed by Buenfeld and Zhang (1998) which was designed to approximate the diffusion coefficient of mortars with a surface coating. The model was developed based on saturated diffusion cell testing. As stated by Stewart et al. the results based on this model are very much preliminary due to the lack of a sophisticated and well developed model to simulate chloride movement in concretes which have been surface coated. It is stated by the authors that the probabilistic modelling of chloride movement in surface treated concrete requires further thorough investigation.

The study considers three surface coatings; silane, polymer modified cementitious coating and polyurethane sealer. The analysis assumes that the surface coating utilised is maintained intact for the duration of the service life. As can be seen from Figure 3.6 the silane treatments easily mitigate the effects of climate change. The silane was found to be the least effective surface treatment yet the use of silane resulted in a drop in probability of corrosion initiation from 27.9% to 6.9% by the year 2100.

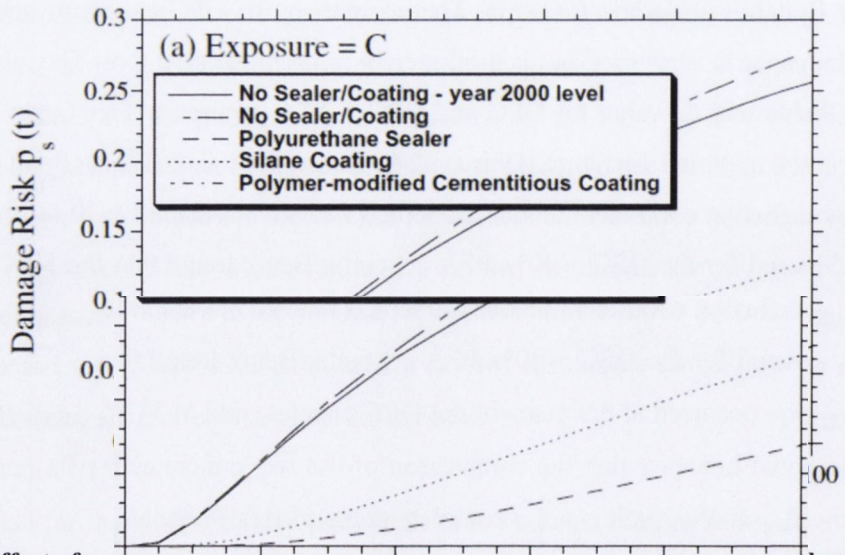


Figure 3.6 Effect of surface treatment on probabilistic of damage due to chloride induced corrosion (Stewart et al. 2012)

Considering the effect of concrete cover the authors state that this adaption measure was the most obvious and simplest considered. The probabilistic analysis revealed that a 5mm increase in cover would offset the effects of climate change in circumstances where durability was governed by chloride related corrosion. The reduction in diffusion coefficient measure is not explored on a concrete design mix or materials level. Instead a simple reduction factor approach is used. This indicates the amount by which the diffusion coefficient would have to be reduced to mitigate the increasing effects of climate change over time. The required decrease in chloride diffusion coefficient, for a service life extending to 2100, ranges from 5-10% to 6-15% depending on the emission scenario being considered.

In addition to the studies discussed above there is also a significant number of papers which assess the effect of increased cover on chloride induced corrosion using probabilistic deterioration analysis (Stewart and Rosowsky 1998; Stewart and Rosowsky 1998; Val and Stewart 2003; Vu et al. 2005; Vu and Stewart 2000; Zhang et al. 2011). The extensive investigation into the effect of cover depth on probabilistic output is likely to be due to the fact that cover depth is the only parameter in Fick's second law which is not dependent on materials or environment. It can be specified for a given structure and its effect on probabilistic analysis is easily investigated by simply varying the mean cover depth value. This will be carried out in Chapter 7 to investigate the durability performance of the increased cover option used on Ferrycarrig Bridge crosshead No. 2. However the papers investigating the increased cover using probabilistic methods will not be discussed in detail in this literature review due to a) the straightforward nature of the investigative procedure involved, b) analysis of increased cover has been discussed in the context of Stewart et al. (2012) above and c) and the fact that investigation into the effect of increased cover does not form a part of any of the key research questions presented in Chapter 1.

3.4.3 Probabilistic Modelling Summary

The discussion above has presented the available literature which utilises probabilistic methods to assess the impact of different concrete durability options on service life predictions. The following gaps have been highlighted from the discussion;

The existing probabilistic studies have compared different concrete durability options concrete, using test results that have utilised an electric current to accelerate testing. The limitations of such methods have been discussed in detail in this chapter. Thus in order to apply probabilistic predictive modelling techniques in this area to greatest effect results from the real marine

environment, or from natural chloride migration testing, should be incorporated into the assessment of the service life implications of concrete durability option selection.

Probabilistic service life prediction has not been used to date to examine SCC over timeframes appreciable to service life predictions. This is particularly important when it is considered that little or no long term monitoring details are currently available for SCCs due to the relatively recent development of this concrete technology.

None of the studies which utilised probabilistic methods to compare different concrete durability options to date have calibrated experimental results using data from the real marine environment.

The following shortcoming in the existing literature lead to the development of the third research aim for this PhD, presented in Chapter 1 and again below for clarity sake.

Research Question 3: How do different concretes, specifically SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

This research question was addressed in this PhD through the following;

The results of natural chloride migration tests were utilised in a probabilistic model to compare a number of different SCC types over timeframes applicable to service life. In addition this framework was used to compare SCCs and equivalent VCs.

The probabilistic framework was also utilised to investigate the effect of calibration of experimental test results on the relative performance. This was achieved using data from Ferrycarrig Bridge collected in 2007 by Kenshel (2009).

Finally, Ferrycarrig Bridge was used as a case study to illustrate the implementation of the probabilistic framework and calibration methods by predicting time to initiation of corrosion in each of the marine structure's SCC crosshead beams.

The development of the probabilistic model is presented in Chapter 6 of this thesis, with the modelling results presented in Chapter 7.

3.5 LITERATURE REVIEW CONCLUSIONS

The literature review has presented a detailed discussion of the relevant publications in the three key areas where the work carried out in this thesis constitutes a significant contribution to existing knowledge. The areas discussed were as follows;

- Self-compacting concrete
- Effect of exposure conditions on performance of concretes
- Probabilistic deterioration modelling

The discussion in each of these areas has highlighted a number of shortcomings in the existing literature. A research question has been developed for each of the three key areas based on these gaps in the existing literature. Finally the manner by which each research question has been addressed in this PhD has been discussed in order to give a precise description as to how the work herein contributes to existing knowledge in the area of chloride induced reinforcement corrosion.

By way of a literature review summary, the main gaps highlighted in the existing literature are presented below in concise bullet point format for each of the three research areas;

The most important shortcomings relating to marine durability in the initiation phase of reinforcement corrosion which have been highlighted in the literature, and are addressed in this thesis as follows:

SCC

- There is a lack of information relating to the chloride transport properties of SCCs obtained from natural chloride migration testing. No studies have been published to date which examine SCC C_s or D_{app} after wetting and drying cyclic exposure.
- Few studies have compared SCCs to equivalent VCs using appropriate testing techniques. In particular there is a shortage of data obtained from natural chloride migration testing with only a handful of concretes compared to date, and only one study published which carried out the comparison based on cyclic wetting and drying exposure.
- Studies examining the use of PFA as a partial cement replacement in SCC are very limited and the effect of silane surface treatment on the transport properties of SCC have not been assessed in the available literature.

Effect of Exposure Condition on Relative Performance

- Very limited research has been carried out in this area
- Existing studies indicate that exposure condition can have a notable effect on the relative performance of different concretes
- No study has to date in this area has examined OPC concrete, OPC + PFA concrete and OPC + GGBS concrete
- No publications investigate this effect in the context of SCCs

Probabilistic Modelling

- No probability based study published in the literature to date has compared OPC to OPC + PFA and OPC + GGBS, using test results obtained from natural chloride migration testing.
- Probabilistic service life prediction has not been used to date to examine SCC over timeframes appreciable to service life predictions. This is particularly important when it is considered that little or no long term data currently exists in the published literature for SCC due to the relatively new development of this concrete technology.
- None of the studies which utilised probabilistic methods to compare different concrete durability options with data from accelerated tests have calibrated experimental results using data from the real marine environment.

The key research questions for this PhD were developed based on these gaps in the existing literature. These research questions together with the manner by which they were addressed herein were presented in the Thesis Introduction, but are presented again here for completeness, together with details of the chapters where the work is presented.

Research Question 1: How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

This research question was addressed in this PhD through the following;

c) A comparison was carried out between the chloride transport properties of a number of different SCCs options through natural chloride migration laboratory testing which incorporated wetting and drying cycles.
(Phase 1 and Phase 2 testing: Chapter 4 and 5)

d) The same experimental techniques were used to compare SCC to the more established traditional vibrated concrete type for both OPC concretes and concretes with SCMs used as partial cement replacements.
(Phase 2 Testing: Chapter 5)

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

This research question was addressed herein by examining the effect of three different experimental chloride exposure conditions on the relative performance of OPC SCC and SCC containing SCMs utilised as partial cement replacements. The comparison was also considered in a probabilistic framework to investigate if the same patterns exist over timeframes applicable to the service life of marine structures.
(Phase 1 and Phase 2 Testing: Chapter 4 and 5, Probabilistic Modelling: Chapter 7)

Research Question 3: How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

This research question was addressed in this PhD through the following;

d) Probabilistic modelling was used, together with results of natural chloride migration laboratory tests, to compare the performance of different SCC types and the performance of SCCs and VCs, over timeframes applicable to service life
(Probabilistic Modelling: Chapter 6 & 7)
e) The probabilistic framework was utilised to investigate the effect of calibration of experimental test results on the relative performance using data from the real marine environment
(Probabilistic Modelling: Chapter 6 & 7)
f) Ferrycarrig Bridge was used as a case study to illustrate the implementation of the probabilistic framework and calibration methods by predicting time to initiation of corrosion in each of the marine structure's SCC crosshead beams
(Probabilistic Modelling: Chapter 7)

4. PHASE 1 EXPERIMENTAL METHODOLOGY & RESULTS

4.1 INTRODUCTION

The Phase 1 testing program was designed to address the first part of Research Question 1 by comparing the chloride transport properties of a number of different SCC options through natural chloride migration laboratory testing which incorporated wetting and drying cycles. As discussed in Chapter 3 this part of Research Question 1 was developed based on an urgent need in the literature for experimental studies assessing the marine durability of SCCs using natural chloride migration testing (Loser et al. 2010; Nehdi et al. 2004; Rilem Technical Committee et al. 2008). The Phase 1 testing was carried out in the TCD Materials Laboratory over a period of 36 weeks, with the basis for the experimental programme derived from the SCC used at Ferrycarrig in 2007. The 36 week exposure period in the accelerated chambers simulates decades of exposure in the real marine environment and thus facilitate examination of the relative performance of the Ferrycarrig Bridge SCC options in a practical laboratory timeframe. As outlined in the previous chapters the work in this thesis is solely concerned with the initiation phase of reinforcement corrosion. Thus the Phase 1 experiments investigate the marine durabilities of the SCC options through examination of the relative resistance of the SCC options to the ingress of chloride, over the testing period.

This chapter consists of three main parts. Firstly, the background to the Phase 1 experimental program is discussed. Secondly, the experimental work itself detailing the testing and analysis methods are presented. Finally a robust analysis of the results is conducted. This analysis includes a detailed discussion of the experimental program findings, comparison of the results to the available literature and a discussion on the findings of the Phase 1 testing which were investigated further in the Phase 2 testing program.

4.2 PHASE 1 EXPERIMENTAL BACKGROUND

4.2.1 Basis for Phase 1 Experimental Program

This Sub-section presents what exactly was examined in the Phase 1 experimental programme and discusses the reasons behind the experimental programme design. The primary basis for the Phase 1 experimental programme arose from the need to obtain information about the relative performance of SCCs in the short to medium term by comparing different SCCs under natural chloride migration conditions. The basis for the type of SCCs examined was derived from Ferrycarrig Bridge, ensuring that the concretes examined were representative of the SCCs actually being used in industry, and allowing assessment of the Ferrycarrig Bridge SCC options in the short

to medium term. I t is noted however, that other SCC options (i.e. OPC + PFA SCC) were also investigated in the Phase 1 experimental programme, as will be discussed below.

As discussed in detail in Chapter 2 Ferrycarrig Bridge underwent substantial repairs in 2007 with the National Roads Authority taking advantage of a research opportunity to gather information regarding the efficiency of five SCC durability options. The SCC options used to repair Ferrycarrig Bridge were chosen by the NRA in line with enhanced durability concrete options the NRA specified up to 2007, and the options which the bridge management authority would consider specifying on Irish marine bridges in the future (C Probe Ltd. 2007). A number of these SCC durability options form the focus of this PhD research and consequently, have formed the basis of the first phase of testing. To recap the five options utilised at Ferrycarrig Bridge are presented below:

- CEM I Ordinary Portland Cement (OPC), standard 50mm cover
- CEM I OPC with 70mm cover
- CEM I OPC with silane surface treatment, 50mm cover
- CEM I OPC with admixed corrosion inhibitors, 50mm cover
- CEM I OPC with Ground Granulated Blast-Furnace Slag (GGBS) as partial cement replacement (60% GGBS), 50mm cover

The Phase 1 experimental testing was solely concerned with the ability of the various SCC options to resist the ingress of chlorides in the initiation phase of reinforcement corrosion. This was achieved through examining the chloride transport properties of the SCC options in the form of C_s and D_{app} values. The SCC used in the repair of Ferrycarrig bridge were designed for use in the XS 3 exposure class specified in EN 206-1:2000 and are thus have a high resistance to chloride induced corrosion (Centre Europeen de Normalisation CEN 2000). From examination of the literature it was expected that the durations required to induce corrosion in embedded reinforcement in samples placed in the accelerated test chambers could be in excess of 2 years (Asrar et al. 1999; Mangat and Molloy 1991). In order to induce corrosion in a shorter laboratory duration other researchers have tested lower grade concretes with higher w/c ratios than those allowable for marine exposure in accordance with EN206-1:2000 (Dhir et al. 1994). This approach was not adopted herein as it was considered more useful to investigate the marine durability of concretes which could be used in new marine structures in accordance with existing standards. It was envisaged that two phases of experimental testing would be required to fully investigate the C_s and D_{app} characteristics of the SCC durability options utilised at Ferrycarrig Bridge. Given the lifetime of the PhD research it was thus judged that the study of corrosion in the test samples was outside the scope of the

experimental testing carried out in this PhD. The experimental testing herein instead focuses solely on obtaining C_s and D_{app} values for the various concretes examined. Consequently, no reinforcement was utilised in the slab or in the cube specimens tested. This timeframe restriction meant that the full analysis of the OPC + Corrosion Inhibitor falls outside the scope of the experimental testing. This SCC will however be examined in terms of C_s and D_{app} to investigate the findings of Luo and de Schutter (2008), whose results indicated that the use of corrosion inhibitors reduced resistance of concrete to chloride ingress. This will be discussed in detail later in this chapter.

In addition to the five SCC durability options utilised at Ferrycarrig Bridge the use of Pulverised Fuel Ash (PFA) as a partial cement replacement was also investigated in the Phase 1 testing program. PFA, also referred to as fly ash, is an artificial pozzolanic which is a by-product of power generation stations utilising pulverised coal. ASTM International defines pozzalana material as a siliceous, or siliceous and aluminous material, which in itself possess little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide (produced by hydrating Portland cement) in the presence of water to form compounds possessing cementitious properties (ASTM 2008). Thus, PFA is always used as a partial cement replacement and will be studied in this capacity in this thesis.

The durability advantages of PFA are widely published in the literature as discussed in Chapter 3 (Dhir et al. 1998; McPolin et al. 2005; Thomas and Matthews 2004; Yazici 2008). PFA is a by-product of coal fired power stations in both the Republic of Ireland and in Northern Ireland thus making it a viable option for utilisation as a partial replacement for OPC on Irish Bridges. It is noted however that only the PFA produced in Kilrute Power Station Co. Antrim is suitable for use as a major constituent in blended cements used in structural members. The local availability of PFA, together with the wealth of research indicating its suitability for use in marine environments, makes it an attractive durability option in an Irish context.

The number of parameters or concrete durability options which could be investigated in the Phase 1 experimental program was limited for the following three reasons; a) space limitations in the test equipment, b) an exposure duration of eight months was required as will be discussed later in this chapter and c) the process of sample analysis was very time consuming and costly. Based on the sample number limitations, and the points in favour of investigating PFA outlined above, the five durability options investigated in the first phase of testing are listed below, together with the abbreviations used to refer to each option throughout this chapter:

- CEM I Ordinary Portland Cement (OPC)
- OPC with silane surface treatment (OPC-S)
- OPC with admixed corrosion inhibitors (OPC-CI)
- OPC with 60% GGBS as partial cement replacement (GGBS)
- OPC with 30% PFA as partial cement replacement (PFA)

As will be discussed in detail in Section 4.4.3 full chloride profiles were obtained for each sample upon completion of the exposure period. This allowed the examination of the effects of varying the cover depth for any of the above SCC durability options, facilitating the examination of the OPC with increased cover option, utilised on crosshead 2 at Ferrycarrig Bridge in 2007.

To assess the durability of the SCC options listed above, in a marine environment, concrete specimens were exposed to a chloride rich environment in salt fog chambers for 36 weeks. After this period detailed sample analysis was carried out. In addition to testing the chloride ingress resistance capabilities of the alternative SCC options, the salt fog chamber experiments were also designed to investigate the effect of testing on different scales i.e. if identical concrete mixes were tested in two different models of chamber under an identical wetting and drying cycle program, would the relative performance of the different concretes be equal? This was deemed to be an interesting explorative investigation in the context of the difficulties cited in Chapter 2 with completely standardising such tests due to different exposure conditions occurring in different laboratories which use different test apparatus.

The effect of testing on different scales was achieved through the utilisation of a two different size salt fog chambers with test samples sized appropriately for the different chamber sizes. As will be discussed in detail in Sections 4.3.2 100mm cube samples were tested in the smaller salt fog chamber while 300 x 300 x 120mm slab samples were tested in the larger salt spray fog chamber. The two chambers were obtained from the same supplier and were programed identically. Consequently, it was expected that the results would highlight the effect of differences in chamber environment brought about by different chamber models, and the possible effects of the different specimen sizes. The Phase 1 tests were therefore designed to achieve one primary experimental aim, which addresses Research Question 1, and one subsidiary experimental aim as follows:

- Primary Experimental Aim (Research Question 1); Compare the chloride transport properties of a number of different SCC options through natural chloride migration laboratory testing which incorporated wetting and drying cycles

- Subsidiary Experimental Aim; Investigate the effect of carrying out salt fog chamber testing on two different scales

4.2.2 Brief Summary of Laboratory Testing Stages

Each aspect of the Phase 1 testing is discussed in detail in this chapter. It is however necessary to mention some aspects of the testing program before they are discussed in detail. In order to ensure ease of interpretation of the detailed discussion of the experimental work the main stages of the Phase 1 laboratory program are presented in brief below in bullet point format.

- Concrete slab samples 300mm x 300mm x 120mm and 100mm cube samples were poured and wet cured for 28 days in accordance with EN 12390-2:2009 and BS 1881-111:1983 (British Standards Institution 1983; Centre Europeen de Normalisation CEN 2009)

- 7 day and 28 day 100mm cube compressive strength tests were carried out for each mix in accordance with EN 12390-3:2009 (Centre Europeen de Normalisation CEN 2009)

- The slab samples and remaining cube samples were prepared for accelerated chloride ingress testing

- The prepared slab and cube samples were exposed to accelerated chloride ion ingress environments for 36 weeks in two salt spray chambers which were manufactured in accordance with EN 9227:2006 (Centre Europeen de Normalisation CEN 2006)

- After the 36 week exposure period the samples were removed from the chambers and oven dried for three weeks in accordance with the recommendations of NT Build 443 (Nordtest Method 1995)

- Dust samples were then collected at depth increments of 1-2mm for each sample using a profile grinder in accordance with NT Build 443 (Nordtest Method 1995)

- The dust samples were analysed using acid soluble potentiometric titration to ascertain the total chloride content at each depth increment, for each sample, in accordance with EN 14629:2007 (Centre Europeen de Normalisation CEN 2007)

4.2.3 Phase 1 Testing Apparatus

Having outlined the basis for the experimental program and the main laboratory testing stages it is now necessary to discuss the Phase 1 test apparatus' before the experimental work itself is presented. Two types of testing equipment were utilised in the Phase 1 test program. The first was the Toni Technik Toni-Trol testing rig which was used to carry out 100mm cube compressive strength tests at 7 and 28 days after pouring. This is a standard piece of equipment and thus is not discussed in detail herein. The second type of test equipment used in the Phase 1 testing program were the two salt fog chambers (SFCs) which were manufactured by Weiss Technik in accordance with the chamber specifications outlined in EN 9227: 2006 (Centre Europeen de Normalisation CEN 2006; Weiss Technik). As discussed in Chapter 2, studies utilising this piece of equipment are uncommon in the literature. Due to the relatively unique nature of the test apparatus it is discussed in some detail in this Sub-section.

As stated two salt fog chambers were used to conduct accelerated chloride ingress testing in the TCD materials laboratory. The smaller of the two chambers is the SC1000 which has a volume of $1.0m^3$. This is an open lid spray chamber and can be seen in Figure 4.1 with the Phase 1 test samples in position within the chamber. This open lid spay chamber is suited to the testing of smaller concrete samples up to a cube size of approximately 150mm. The larger SFC, the SKWT4300 is a walk in salt fog chamber with a volume of $4.3m^3$ and can be seen in Figure 4.2. This walk in salt fog chamber is suited to the testing of larger test samples.

Figure 4.1 SC1000 Open Lid Salt Fog Chamber

Figure 4.2 KWT4300 Walk in Salt Fog Chamber

The salt fog chambers subject the test samples to periodic wetting and drying cycles. During the wetting cycles a 5% (or 0.86M) NaCl solution fog fills the chamber. The SKWT4300 and the SC1000 were manufactured by Weiss Gallenkamp, a specialist company who design and build a variety of environmental test chambers. The purpose designed and built chambers use a sophisticated salt fog system to limit spatial variability within the chamber. The salt solution in the chambers is dispensed via nozzles which output salt solution in a fine mist or fog through the utilisation of compressed air in a Venturi siphon system. The mist or fog emitted from the chambers can be seen in Figure 4.3, which shows the SKWT 4300 fog chamber during a wet cycle, with the chamber door open. The nozzles utilised in the chamber can be seen in Figure 4.4 (a) & (b).

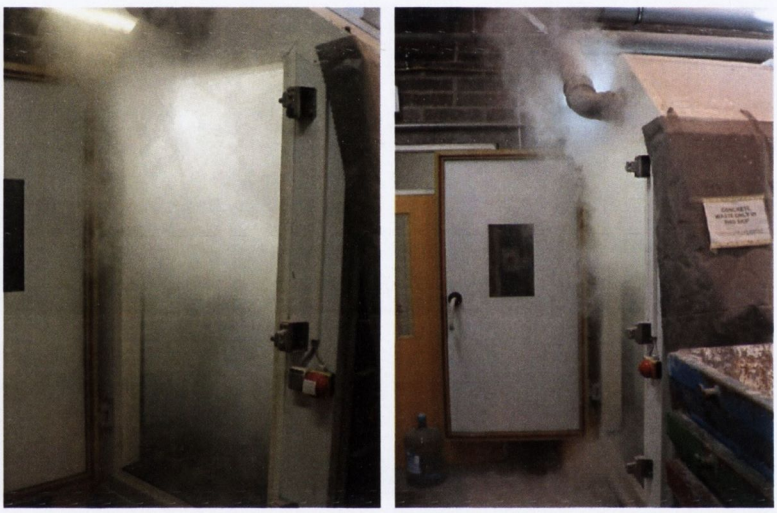
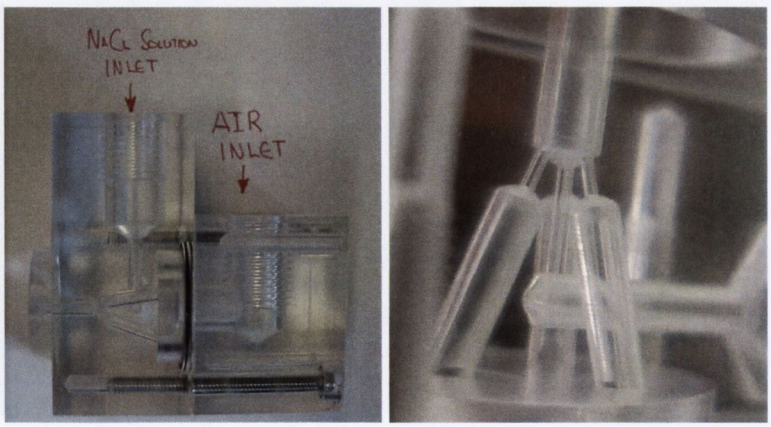


Figure 4.3 SKWT 4300 salt fog chamber with door open during wet cycle operation



(a) **(b)**

Figure 4.4 (a) & (b) Salt chamber spray nozzles utilising venturi siphon system

As can be seen from Figure 4.4 (a) the spray nozzle is fed by an air inlet and an NaCl inlet. In accordance with the Venturi principle, which is based on a combination of the continuity equation and Bernoulli's principle, the narrowing of the air inlet pipes, seen in Figure 4.4 (b), causes an increase in the air velocity for a given flow rate. According to the principle of conservation of energy this results in a drop in pressure. This drop in air pressure causes the NaCl to be sucked into the outlet tube in small particle form creating a fine mist or fog of NaCl solution which fills the chamber saturating the surface of the samples. The mobility of the mist results in a uniform level of salt solution exposure throughout the chamber. As discussed in Chapters 2, this salt fog chamber system represents a considerable improvement over a cruder salt spray chamber system where by salt solution is simply sprayed onto the samples via water pressure (Dhir et al. 1994; Li 2001; Mangat and Molloy 1995). The test results obtained from these cruder spray chambers are likely to be influenced by spatial variability due to uneven dispersion of salt solution droplets within the chamber. This could bring about misleading results, with some test samples performing better than others due to location in the spray chamber rather than actual material properties.

The dry cycles in the chambers are induced via an air exchange system. Upon commencement of the dry cycle the chamber is cleared of the salt fog. In the walk-in spray chamber this is done via a 75 watt air insertion fan which blows air into the chamber for the duration of the dry cycle. This fan inlet is at one end of the chamber with an air outlet hole at the other end of the chamber. In the small chamber the salt fog is cleared via a compressed air input and a small air outlet hole. This inlet and outlet air exhaust system means that there will be a certain amount of spatial variability within the chambers due to means of dry cycle provision. This spatial variability, together with measures taken to minimise the spatial variability effects, will be discussed further in Section 4.3.2 when the experimental test set-up is presented.

4.3 LABORATORY BASED EXPERIMENTAL WORK

This section details the work which was carried out in the laboratory as part of the Phase 1 testing. Firstly the sample preparation in the TCD concrete laboratory is discussed. Subsequently the testing set-up in the SFC is presented with the aid of photographs and drawings. Finally the analysis of the test samples after the 36 weeks accelerated exposure is presented.

4.3.1 Sample Preparation

As mentioned in Section 4.2.1 the SCC options being investigated in the Phase 1 testing are as follows:

- CEM I OPC (Ferrycarrig Bridge)
- CEM I OPC with silane surface Treatment: OPC-S (Ferrycarrig Bridge)
- CEM I OPC with admixed corrosion inhibitors: OPC-CI (Ferrycarrig Bridge)
- CEM I OPC with 60% GGBS as partial cement replacement: GGBS (Ferrycarrig Bridge)
- CEM I OPC with 30% PFA as partial cement replacement: PFA (Additional SCC durability option)

With the exception of the OPC-CI, three 300mm x 300mm x 120mm slabs and nine 100mm concrete cubes were prepared for each of the SCC options. As stated the testing carried out as part of this PhD thesis was solely concerned with the ability of the various durability options to resist the ingress of chlorides in the initiation phase of reinforcement corrosion. The corrosion inhibitor is not designed to resist the ingress of chlorides but rather designed to delay the onset of corrosion through the provision of additional protection to the embedded steel reinforcement (Neville 2011). Consequently, when examining total chloride content at various depths it would be expected that the OPC-CI concrete would perform similarly to the OPC concrete. A recent study has however indicated that the addition of corrosion inhibitors increases the rate of chloride ingress in concrete (Luo and Schutter 2008). Consequently it was necessary to investigate the effect of the corrosion inhibitor on the ingress of chlorides over the exposure period. The nature of the accelerated testing however, with the exclusion of reinforcement, meant that the OPC-CI durability option could not be fully investigated within the scope of the objectives of the work. The analysis of OPC-CI is considered to be only a partial examination of this durability option as it does not examine the ability of the corrosion inhibitor to delay the onset of corrosion in the manner in which the corrosion inhibitor was designed. Consequently the investigation into the OPC-CI did not form a key part of the experiment, but rather a verification of a proposed assumption. Thus it was considered sufficient to examine this durability option through cube samples only. This decision was influenced by the fact that there was only sufficient space available in the SKWT4300 chamber to test 12 slab samples at a time. This allowed four durability options to be investigated with three duplicate samples for each option. Five durability options with three duplicate samples were tested in the SC1000 through the exposure of 15 x 100mm cube samples.

In total 9 x 100mm cubes were poured for the OPC-CI durability option and nine cubes and 3 slabs were poured for each of the other SCC options considered. Three cubes for each mix were to be used for compressive strength tests at 7 day maturity and at 28 day maturity in accordance with BS EN 12390-3:2009 (Centre Europeen de Normalisation CEN 2009). The results of these cube tests are discussed in Section 4.4.1.

The SCC mix designs utilised for the four Ferrycarrig bridge durability options tested in the salt fog chamber were identical to the mix designs utilised on site in the repair of Ferrycarrig Bridge in 2007 with one exception. The mix designs utilised in the laboratory study had 25% more superplasticiser than those used at Ferrycarrig Bridge. The superplasticiser content utilised in the laboratory study was increased in order to achieve the same workability as was obtained on site during the repair of Ferrycarrig Bridge.

The mix design and concrete preparation for the Ferrycarrig Bridge repairs was completed by Roadstone Wexford Ltd. Discussions with Roadstone Wexford Ltd. revealed the slump flow of the OPC concrete used at Ferrycarrig Bridge was 670mm. The slump flow results for the other Ferrycarrig mixes were not available and the t_{500} times were not recorded in accordance with EN 12350-8:2010, although it is noted that this standard was not published at the time of the Ferrycarrig repair (Centre Europeen de Normalisation CEN 2010). It was stated by Roadstone that the suitability of this OPC mix at the time of the repair of Ferrycarrig Bridge was verified by Roadstone onsite by pouring trial beams. These trial beams consisted of a replica of a section of the crosshead beams with identical reinforcement patterns. One of the trial beams prior to the OPC concrete pour can be seen below in Figure 4.5.

Figure 4.5: Trial beam poured at Ferrycarrig Bridge to verify suitability of SCC mix design (ROD Resident Engineer 2009, Pers. Comm.)

In order to mimic the workability of the Ferrycarrig Bridge SCCs in the laboratory a number of trial pours were carried out and slump flow of 680mm was achieved in the laboratory by increasing the superplasticiser content used by 25% over that specified for the original Ferrycarrig mix design. The original mix design utilised 2.0L and 1.75L of superplasticiser per m^3 of OPC SCC and the GGBS SCC respectively. The increase in superplasticiser saw 2.5L and 2.2L used per m^3 of OPC and GGBS SCC respectively. This dosage volume is still well within the manufacturers recommended dosage for the Ferrycarrig mix designs which is between 1.0L and 5.0L per m^3 of concrete. The mix designs per m^3 of concrete for the SCC durability options investigated in the Phase 1 testing are presented in Table 4.1.

Table 4.1 Mix design details per m^3 of concrete

Mix Type	OPC (kg)	GGBS (kg)	PFA (kg)	Total Binder (kg)	Water (Ltrs)	w/b ratio (kg)	10mm Agg.[2] (kg)	Sand (kg)	Plast.[3] (kg)	S. Plast.[4] (kg)	C.I. (Ltrs)
OPC	500	_	_	500	220	0.44	500	1140	2.7	2.5	_
OPC – Silane	500	_	_	500	220	0.44	500	1140	2.7	2.5	_
OPC + C.I.[1]	500	_	_	500	220	0.44	500	1140	2.7	2.5	15
OPC + GGBS	200	300	_	500	220	0.44	500	1140	2.7	2.2	_
OPC + PFA	350	_	150	500	220	0.44	500	1140	2.7	2.5	_

1, C.I.; corrosion inhibitor; 2, Agg, aggregate; 3, Plast, plastisiser; 4, S. Plast, superplasticiser

As can be seen from the table the water/binder ratio for the mixes was 0.44 in accordance with the XS3 exposure class in EN206-1:200 (Centre Europeen de Normalisation CEN 2000). It can also be noted from Table 4.1 that in this first phase of testing the PFA was used as a direct replacement for OPC. EN 206-1:2000 specifies that when using PFA as a partial cement replacement in reinforced concrete structural elements an efficiency factor termed the *k*-value must be applied. No such *k*-value exists for GGBS to date, although this may change in the next revision of EN 206-1 (Sanjuan et al. 2011). The PFA *k*-value concept specifies that, only 40% ($k = 0.4$) of the PFA content used as a partial replacement for OPC CEM I 42.5 strength class and higher in a structural element, can be used in the calculation of the water/binder ratio. The *k*-value for the 32.5 strength class is 0.2. The *k*-value concept also applies to the minimum cement content. The code also states that the maximum PFA:OPC ratio for the *k*-value concept is 0.33. PFA content above this ratio cannot be used in the water/(cement +*k* x PFA) calculation. The values for the *k*-value parameter were primarily estimated based on compressive strength characteristics (Bentur and Mitchell 2008; Papadakis and Tsimas 2002; Sanjuan et al. 2011).

As will be discussed in the Chapter 5, the Phase 2 experimental testing will investigate the effect of the PFA *k*-value concept on marine durability of SCC. The OPC + PFA SCC was studied in the first phase of testing without the implementation of the *k*-value to examine the diffusion characteristics of the PFA concrete with the same volume of binder and the same water/binder ratio as the other mixes. This approach has been taken by a substantial number of researchers examining the durability characteristics of PFA concretes (Bentz et al. 1996; Elahi et al. 2010; Li 2001; Loser et al. 2010; Mangat and Gurusamy 1987; McPolin et al. 2005; Stanish and Thomas 2003). Some researchers have examined PFA concretes using a greater proportion of PFA for a given weight of OPC replacement meaning a form of binder efficiency factor was employed (Dhir et al. 1993; Dhir et al. 1994). PFA with and with the *k*-value efficiency factor specified in Section 5.2.5.2 of EN 206-1:2000 was however examined in the Phase 2 testing programme as will be discussed in Chapter 5 (Centre Europeen de Normalisation CEN 2000).

As Seen in Table 4.1, the fine aggregate proportion of the total aggregate volume used in the Ferrycarrig SCC mixes is quite high at 69%. As will be discussed in detail in Chapter 5 in the context of the SCC workability study, a lower fine aggregate portion of approximately 50% is most common in the literature. However as stated in the EFNARC guidelines a variety of different mix design approaches are adopted to achieve SCC (EFNARC et al. 2005). Many researchers existing literature have employed fine aggregate proportions of between 60% and 72% of the total aggregate (Corinaldesi and Moriconi 2004; Ouchi et al. 2003; Sahmaran et al. 2009; Yu et al. 2010)

For both the Phase 1 and Phase 2 experimental programmes every effort was made to utilise the same materials in the laboratory investigation as were used in the repair of Ferrycarrig Bridge in 2007. Communications with Roadstone Wexford revealed that the coarse aggregate used at Ferrycarrig Bridge was 10mm crushed limestone while the fine aggregate type used was a well graded sand. These material types were mimicked in the laboratory study. The product names and manufacturers of the plasticiser, super-plasticiser and corrosion inhibitor used in the repair of Ferrycarrig Bridge were supplied by Roughan O'Donovan Consulting Engineers. These products are listed below:

- Plasticiser: Pozzolith 320N manufactured by the BASF chemical company
- Super-plasticiser: Glenium 51 manufactured by the BASF chemical company
- Corrosion inhibitor: Ferrogard 901 manufactured by the Sika Group

Each of these concrete mix additives was obtained from the manufacturers listed above.

A chemical constituent analysis was carried out on the cementitious materials utilised in the preparation of the test samples, on behalf of TCD, at the Earth Sciences Department of St Andrews University, Scotland using x-ray fluorescence (XRF) technology. The results of this analysis are presented below in Table 4.2

Table 4.2 Chemical composition (%) of the cementitious materials

	CaO	SiO_2	Al_2O_3	MgO	Fe_2O_3	SO_3	LoI
OPC	60	24	6	2	3	3	1
PFA	3	58	24	1	6	1	3
GGBS	40	37	12	7	1	2	0

The XRF analysis indicates that the PFA utilised in the experimental program is siliceous fly ash falling into the Loss of Ignition (LOI) category of 0-5% (Centre Europeen de Normalisation CEN 2000). The PFA utilised in the laboratory study was tested by Kirton Concrete Testing on behalf of Conexpo who managed the PFA quality control for Kilrute Power Station at the time of testing. The PFA has been tested in accordance with EN 450-2:2005 to ensure compliance with EN 450-1:2005 (Centre Europeen de Normalisation CEN 2007; Centre Europeen de Normalisation CEN 2007). The GGBS used in the Phase 1 testing was supplied by Ecocem Ireland Ltd. The GGBS produced by Ecocem Ireland Ltd. is manufactured in accordance with EN 15167-1: 2006, and is approved for use in the manufacture of concrete and concrete products by EN 206–1: 2000 (Centre Europeen de Normalisation CEN 2000; Centre Europeen de Normalisation CEN 2006). The OPC used in the Phase 1 testing was CEM I OPC cement supplied by the Quinn Group Ltd. The Quinn Group Portland Cement is manufactured to comply with the requirements of BS EN 197-1:2000 type CEM I Portland cement strength class 42.5/52.5N and is approved for use in the manufacture of concrete and concrete products by EN 206–1:2000 (Centre Europeen de Normalisation CEN 2000; Centre Europeen de Normalisation CEN 2000).

To reduce experimental error when working with fresh concrete a pouring and testing procedure was developed through consultation with the TCD technical staff, BS1881-125:1986 and other relevant fresh concrete testing standards. This mixing procedure, described in Appendix A, was then strictly adhered for each mix. The results of slump flow tests and t_{500} times for the Phase 1 pours are presented in Table 4.3. As can be seen from the table the OPC SCC and OPC-CI SCC fall into the SF2 category outlined in EN 206-9:2010, while the PFA and GGBS SCC fall into the SF3 slump flow category (Centre Europeen de Normalisation CEN 2010). The t_{500} times for all the Phase 1 concretes fall into the VS2 category specified in EN 206-9:2010.

Table 4.3 Workability Results for Phase 1 Mixes

SCC options	Slump Flow (mm)	T_{500} Time (sec)
OPC	680	2.1
OPC-C.I.	710	2.6
GGBS	790	2.8
PFA	760	3.5
OPC-S	660	2.6
EN 206-9:2010 Limits	SF1: 550-650 SF2: 660-750 SF3: 760-850	VS1 $\leq$ 2.0 VS2 $\geq$ 2.0

No segregation of the concrete mixes was identified during the slump flow tests. This is illustrated by Figure 4.6 which shows a close-up of the OPC concrete after the slump flow test. As can be seen from the figure the coarse aggregate is evenly dispersed throughout the concrete.

Figure 4.6: Slump flow test concrete illustrating good cohesion in the mix

It should be noted that the focus of the experimental study was not on the improvement of SCC mix designs or on the workability of the concrete. There was little scope for alterations to the SCC mix designs given that a four of the SCC mix designs investigated mimicked the SCCs used at Ferrycarrig Bridge in 2007. The applicability of these Ferrycarrig SCC mix designs was established on site and thus the SCCs have been verified as a suitable SCC mix design for bridge applications. Consequently, detailed investigation into the fresh concrete characteristics was not carried out in the first phase of testing. However a detailed workability study was carried out in the second phase of testing to facilitate classification of the SCCs investigated herein in each of the categories

specified in EN 206-9:2010, which was published after the Phase 1 concrete pour (Centre Europeen de Normalisation CEN 2010). The details of this workability study are provided in Chapter 5.

The cubes and slab samples to be tested in the salt fog chambers were wet cured until 28 days maturity in a curing tank at 20 ± 2°C in accordance with the relevant standard up to the date of testing (British Standards Institution 1983; Centre Europeen de Normalisation CEN 2009). The hydration of concrete, particularly concrete containing SCMs such as PFA and GGBS, is a relatively slow process. The Phase 1 testing commenced when the samples were approximately seven weeks old and lasted for 36 weeks. Thus, as with the majority of experimental programs, the concrete was relatively young during the testing period. The 28 day wet curing period was selected to promote the maximum level of hydration in the specimens prior to testing and minimise the effect of a lack of curing in young concrete. This approach of wet curing or curing in a fog room at relative humidity levels > 90% up to 28 days maturity has adopted by a number of researchers studying OPC and blended cements (Andrade et al. 2011; Audenaert et al. 2005; Audenaert et al. 2010; Hong and Hooton 1999; Li 2001; Mohammed et al. 2002; Yu et al. 2010). Other researchers have adopted a combination of wet curing for a number of days followed by curing at ambient laboratory conditions up to 28 days maturity stating that this better reflects on-site conditions (Wang et al. 2011).

The effect of the curing regime adopted has been investigated by researchers with varying results. Thomas and Matthews (2004) conducted a 10 year study examining OPC and PFA concretes and found that the curing regime utilised before exposure did not affect the durability performance of the concretes considered. Guneyisi et al (2005) however found, in short term laboratory tests, that the nature of curing adopted did effect the durability performance of the concretes considered. In a study of SCC containing 20% PFA Zhao et al. (2012) found that the duration of wet curing did not affect the migration coefficient of the concrete samples considered with the exception of one unusual result obtained for 7 day wet curing. It is recognised that the nature of curing adopted may affect the relative performance of the durability options considered herein, however the study of the curing effect is outside the scope of the testing described in this thesis.

After the curing period two coats of Sikaguard 63N sealant paint, manufactured by the Sika group, were applied according to the manufacturer's. The sealant paint was applied to all but one face of each sample to rule out edge effects. This meant that chlorides could only ingress into the test samples from one face. If chlorides were allowed to ingress from all faces the size of the test sample would have greatly influenced the results i.e. the sample results would not be representative

of the behaviour of a full sized beam exposed to an aggressive chloride rich environment. The slab samples can be seen during the Sikaguard application process in Figure 4.7.

Figure 4.7: Application of Sikaguard sealant paint to slab samples in preparation for testing

The final step in the sample preparation was to coat the OPC-S test samples with the silane treatment. The Silane product used was identical to that used during the repair of Ferrycarrig Bridge (Protectosil BHN, manufactured by Evonik Industries). The silane was applied to the face of the sample which was to be exposed during salt spray chamber exposure using an airless spray gun in accordance with the manufacturer's guidelines (Evonik Industries 2011).

4.3.2 Spray Chamber Test Set-Up

The wetting and drying cycles for the two salt fog chambers were pre-programed for the 36 week exposure period prior to the commencement of testing using the on-board computers and the Data link operating system. The testing duration was developed based on preliminary studies which indicated that this duration was necessary to ensure penetration of chlorides to a sufficient depth to allow for the fitting of full chloride profiles for each sample. Both chambers were programmed identically on seven day cycles as follows:

- 24 hours continuous salt fog spraying
- Followed by 48 hours drying time
- Then 24 hours continuous salt fog spraying
- Followed by a 72 hours drying period

Both of the salt fog chambers were monitored during the exposure period using temperature and humidity probes. A wireless probe system comprising of a Testo 635-2 data logger with two

wireless combined temperature and humidity probes were used due to the highly corrosive nature of the chamber environment.

Despite the venturi spray system, discussed in Section 4.2.3, it was expected that there would still be some level of spatial variability within the chamber. The primary source of spatial variability within the chamber was envisaged to be caused by the drying process. For the SKWT 4300 chamber the drying inlet for the blow through fan is located at the inner end of the chamber and the outlet for the blow through fan is located at the door end of the large chamber as can be seen in Figure 4.8. Samples which are located closer to the fan inlet will experience quicker drying than samples at the fan inlet end of the chamber closest to the door. This suspicion was confirmed from observation during testing with the samples at the outlet end of the chamber appearing surface dry approximately 30 minutes sooner than the samples at the fan outlet end of the chamber.

In order to minimise the spatial variability effects within the chamber statistical blocking was utilised. Figure 4.8 shows the sample set up with statistical blocks labelled within the SKWT 4300 chamber. Statistical experimental design dictates that unwanted known effect should be minimised through statistical blocking samples, while unwanted unknown effects should be minimised through randomising sample location. The three slab samples for each SCC durability option represent a statistical block as can be seen from Figure 4.8. The effect of spatial variability, which could neither be quantified nor eliminated, was targeted by randomising the original block order and then rotating the block order periodically so that each statistical block was located at each location within the chamber for an equal time during the exposure period. This practice was also employed by Leung et al. (2008) in an experimental programme which utilised a salt spray chamber. Figure 4.8 illustrates the starting test set-up, with the four block positions which were rotated periodically.

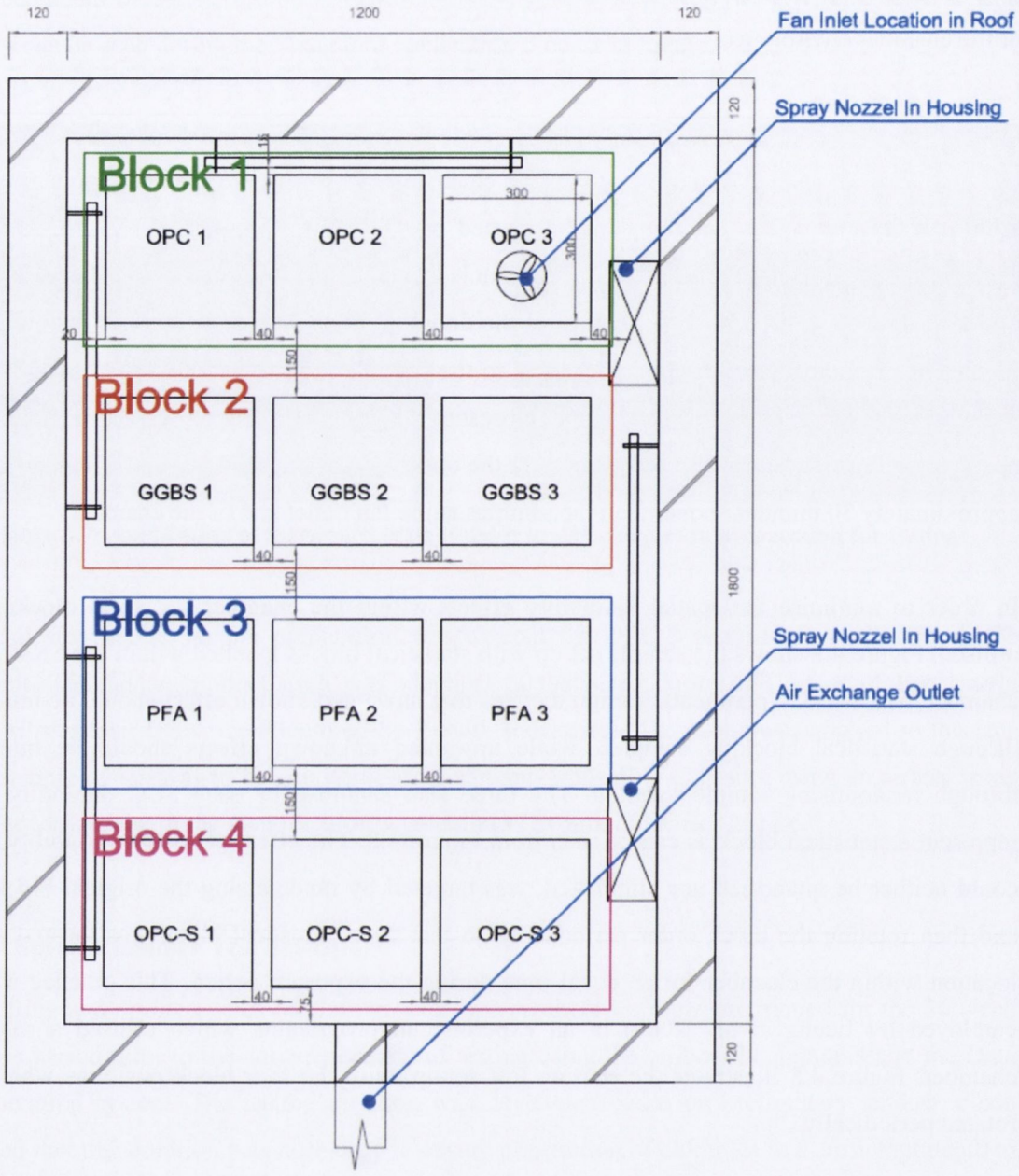


Figure 4.8: Sample layout for SKWT salt fog chamber

The same statistical blocking system was adopted for the cube specimens which underwent exposure in the SC1000 SFC. It must be noted the air exchange system in the SC1000 is not as powerful as the fan based air exchange system in the SKWT 4300 chamber. The SC1000 is not fitted with a blow through fan. Instead the exhaust air is supplied by a 20mm diameter PVC pipe which supplies compressed air. This results in less powerful drying in the SC1000 when compared to the SKWT4300. The effects of the reduction in drying power in the SC1000 chamber will be discussed in detail in the context of the overall results in Section 4.4.

4.3.3 Sample Analysis

Concrete Dust Sampling

Upon completion of the exposure period the samples were removed from the chamber and dried in an oven at 105°C to a constant mass, defined as < 1% weight loss in a 24 hour period (Centre Europeen de Normalisation CEN 2007), in order to ensure compliance with the NT Build 443 analysis procedure (Nordtest Method 1995).

As stated in Section 4.2.1 it was planned to examine the durability characteristics of each test specimen through obtaining C_s and D_{app} values. In order to do this full chloride profiles for each of the concrete specimens had to be obtained. This was a time consuming process which started with the gathering of dust samples from each of the test specimens. The dust samples were collected using the Profile Grinder PF 1101 with a diamond tip grinding bit supplied by Germann Instruments Inc. This apparatus operated by rotating the grinding bit within a 73mm confining ring which is centred on the sample and clamped in position. Figure 4.9 shows the profile grinder pictured with a slab sample and cube samples before and after the grinding process. Although, this process is slow, the apparatus ensures a very accurate sampling procedure. The purpose build grinding unit allows dust to be collected to a depth accuracy of ± 2% (Germann Instruments Inc. 2000). In addition a dust sample obtained from a 73mm profile ground hole is not subject to the errors and inaccuracies associated with collecting dust from relatively small diameter holes. This drilling practice, although quicker, can result in dust collected at a given depth comprising almost entirely of aggregate, especially when the drill bit diameter is comparable to or smaller than the maximum aggregate size (McPolin et al. 2005). This would result in significantly reduced chloride content in the given dust sample. This influence of the aggregate in drilled sample process leads to more variation from sample to sample making it more difficult to detect systematic changes due to actual material properties.

The profile grinding for the Phase 1 test samples was carried out in accordance with NT Build 443 (Nordtest Method 1995). The standard states that at least eight layers be ground off with layer thickness adjusted to ensure that a minimum of 6 points are obtained from sample surface to chloride content of C_i + 0.03 mass %, where C_i is the initial chloride content in the sample discounting the first 1mm. Based on the existing literature it was predicted that the GGBS SCC specimens would result in the shallowest depth of chloride ingress (Loser et al. 2010; McPolin et al. 2005; Mohammed et al. 2002). Consequently, for the Phase 1 testing the GGBS specimens were profile ground at 1mm depth increments to ensure at least 6 points would be obtained per profile while dust was collected for the remainder of the specimens at 2mm depth increments. These depth

increments resulted in a minimum of 9 plotable points for the slab specimens and 10 plotable points for the cube samples complying with the NT Build 443 guidelines.

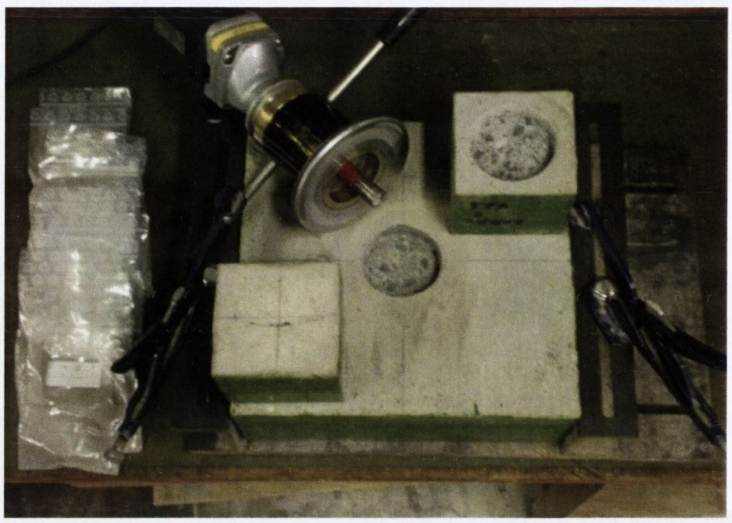


Figure 4.9: Profile grinder, ground samples and dust samples in sealed bags

Once the dust samples were collected acid soluble potentiometric titration analysis was used to find the total chloride content at each depth increment for each sample. This procedure extracts both bound and free chlorides from the concrete dust sample to give the total chloride content. It is recognised that the free chloride content has a larger influence on the initiation of corrosion, however it is acknowledged in the literature that interaction between chloride and cement based material is not clearly understood (ChlorTest 2005). In the context of blended cements it has been found by some researchers that blended cements have a greater binder capacity than OPC cements (Dhir et al. 1994). It has however also been found that the addition of blended cements reduces the quantity of chlorides bound in the cement matrix (Mohammed et al. 2002). Some of the confusion may be due to the fact that free chlorides can be released by carbonation or sulphate attack making them available in the pore solution to cause reinforcement corrosion (Neville 2011).

The complicated nature surrounding chloride binding and the resultant lack of understanding has lead researchers to conclude that the most appropriate representation of the critical chloride content C_{cr} appears to be in terms of total chloride content (DuraCrete 1998). The experimental examination of bound chlorides was considered outside the scope of this thesis, however this area will be discussed further in Chapter 6 in the context of the selection of appropriate C_{cr} values for the different concrete types.

In order to obtain the acid soluble total chloride content at each depth increment the total chlorides had to be extracted from the concrete dust sample. A procedure was developed for this process

based on EN14629:2007 and the work of Kenshel (2009). This procedure is presented in Appendix A. Upon completion of the total chloride extraction each sample was analysed using the Metrohm 785 DMP Titrino device with the Metrohm 760 sample changer. This device preforms potentiometric titration using a silver nitrate solution. A 0.02Mol/l silver nitrate solution was used to increase the sensitivity of the analysis and to ensure that total chloride content values were obtained at low chloride contents.

The total chloride content was thus obtained for each concrete sample at 1-2mm depth intervals up to a maximum depth of 34mm. The analysis of the 27 concrete specimens was a time intensive process with each titration taking approximately 30 minutes. In total over 300 titrations were carried out representing over 3.5 months of laboratory work. The accuracy and precision of the totration analysis system throughout the titration analysis was ensured by robust procedure described in Appendix A.

4.4 PHASE 1 TEST RESULTS

4.4.1 Compressive Strength Tests

Compressive strength testing was carried out at 7 days and 28 days on 100mm concrete cube samples for each of the SCC mixes to assess the concrete strength characteristics in accordance with EN 12390-3:2009 (Centre Europeen de Normalisation CEN 2009). As previously stated three cubes were tested at 7 days and three cubes were tested at 28 days for each mix. The compressive strength test results and standard deviations for the five SCC mixes are shown in Table 4.4.

Table 4.4 COMPRESSIVE STRENGTHS OF SCC AT 7 AND 28 DAYS

SCC Mix	7 day strength (N/mm^2)	7 day std. dev. (N/mm^2)	28 day strength (N/mm^2)	28 day std. dev. (N/mm^2)	t-test lower bound vale (28 Days)
OPC	57.0	0.4	68.9	0.5	67.4
OPC-S	58.1	1.0	70.7	1.9	65.1
OPC-CI	54.2	0.2	65.9	0.3	65.0
GGBS	46.7	0.7	67.0	0.4	64.4
PFA	39.9	0.2	57.5	0.3	56.6

The 28 day characteristic strength for the Ferrycarrig Bridge SCC mix designs was $50N/mm^2$. As can be seen from Table 4.4 all of mean strengths for SCCs mixed in the laboratory exceed this 28

characteristic strength. A mean based on three samples however requires further inspection. If a one tailed t-test is performed using the compressive strength sample data to investigate if the sample mean 28 day strengths are statistically significantly larger than 50N/mm^2 with 95% confidence, the critical t-value would be 2.92. The value of the μ - 2.92σ where μ is the sample mean and σ is the sample standard deviation is shown in the last column in Table 4.4. As can be seen from the table, based on the mean and standard deviations obtained from the compressive cube testing all the concrete mixes have a mean strength which is statistically significantly greater than 50N/mm^2.

It is notable from Table 4.4 that the PFA mix has a lower strength than the other concrete mixes. At 7 days the PFA mean compressive strength is 15% lower than the OPC + GGBS mix and 30% lower than the OPC mix. At 28 days the PFA has gained strength in comparison to the OPC concrete however it is still 16.5% lower than the OPC. The gap between GGBS remains constant at 14%. This pattern of slower strength gain in concretes containing SCMs has been identified in the published literature. Guneyisi et al. (2005) found that concretes containing blended cements had a lower 28 day strength than OPC concretes however it was also found that with increasing age this pattern was reversed. Similarly Bai et al. (2002) found that OPC concretes had a greater 28 day strength than the concretes containing 30% PFA as a direct replacement, however, with continued hydration it was found that the compressive strength for the concrete containing 30% PFA surpassed the OPC concrete. Compressive strength tests were not carried out in the Phase 1 testing beyond 28 days. The Phase 2 testing did see there materials tested at 7, 28 and 56 days. As discussed in Chapter 2 the *k*-value concept is introduced in EN 206-1:2000 primarily to compensate for the reduced binder capacity of PFA, particularly at early ages (Bentur and Mitchell 2008; Papadakis and Tsimas 2002; Sanjuan et al. 2011). The *k*-value concept was not utilised in this first phase of testing due to the fact that the experimental testing was focused on the ability of concretes with equal w/b ratios, to resist the ingress of chlorides, and not on strength. Concrete containing PFA with and without the implementation of the *k*-value efficiency factor concept was however investigated in the Phase 2 testing, as will be discussed in Chapter 5.

4.4.2 Temperature and Humidity Data

Temperature and humidity probes were placed in both the SKWT4300 chamber and the SC1000 chamber allowing the environmental conditions to be recorded. As previously stated both chambers were programed identically on seven day cycles. Recording the temperature and humidity however presented an opportunity to quantify what effect, if any, the chamber type had on environmental conditions.

The average temperature in the chambers, which was controlled by the chambers' electric heaters, was almost identical. The SC1000 chamber had an average temperature over the monitoring period of 22.2°C while the SKWT4300 had an average recorded temperature of 22.8°C. The Relative Humidity (RH) monitoring however highlighted a stark difference in the conditions in the two chambers. Figure 4.10 shows the relative humidity conditions in the two salt spray chambers over a seven and a half day monitoring period. As can be seen from the plot the relative humidity conditions in the two chambers differ significantly. The SC1000 chamber, containing the cube samples, experienced higher humidity levels for far longer than the SKWT4300 chamber which contained the slab samples.

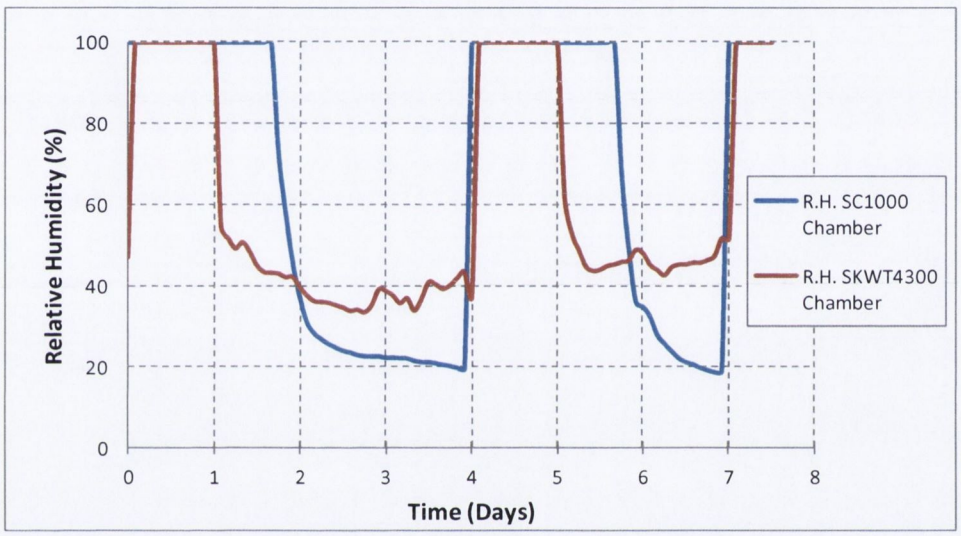
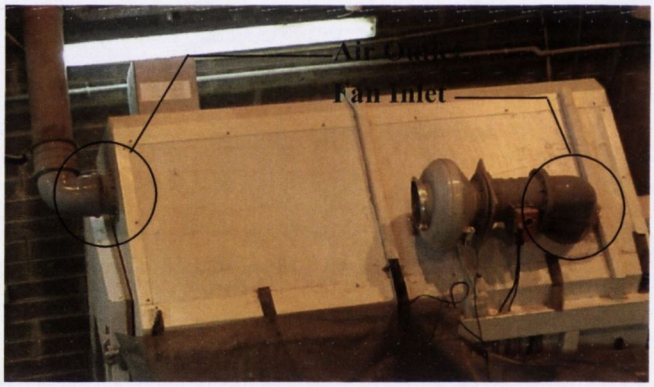


Figure 4.10 Relative humidity data from both chambers for a one week sample period

The difference in RH conditions was due to the different air exchange systems used in the two salt fog chambers. As previously mentioned in Section 4.3.2 the air exchange system in the SKWT4300 is supplied by a 75 watt blow through fan, shown Figure 4.11. The fan can also be seen mounted in the chamber in Figure 4.12(a) and Figure 4.12 (b). The fan inlet can be seen on the roof of the SKWT 4300 chamber on the right of Figure 4.12 (b) with pipe serving as the air outlet or exhaust vent in the chamber shown on the left of Figure 4.12 (b).

Figure 4.11 75 watt blow through fan used which ventilates the SKWT4300 salt fog chamber

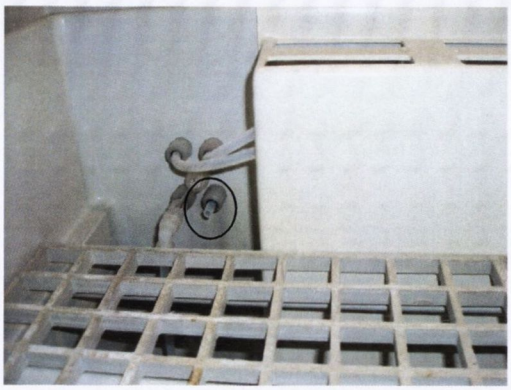
(a)



(b)

Figure 4.12 Air exchange system inlet and outlet for SKWT4300 salt fog chamber

In the SC1000 chamber the exhaust air system was supplied by the compressed air line. The air inlet line and outlet vent can be seen below in Figure 4.13 (a), (b) and (c). The inlet air pipe is circled in black in the figures, the air outlet or exhaust vent is circled in red in Figure 4.13 (b). Figure 4.13 (c) shows the location of the air exchange system in the SC1000 spray chamber circled in blue.



(a)

(b)

(c)

Figure 4.13 SC1000 spray chamber air exchange system

It is clear from the photographs and from Figure 4.10 that the SKWT4300 chamber has a more powerful air exchange system than the SC1000 chamber. This results in the humidity levels in the chamber reaching a steady 'dry' state of approximately 55% relative humidity after just 2 hours while the SC1000 takes approximately 19 hours to reach a relative humidity of 55%. This can be seen in Figure 4.14 below which shows the relative humidity in both chambers together with the programmed relative humidity i.e. 100% relative humidity during spraying hours only. The points at which both chambers reach a relative humidity of 55% are highlighted in the plot by the dashed grey lines. It can be noted that the baseline 'dry' relative humidity in the SC1000 is lower than the baseline 'dry' relative humidity in the SKWT4300. This is due to the fact that the exhaust air intake for the SKWT4300 comes from the laboratory while the exhaust air intake for the SC1000 is supplied by a compressor. The compression of the air causes vapour in the air to drop out in the form of condensate resulting in a lower relative humidity in the exhaust air-line in.

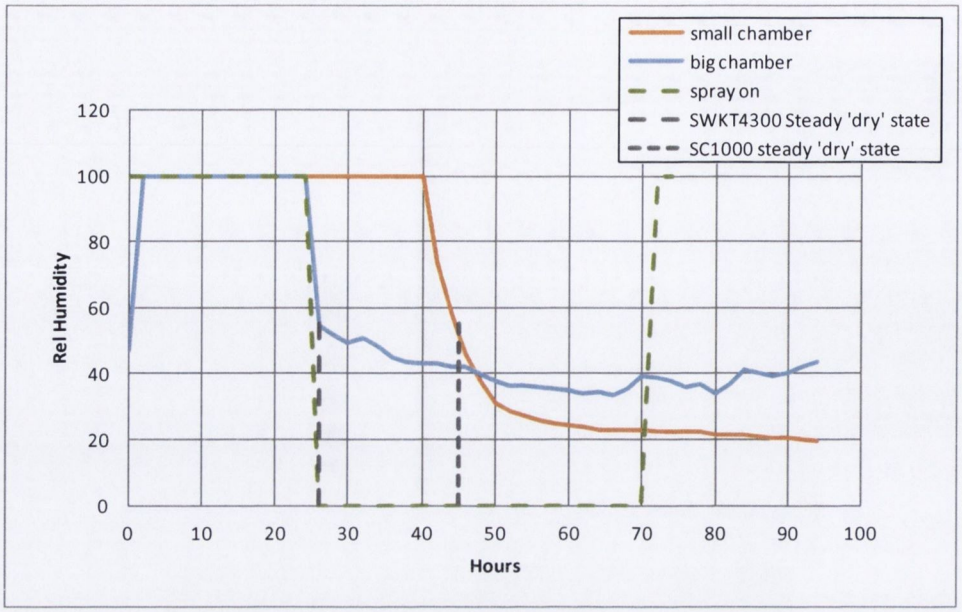


Figure 4.14 Effect of Humidity on Drying

The average RH of the SKWT4300 chamber was 56.0% while the average RH in the smaller SC1000 chamber was 62.9%. From Figure 4.14 it can be noted that the duration of the drying periods in the SKWT chamber were far longer. Consequently it can be said that the slab sample located in the SWT4300 experienced more drying than the cube samples in the SC1000 chamber. This is an interesting finding in the context of the subsidiary Phase 1 experimental aim which seeks to investigate the effects of testing on different scales. The effect of the difference in environmental conditions brought about by testing on different scales on the experimental results will be discussed in detail in Section 4.4.7.

4.4.3 Chloride Profile Results

Having obtained values for total chloride content at 1-2mm depth increments for each sample using potentiometric titration, chloride profiles for each of the SCC specimens were developed. An example of one of the chloride profiles for a PFA slab specimen can be seen below in Figure 4.15.

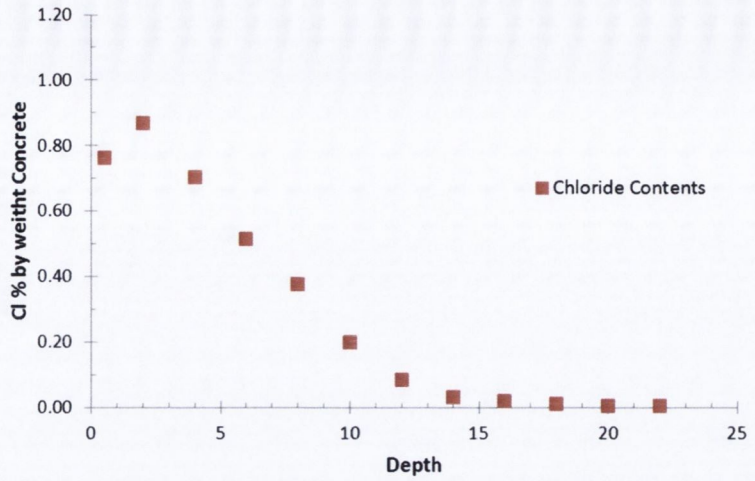


Figure 4.15 PFA slab specimen chloride profile showing total chloride content versus depth

The chloride profile shows a clear reduction in total chloride content with distance from the exposed face as one would expect from externally sourced chlorides. Values of C_s and D_{app} were obtained for each concrete sample by fitting Fick's second law of diffusion with Crank's error solution to the chloride profiles using multi variable non-linear regression analysis. This practice is common place in the literature for laboratory experiments, (McPolin et al. 2005; Nokken et al. 2006) marine exposure sites, (Thomas and Bamforth 1999; Thomas and Matthews 2004) and for real structures where chloride contents are obtained from dust collected from drill sampling or concrete cores (Costa and Appleton 1999; Kenshel 2009; Morcous et al. 2010; Salta et al. 2011). Fick's second law of diffusion with Crank's error solution is shown below as Equation 4.1.

$$C(x,t) = C_s\left[1 - \text{erf}\left(\frac{x}{2\sqrt{tD_{app}}}\right)\right] \qquad \textbf{Equation 4.1}$$

where C(x,t) is the percentage chloride content at depth x and time t, C_s is the surface chloride content, erf is the error function, x is the depth below the concrete surface being considered, D_{app} is the apparent diffusion coefficient and t is time.

The PFA slab specimen chloride profile presented in Figure 4.15 is shown again in Figure 4.16 with Equation 4.1 fitted to the total chloride content values. The regression variables utilised in the multi variable non-linear regression analysis were C_s and D_{app}.

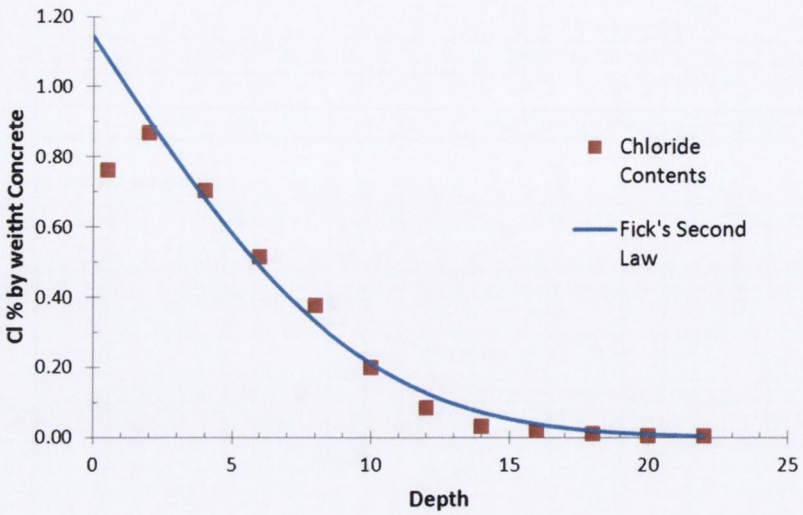


Figure 4.16 PFA slab specimen chloride profile with Fick's second law fitted using regression analysis

As can be seen from Figure 4.16 the fit of Fick's second law with Crank's error solution is very good for the PFA slab with an R^2 value of 0.991 obtained from the regression analysis. The fit of the experimental data to Fick's second law with Crank's error solution was very good across all the samples. R^2 values ranging from 0.984 to 0.998 were obtained for the slab samples and the R^2 values from 0.942 to 0.998 were obtained for the cube samples.

It must be noted that many of the chloride profiles obtained from the experimental testing showed slightly lowered chloride content in the outermost concrete layers with the peak chloride content appearing slightly deeper in the samples. This phenomenon, known as the 'skin effect', is recognised in the literature (Andrade et al. 1997). Andrade et al. stated that concrete skin has a different matrix composition to the internal concrete due to the effects of contact with moulds and segregation of aggregates. The reduction in chlorides close to the surface can also be significantly reduced due to wash-out, caused by the removal of chlorides by rain or by cooling water during core sample extraction. This washout effect tends to penetrate further into the concrete than the skin effect. Washout is however not applicable herein due to the fact that testing was carried out in an environmental chamber and specimen sampling was conducted using a dry grinding method.

Figure 4.16 is typical of many of the chloride profiles obtained from the Phase 1 test specimens with the second point in the chloride profile showing the maximum chloride content. Some of the Phase 1 chloride profiles however displayed very high chloride concentrations for the first titration reading. It is stated by Andrade et al. (1997) that the skin effect can also result in an anomalously high chloride content right at the concrete surface. Consequently the first total chloride content

reading, which is at the depth range 0-1mm, was not considered in the chloride profile multi variable non-linear regression curve fitting process. For one of the 27 samples, PFA-Cube-3, the third shallowest total chloride reading at 3-5mm represented the greatest chloride content value in the profile. It would thus appear that the skin effect has penetrated further for this sample. Consequently, for this profile the 3-5 mm titration result is the first point used in the regression analysis.

The next section will present the cube sample results in three different forms. Firstly the C_s and D_{app} results will be assessed using dot plots which facilitate an interpretation of the degree of scatter of the data. The average chloride profiles will then be presented facilitating a graphical analysis of the relative performance of the five durability options examined in terms of the 100mm cube samples. Finally the relative performance of the SCC durability options will be assessed utilising quantitative figures, by comparing the predicted time to initiation of corrosion for the various durability options utilising an illustrative example. Section 4.4.5 will present the slab specimen results in an identical format. Subsequently, having analysed both sets of results separately, the slab and cube results will be compared in Section 4.4.6. Finally both slab and cube results will be compared to the results from the existing literature in Section 4.4.7. This comparison will be based on relative performances or relative merits values in order to facilitate the broadest possible comparison, taking in data from various exposure conditions and testing methods.

4.4.4 Cube Specimen C_s, D_{app}, Chloride Profiles and Relative Merits

Figure 4.17 presents the C_s results for the cube samples. As previously stated there are three duplicate cube specimens for each durability option investigated. The three C_s values for each of the five durability options are presented in Figure 4.17. The individual C_s values are shown in the figure as solid red points, the sample mean value for each SCC durability option is shown as a blue circle with cross-hairs.

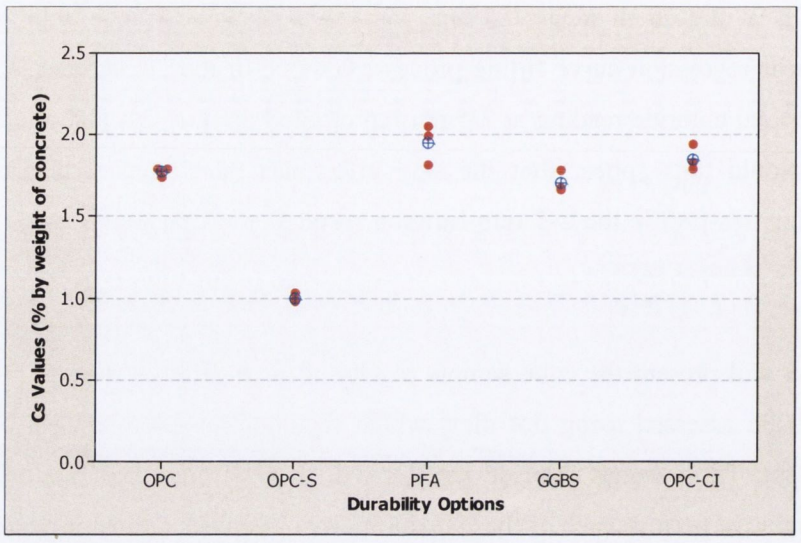


Figure 4.17 C_s values obtained for cube samples

As can be seen from the plot the C_s values obtained range from 0.98% total chloride by weight of concrete to 1.99% total chloride by weight of concrete. Using the average density obtained from the cube tests and the binder content of 500kg/m^3 these values can be expressed as 4.61% to 9.35% chlorides per weight of binder. The values of C_s will however be in general discussed in terms of % by weight of concrete herein unless otherwise stated. From Figure 4.17 it can be noted that the C_s value for the OPC-S durability option is considerably lower than the C_s values obtained for the other SCC options with an average value of 1.00% by weight of concrete. A lower C_s value for OPC-S is to be expected as the silane is a surface applied hydrophone which is designed to prevent the ingress of water and water borne contaminants, such as chlorides, into the concrete. The OPC, GGBS and OPC-CI can be seen to have similar values of C_s to one another, with the mean value estimates of 1.77%, 1.70% and 1.85 respectively. The PFA C_s value appears to be slightly elevated with a mean C_s value of 1.95% however the difference between PFA and OPC, GGBS and OPC-CI does not appear to be large and could be down to random variation.

In order to fully investigate the C_s values presented above statistical methods must be utilised to ascertain if the difference observed between two sets of results is statistically significant. To this end Figure 4.18 shows the C_s mean value estimates obtained for each durability option from the experimental testing, together with interval bars which represent the 90% confidence interval for the actual long run mean value. The mean value estimates are obtained from the three duplicate specimens in each sample set, while the actual long run mean value is the mean value which would be obtained if an infinite number of samples were analysed. The interval bars are obtained using the standard error associated with each mean value estimate and are calculated using Equation 4.2.

$$Conf.Int. = \bar{x} \pm t_s \times \frac{s}{\sqrt{N}} \qquad \textbf{Equation 4.2}$$

where *Conf. Iint.* is the confidence interval associated with the mean value estimate, $\bar{x}$ is the mean value estimate of the true mean based on the sample set, s is the standard deviation associated with the sample set, N is the number of samples in the sample set, t_s is the test statistic obtained from the t-distribution table using the degrees of freedom and the α value; the confidence with which the interval is concerned i.e. an α value of 0.05 corresponds to a 95% confidence interval.

As a general rule, when considering two mean value estimates, it can be said that there is a statistically significant difference between their long run means with (1-α) x 100% confidence, if their confidence intervals as described by Equation 4.2 do not intersect. For the current analysis it is however difficult to prove statistical significance. This is due to the relatively small sample set size of N = 3 used for each SCC durability options being considered i.e. three duplicate samples for each sample set. It is desirable to have a much larger sample size, perhaps 10 samples. This however is not possible for the laboratory studies described in this thesis for the following reasons:

- There is limited space in the salt fog chambers. Obtaining addition chambers was not a feasible option for two reasons a) salt fog chambers are expensive pieces of equipment and b) insufficient space was available in the TCD civil engineering laboratories for additional environmental chambers.
- The Phase 1 testing duration was approximately 10 months including sample pouring, curing and preparation.
- There is considerable time and cost associated with sample analysis. The Phase 1 samples took over 3 months to analyse with N=3. If N was 10 the sample analysis work associated with the testing would have taken approximately 10 months. A 10 months sample analysis combined with a 9.5 month testing period would make it impossible to carry out two phases of testing over the course of a single PhD.

The sample set size of N=3, utilised in this thesis, results in a wide confidence interval when considering the mean estimate for two reasons. Firstly, the t_s term in Equation 4.2 is based on two input variables, the α-value being considered and the degrees of freedom associated with the mean estimate. The degrees of freedom value is equal to N-1. The smaller the degrees of freedom the larger the test statistic will be and the wider the resulting confidence interval will be. Secondly, the smaller the value of N, the larger the $\frac{s}{\sqrt{N}}$ term will be and the wider the confidence interval will be. Thus for a sample size of N=3 it is difficult to prove statistical significance if the difference

between two mean estimates is not very large. To highlight the effect of the small N value on the statistical analysis of the SCC samples a brief sensitivity analysis is presented when discussing the cube sample D_{app} values later in this section.

The difficulty in proving statistical significance is worsened when it is considered that this analysis investigates concrete which is not a heterogeneous or homogenous material. Consequently it is inevitable that there will be some variation from sample to sample. This variation combined with sampling error, spatial variability in testing and analysis error means that the *s* value in Equation 4.2 is likely to be relatively large for this type of testing. Thus, a notable difference between two mean values may be obtained, however it may not be possible to prove that this notable mean difference represents a statistically significant difference between long run means with 90% confidence. In this context it is suggested that patterns even if it is not possible to show statistical significance for a given pattern i.e. it is proposed that statistical difference is not the only valid measure of difference between observations. Consequently, in this chapter, the confidence interval plots investigating statistical significance are presented and discussed separately to the dot plots showing the individual values of C_s and D_{app}.

Figure 4.18 below presents the confidence intervals and mean value estimates for the cube specimen C_s values. Based on the mean estimates of C_s obtained from the experimental testing, it can be stated with 90% confidence that the OPC-S long run mean is statistically significantly different to the other four durability options considered. No statistically significant difference was detected between the long run means for the OPC, OPC-CI, PFA and GGBS durability options. This is represented Figure 4.18 by the intersection of the interval bands for OPC, OPC-CI, PFA and GGBS while the OPC-S interval band intersects none of the other SCC durability options.

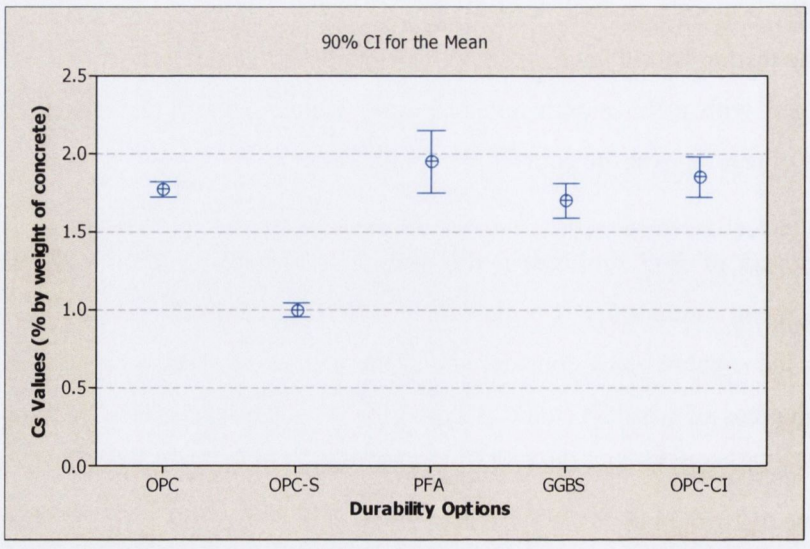


Figure 4.18 90% confidence interval plot for C_s cube results

Figure 4.19 below presents the D_{app} values obtained for the cube specimens. As can be seen from the plot the OPC and OPC-CI durability options have the greatest D_{app} values with mean estimate values of 72mm^2/yr and 76mm^2/yr respectively. As previously discussed in this chapter it was not expected that the addition of corrosion inhibitor would alter the diffusion characteristics of the OPC mix, and thus it is not surprising that these two durability options perform similarly in terms of D_{app}. The variation associated with the OPC-CI is the greatest of the durability options considered. The precise reason for the increased variation is uncertain but is likely to be due to the inherent random variability associated with concrete. The GGBS specimen displayed the lowest D_{app} values with a mean D_{app} estimate of 39.2mm^2/yr. The performance of the GGBS is followed by the PFA concrete and the OPC-S concrete with D_{app} values of 43.4 and 52.0mm^2/year, respectively.

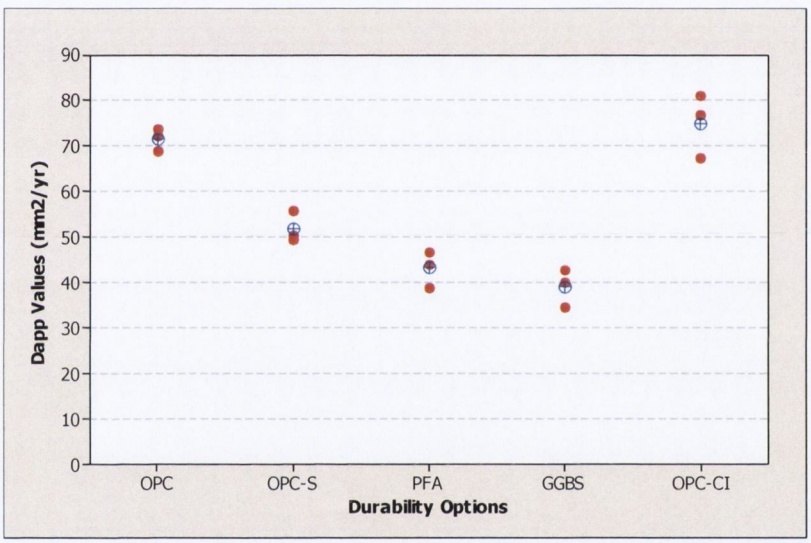


Figure 4.19 D_{app} values obtained from cube specimens

The D_{app} mean values estimates are again presented in Figure 4.20 with the interval bars associated with each durability options displayed in order to assess the statistical significance of the differences in the D_{app} results. As can be seen from the plot, for the 90% confidence interval the OPC and OPC-CI have statistically significantly larger D_{app} values than the other three durability options. Based on the mean estimates and the 90% confidence interval the PFA long run mean is not statistically significantly different from the OPC-S and the GGBS long run means. The GGBS long run mean is however statistically significantly lower than the OPC-S long run mean for the 90% confidence interval.

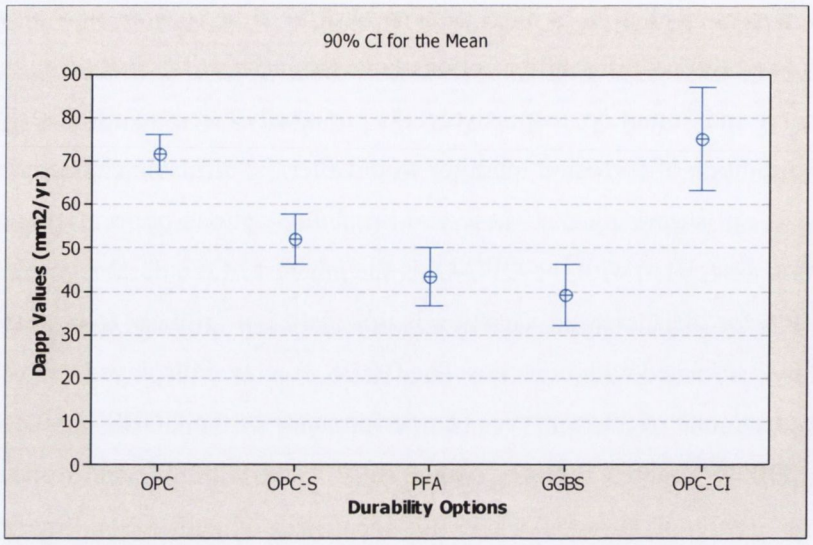
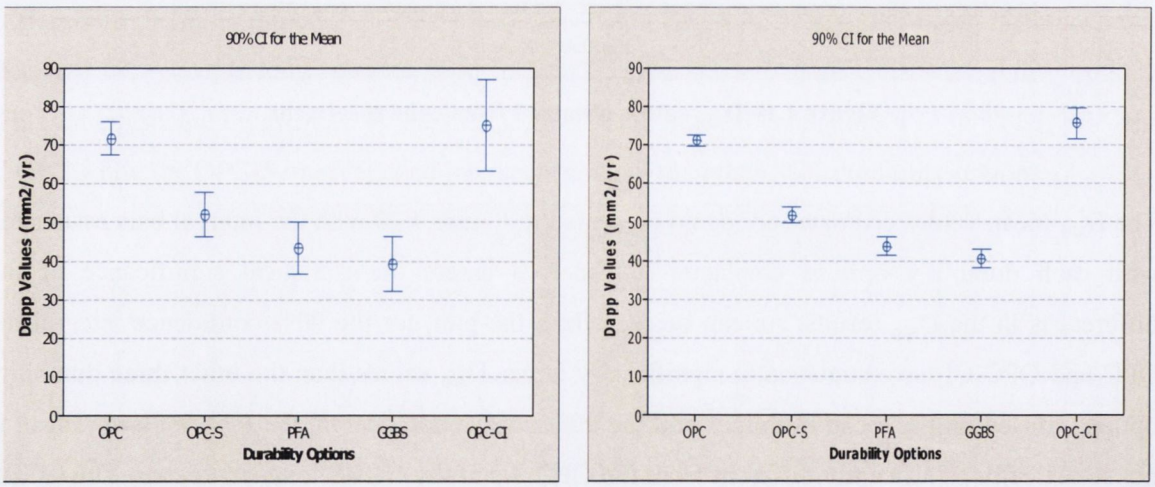


Figure 4.20 90% confidence interval plot for D_{app} cube specimens

The D_{app} values for the cube sample will now be used as an illustrative example to highlight the effect of the size of the N value on the statistical analysis through a mini sensitivity analysis. A data set was generated with the same mean value and standard deviation value as the cube specimen D_{app} values obtained from the experimental testing. 10 sample values were however generated for each SCC durability option. The 3 sample interval plot and the 10 sample interval plot can be seen below as Figure 4.21(a) and Figure 4.21(b) respectively.



(a) (b)

Figure 4.21 90% interval plot for (a) D_{app} cube sample results (N=3) and (b) generated D_{app} values with identical mean and standard deviation to experimental D_{app} results (N=10)

The impact of the N value can clearly be seen from Figure 4.21 with a statistically significant difference in long run mean with 90% confidence identified in the generated data for PFA and

OPC-S. This statistically significant difference could not be detected in the experimental results, which have the same mean value and standard deviation value due to the smaller sample size of three.

Figure 4.17 to Figure 4.21 presented above are concerned with the individual specimen results, mean values and interval bars for either C_s or D_{app}. These plots facilitate graphical presentation of the C_s and D_{app} parameters in terms of; trends or differences observed between SCC options, variation associated with the parameters for each option and the statistical significance of the differences observed. The preceding discussion however deals with the C_s and D_{app} parameters separately. It must be noted that it is the interaction between these two parameters which dictates the extent of total chloride ingress over an exposure period. Figure 4.22 to Figure 4.25 below utilise both parameters together to present the average chloride profiles for each durability option. These chloride profiles were developed using the mean C_s values and mean D_{app} values obtained for each durability option. Consequently, each of the lines plotted in Figure 4.22 to Figure 4.25 represents an average of three chloride profiles obtained from the three 100mm cube specimens. The plots facilitate quantification of the combined effect of the C_s and D_{app} and allow the performance of the five durability options to be compared graphically over the exposure period. Each of the SCC option chloride profiles is plotted together with the OPC SCC chloride profile in Figure 4.22 to Figure 4.25, with the OPC chloride profile serving as a benchmark or control.

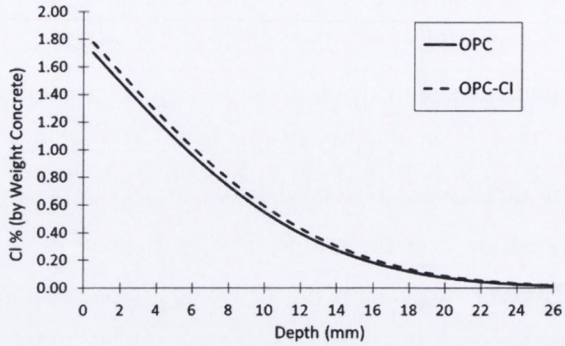


Figure 4.22. Chloride profiles for OPC & OPC with a Corrosion Inhibitor resulting from 36 weeks accelerated exposure

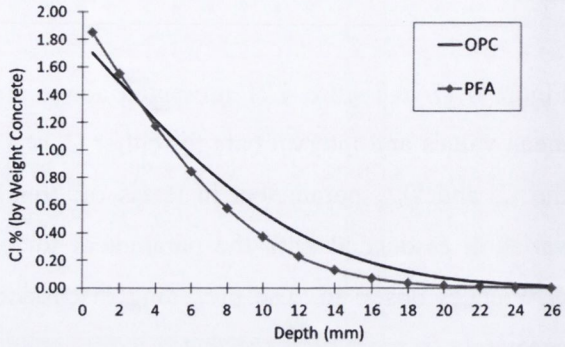


Figure 4.23. Chloride profiles for OPC & OPC with 30% PFA resulting from 36 weeks accelerated exposure

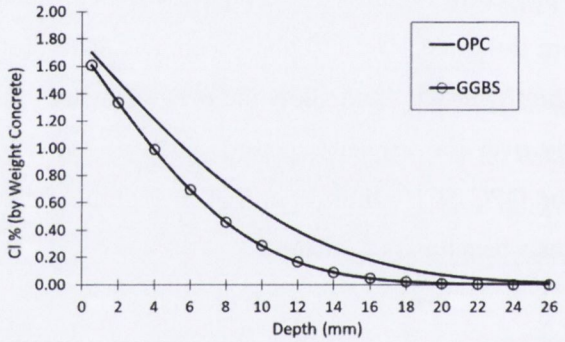


Figure 4.24. Chloride profiles for OPC and OPC with 60% GGBS resulting from 36 weeks accelerated exposure

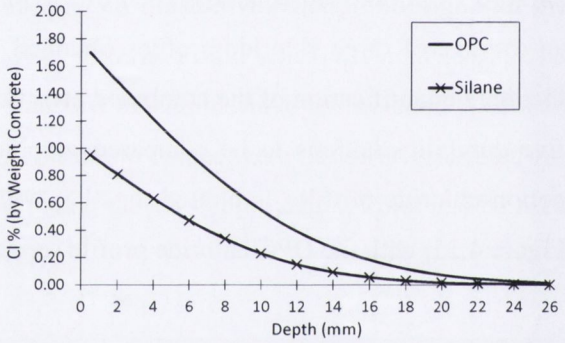


Figure 4.25. Chloride profiles for OPC & OPC with Silane resulting from 36 weeks accelerated exposure

As can be seen from Figure 4.22 the average chloride profiles for OPC and OPC-CI are very similar with both chloride profiles following an almost identical path. The cube sample test results thus indicate that addition of the corrosion inhibitor to the OPC mix neither increases nor decreases the rate of ingress of chlorides. This finding is in contrast with the recent study by Luo and Schutter (2008) which found that the addition of corrosion inhibitors could increase the rate of chloride ingress in concrete.

Figure 4.23 compares the performance of the traditional OPC option to the PFA durability option. As previously seen in Figure 4.17 the PFA cube samples have a slightly elevated mean C_s value. This results in the PFA chloride profile having a higher total chloride content than the OPC option up to a depth of approximately 4mm. There is however a more rapid reduction in total chloride content with depth into the sample for the PFA profile evidenced in Figure 4.23 by the steeper

slope of the PFA chloride profile. This is due to the fact that the PFA SCC has a lower mean D_{app} value than the OPC SCC as discussed in the context of Figure 4.19. Consequently, it is clearly seen that the PFA SCC out performs the OPC SCC for the Phase 1 100mm cube specimens.

Figure 4.24 compares the GGBS SCC durability option to the standard OPC durability option. It can be seen from the plot that the GGBS option results in a reduction in the quantity of total chlorides ingressed at each depth increment. The slope of the GGBS chloride profile is the largest of the five chloride profiles seen in Figure 4.22 to Figure 4.25. This reflects the fact that the GGBS concrete has the lowest mean D_{app} value, as illustrated in Figure 4.19. Figure 4.25 compares the standard OPC option to the OPC-S durability option. The plot highlights the fact that for the cube samples the silane considerably improves the performance of the OPC concrete, most notably through a large reduction in the surface chloride content as noted in Figure 4.17. This has resulted in the OPC-S durability option notably outperforming the OPC option over the exposure period.

Having compared the performance of the durability options graphically in Figure 4.22 to Figure 4.25 the SCC options will now be assessed in quantifiable form by comparing the predicted time to initiation of corrosion (T_i) for each option. It must be noted that this prediction of the time to corrosion initiation is merely used as an illustrative example to quantify the performance of the SCC durability options using the D_{app} values and C_s values obtained from the 36 week exposure period. The purpose of estimating T_i for the SCC options below is not to predict when corrosion might initiate in a real marine structure. What is important in the analysis however is the relative performance of the SCC durability options in resisting chloride ingress under the uniform laboratory conditions. The illustrative example will use the simplest form of Fick's second law of diffusion to calculate T_i. A more sophisticated model will be used when a full probabilistic analysis of the experimental results is carried out in Chapter 7.

For the purpose of this illustrative example a cover of 50mm was adopted for the SCC durability options in accordance with the guidelines of EN 1992-1-1-2004 for a bridge beam in a marine splash zone (Centre Europeen de Normalisation CEN 2004). A critical chloride content, C_{cr}, of 0.07% by weight of concrete is utilised for the analysis in accordance with the findings of Kenshel (2009). The selection of this parameter will be discussed in detail in Chapter 6 in the context of service life prediction. The time to initiation of corrosion for each durability option is calculated based on Fick's law using Equation 4.3.

$$T_i = \frac{x^2}{4 \times D_{app}} \times \left[erf^{-1}\left(\frac{(C_{cr} - C_s)}{(C_i - C_s)} \right) \right]^{-2} \qquad \textbf{Equation 4.3}$$

where x is cover depth, D_{app} is the apparent diffusion coefficient, erf is the error function, C_{cr} is the critical chloride content, C_s is the surface chloride content C_i is the initial chloride content in the concrete and T_i, is the time to initiation of corrosion.

Table 4.5 below presents the predicted mean time to initiation of corrosion for each of the SCC durability options. As one would expect for accelerated exposure conditions, the times to initiation of corrosion calculated are relatively short when compared to values of T_i one would expect to encounter in the real marine environment. As previously stated however it is the relative performance of the durability options which are of interest herein. To this end Table 4.5 also presents each predicted T_i value normalized with respect to OPC by dividing each T_i value by the T_i value obtained for the OPC durability option. These normalised T_i values are referred to in the table as the relative merits as they describe the performance of each SCC option relative to the control OPC SCC option.

Table 4.5 Calculated cube specimen mean T_i values and mean relative merits

Durability Option	T_i (years)	Mean Relative Merit $\left(\frac{T_i}{T_i OPC}\right)$
OPC	4.1	1.00
OPC-S	7.3	1.78
PFA	6.6	1.60
GGBS	7.7	1.86
OPC-CI	3.9	0.94

As can be seen from Table 4.5 the OPC-S and GGBS SCC durability options are the best performing cube specimens with relative merits of 1.78 and 1.86 respectively. It is interesting to note that based on the 36 week exposure period the use of 60% GGBS in the cube samples is very similar to the effect of applying a silane surface treatment to the OPC concrete. This is discussed further in the context of longer exposure durations in Chapter 7. The cube specimen results indicate that both of these durability options represent a considerable improvement over the OPC concrete in terms of resistance to the ingress of total chlorides. The relative performance of the PFA durability option sits between that of the top two performing options and OPC with a relative merit

of 1.60. Finally it can be noted from the OPC-CI performs similarly to the OPC with a relative merit of 0.94.

Each T_i value and relative merit presented in Table 4.5 are based on the mean C_s and D_{app} values obtained from the three duplicate cube samples for each SCC durability option. Figure 4.26 below is an interval plot based on the T_i values calculated for the durability options using the C_s and D_{app} obtained for each duplicate sample. The nature of the calculation dictates that the OPC option will have a relative merit of one, however this option is till plotted to allow appreciation of the variation in the calculated OPF T_i values.

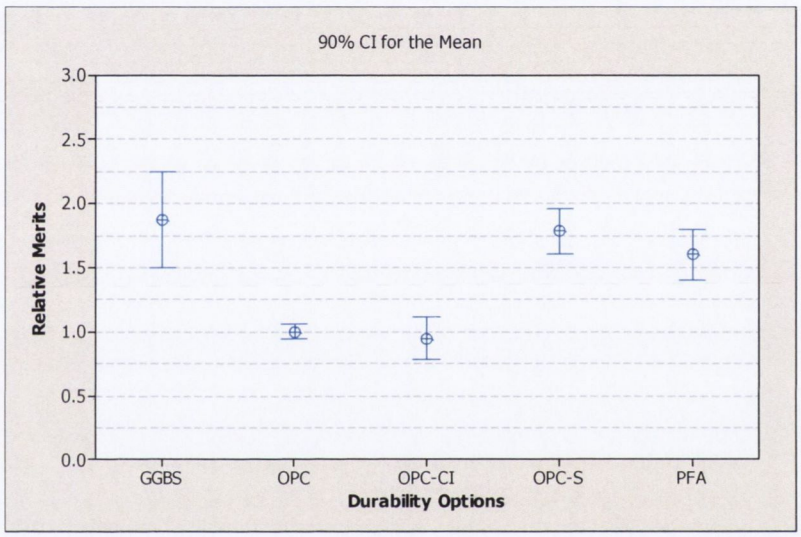


Figure 4.26 Cube specimen interval plots for relative merits based on predicted T_i

From Figure 4.26 it can be said with 90% confidence that the long run mean relative merit for GGBS, OPC-S and PFA are statistically significantly greater than the long run mean relative merit for OPC and OPC-CI. For the 90% confidence interval there is not a statistically significant difference between the relative merit for PFA, OPC-S and GGBS despite the PFA having a 10% and 14% lower mean relative merit than OPC-S and GGBS respectively. Finally it is noted that there is no statistically significant difference between the relative merit of OPC and OPC-CI when considering the total chloride ingress in the cube samples over the 36 week exposure period. These findings will be compared to the findings of other researchers published in the existing literature in Section 4.4.7.

4.4.5 Slab Specimen C_s, D_{app}, Chloride Profiles and Relative Merits

Having discussed the 100mm cube specimen results in detail the 300 x 300 x 120mm slab specimen results will now be analysed in their own right before the slab and cube results are compared and contrasted in Section 4.4.6 to access the effects of the testing on different scales, using different salt fog chambers. Figure 4.27 below presents the C_s values obtained from the slab specimens. As with the cube samples, it can be seen that the OPC-S results in the lowest surface chloride content values with a mean C_s value of 0.60. The GGBS specimens have the largest C_s values with a mean C_s of 1.27%. The PFA option results in the second largest mean C_s of 1.08% while the OPC option has a mean C_s value of 0.94% respectively. It can be noted that the variation associated with the OPC and OPC-S specimens is larger than the variation associated with the PFA and GGBS results. The reason for this is uncertain and is suspected to be due to experimental, material and/or analytical variation.

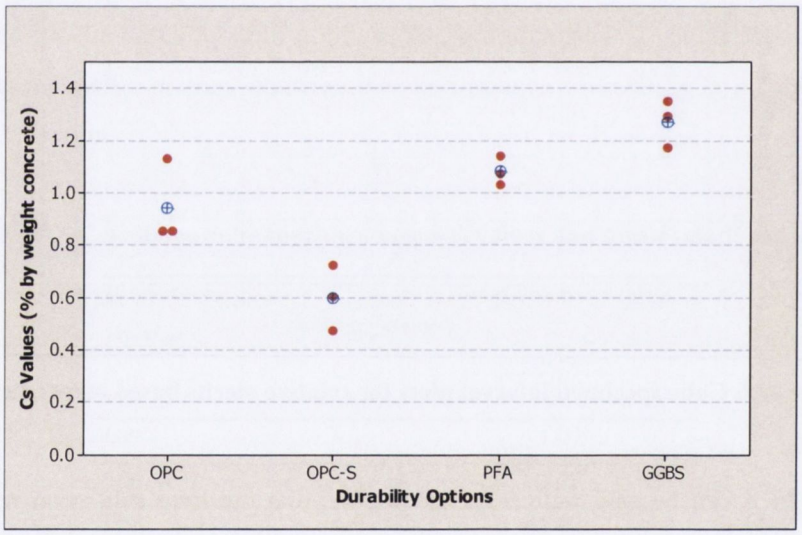


Figure 4.27 C_s values obtained for slab specimens

Figure 4.28 presents the mean C_s values obtained from the experimental testing together with the interval bars to facilitate statistical assessment of the differences identified in Figure 4.27. Due to the larger variation obtained for the OPC-S samples and the OPC samples a statistically significant difference between the long run means for OPC and OPC-S cannot be detected with 90% confidence. It can be noted however from Figure 4.28 that the long run means for both GGBS and PFA are statistically significant different from the OPC-S long run mean value with 90% confidence.

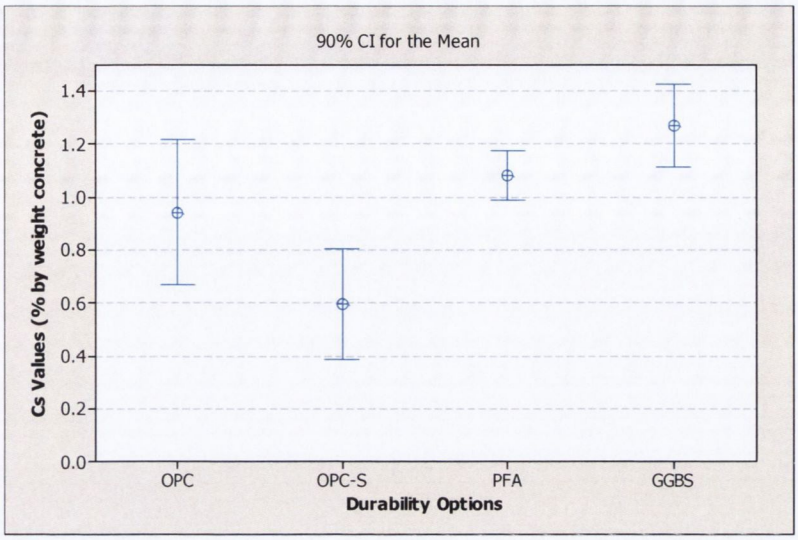


Figure 4.28 90% confidence interval plot for slab specimen C_s values

Figure 4.29 presents the values of D_{app} obtained for the slab samples from the 36 weeks of exposure in the salt fog chamber. It is clear from the plot that the GGBS SCC durability option results in a significantly lower D_{app} value than the other durability options. The GGBS mean D_{app} value is 16.3mm^2/yr while the other three durability options have D_{app} values ranging between 30.1mm^2/yr and 43.0mm^2/yr. The PFA slab specimens have a similar mean D_{app} value to the OPC specimens with values of 39.7mm^2/yr and 43.0 mm^2/yr respectively observed. The three OPC-S D_{app} values are lower than any of the OPC and PFA D_{app} values. As can be seen from Figure 4.29 the variation associated with the D_{app} values is notably small with the exception of the OPC SCC which exhibits similar variation as that seen in the cube samples.

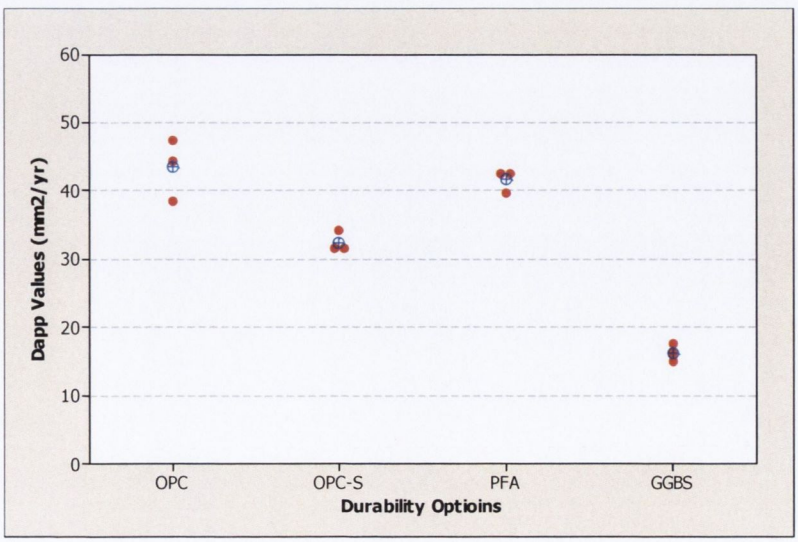


Figure 4.29 D_{app} values for slab specimens

Figure 4.30 below facilitates the statistical analysis of the D_{app} results for the slab specimens. From the plot it can be stated with 90% confidence that the OPC-S D_{app} long run mean and the GGBS D_{app} long run mean are statistically significantly lower than the long run means for OPC and PFA. The long run mean D_{app} value for GGBS is statistically significantly lower than that of OPC-S. A statistically significant difference between the D_{app} long run means for the OPC and PFA slab samples could not be detected with 90% confidence.

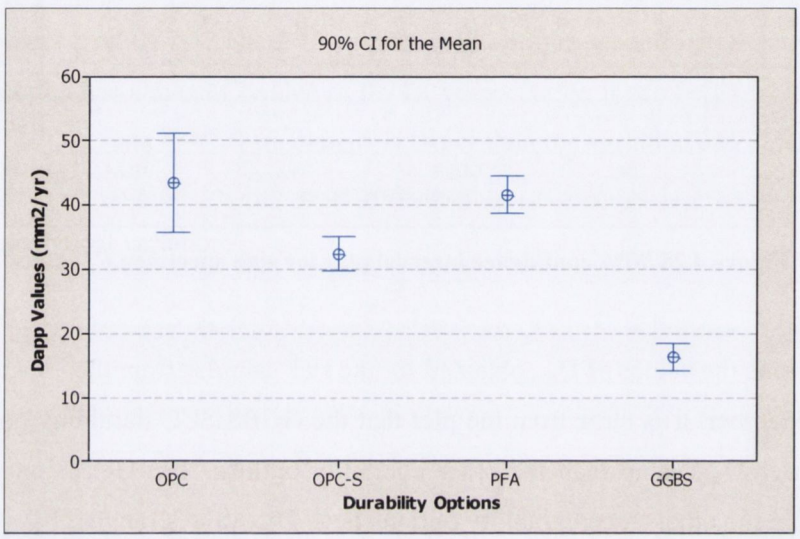


Figure 4.30 90% confidence interval plot for D_{app} slab specimens

Again having examined the individual D_{app} and C_s results and the variation associated with the mean estimates each SCC durability option will now be accessed through the presentation of the slab sample average chloride profiles. Figure 4.31 presents the average chloride profile for the standard OPC. Figure 4.32 to Figure 4.34 present the average chloride profiles for the other durability options plotted together with the average chloride profile for the standard OPC option. Again this facilitates a comparison between each alternative durability option and the control OPC option.

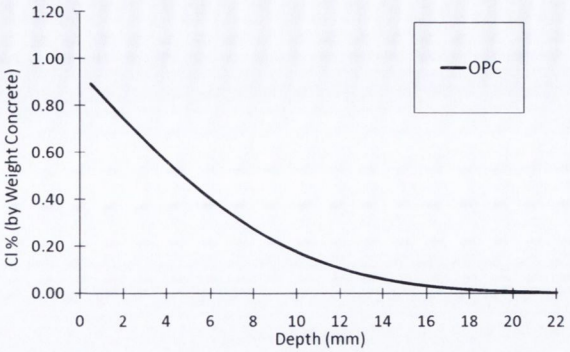


Figure 4.31 Chloride profiles for OPC slab sample resulting from 36 weeks accelerated exposure

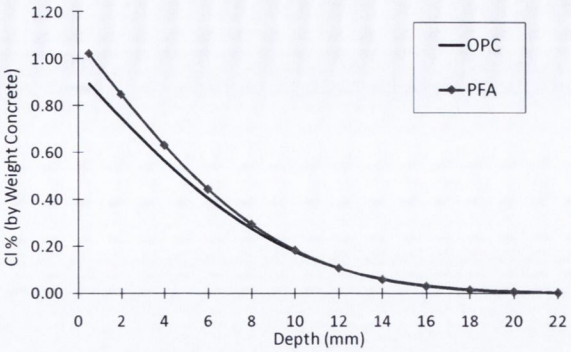


Figure 4.32 Chloride profiles for OPC & OPC with 30% PFA slab samples resulting from 36 weeks accelerated exposure

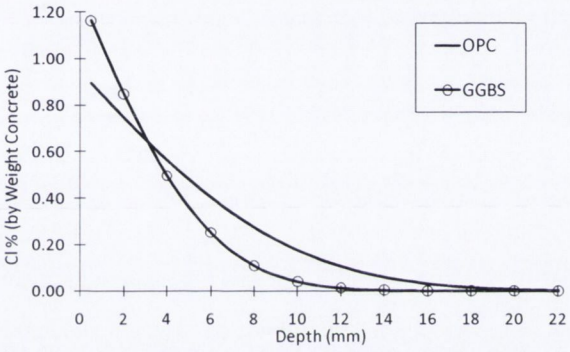


Figure 4.33 Chloride profiles for OPC and OPC with 60% GGBS slab samples resulting from 36 weeks accelerated exposure

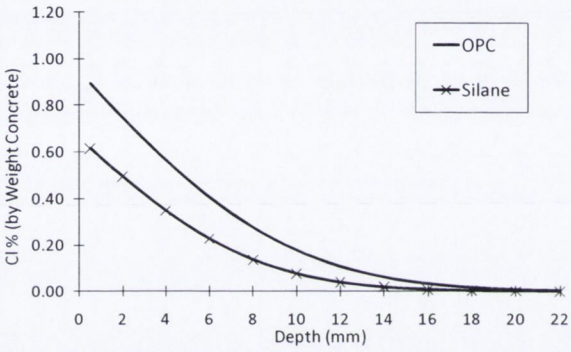


Figure 4.34 Chloride profiles for OPC & OPC with Silane slab samples resulting from 36 weeks accelerated exposure

From Figure 4.31 it can be seen that over the 36 week exposure period significant levels of chlorides have ingressed into the OPC slab sample with non-zero chloride levels found up to a depth of 20mm. From Figure 4.32 it can be noted that the chloride profile for PFA is very similar to the profile for the OPC option. In fact the slightly higher value of C_s observed for the PFA in Figure 4.27 results in an overall poorer performance of the PFA durability option over the exposure period. Figure 4.33 compares the performance of the OPC durability option and the GGBS durability option. The initial heightened mean C_s value obtained for GGBS quickly reduces with depth into the slab sample due to the influence of the low GGBS D_{app} previously highlighted in Figure 4.29. It is clear from Figure 4.33 that the GGBS out performs the traditional OPC option over the exposure period with the GGBS exhibiting lower chloride content at every depth interval beyond approximately 7mm. Finally from Figure 4.34 it can be seen that the low mean C_s value obtained for the OPC-S, previously highlighted by Figure 4.27, results in reduced chloride content over the OPC option at every depth interval.

As with the cube samples the relative performance of the slab durability options will now be assessed in quantifiable form by comparing the predicted time to initiation of corrosion for each option. The T_i values for the slab sample were calculated in an identical manner to the T_i values for the cube samples. Table 4.6 presents the predicted mean time to initiation of corrosion and mean relative merit for each SCC option.

Table 4.6 Calculated slab specimen mean T_i values and mean relative merits

Durability Option	T_i (years)	Mean Relative Merit $\left(\frac{T_i}{T_i OPC}\right)$
OPC	9.1	1.00
OPC-S	16.0	1.75
PFA	9.3	1.01
GGBS	21.0	2.29

As can be seen from Table 4.6 GGBS is the best performing slab specimen with a mean relative merit of 2.29. The OPC-S is next with a time to initiation of corrosion 1.75 times the T_i obtained for the OPC SCC option. The OPC and PFA durability options have very similar T_i values resulting in almost equal relative merit values. This is a somewhat unusual result which will be discussed further in the next section when the lab sample results and the cube sample results are compared and contrasted. Overall the mean T_i slab sample results indicated that both the GGBS and OPC-S represent a considerable improvement over the OPC concrete in terms of resistance to the ingress of total chlorides.

The relative merit interval plot presented as Figure 4.35 is based on the T_i values calculated for each individual duplicate sample.

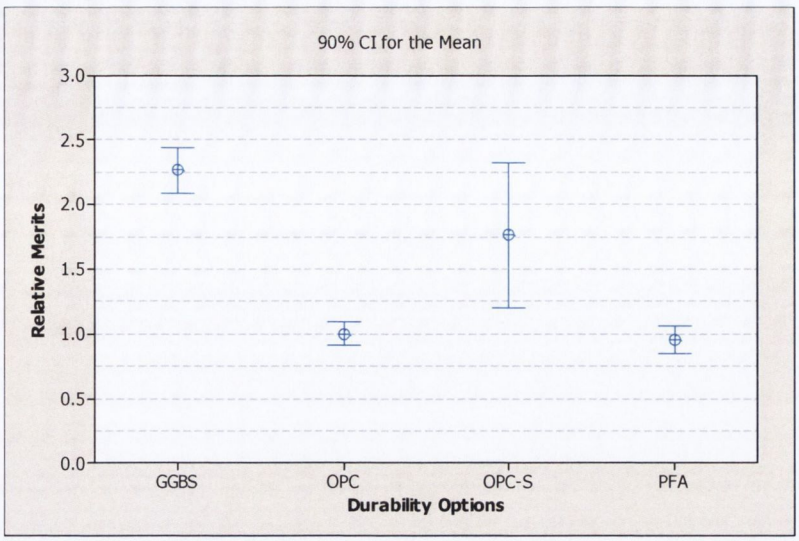


Figure 4.35 Slab specimen interval plots for relative merits based on predicted T_i

From Figure 4.35 it can be said with 90% confidence that the long run mean relative merit for GGBS & OPC-S are statistically significantly greater than the long run mean relative merit for OPC and PFA. It can be noted that based on the slab specimen results and the 90% confidence interval there is no statistically significant difference between the long run mean relative merits for OPC and PFA. There is no statistically significant difference between the relative merit for OPC-S and GGBS for the 90% confidence interval. This failure to detect a statistically significant difference between these two durability options is likely to be as a result of high level of variation associated with the predicted OPC-S T_i values. It is thought that the large variation in the performance of the OPC-S could be due to the method by which the OPC-S is applied. The silane was applied to the Phase 1 samples simply by spraying the surface of the concrete until the face was visibly saturated as per the manufacturer's guidelines. To investigate this issue further a more standardised application of OPC-S will be adopted as part of the Phase 2 testing. This will be discussed further in Section 4.4.8 which details the findings of the Phase 1 testing which were targeted for further investigation in the Phase 2 experimental programme.

4.4.6 Cube and Slab Results Comparison

Having analysed the cube and slab samples individually a comparative analysis will now be conducted. First however, possible causes for differences in the slab and cube specimen chloride profiles are presented below:

1. The cubes and slabs were exposed to the chloride environments in two different chambers. Although the chambers were programed identically the environmental

conditions in the two exposure chambers were different. As discussed in Section 4.4.2 the slab samples experienced more drying over the exposure period than the cube samples.

2. The 300 x 300 x 120mm slab samples were poured into timber formwork while the 100mm cube samples were poured into plastic moulds. All the samples were left in the moulds for two days before being wet cured. It is possible that the different types of formwork may have affected the performance of the concrete samples. The plastic moulds are likely to have left a smoother finish on the concrete. The timber formwork has the ability to absorb moisture. This could result in the skin concrete of the slab samples having a lower w/b ratio than that of the cube samples. This would mean that the skin or surface concrete for the slab specimens could be of better quality than cube samples as will be discussed in more detail in Chapter 5.

3. A size effect could have influenced the relative performance of the slab and cube samples.

4. Finally experimental error and material variation could have affected the relative performance of the slabs and the cube specimens.

As previously mentioned both the slab and cube samples were prepared from the same concrete mix. Consequently, mixing procedure and material variation is not mentioned above as a possible cause for differences in results.

Given the difference in the sample sizes, the formwork utilised and in particular the exposure environments, one would not expect the results for the slab and cube samples to be the same. It might however be expect that the relative merits of the alternative durability options would be the same for the slabs and the cubes. Figure 4.36 below presents the relative merits and interval bars for both the slab and the cube specimens. These are the relative merits previously presented separately in Figure 4.26 Figure 4.35. The mean relative merits and interval bars for the cube specimens are shown in black and denoted as No. 1. The slab specimens mean relative merits and intervals are shown in red and represented by the No. 2 on the x-axis. The OPC relative merits are not shown in the plot as the nature of the relative merit calculation means OPC relative merit must equal 1.0.

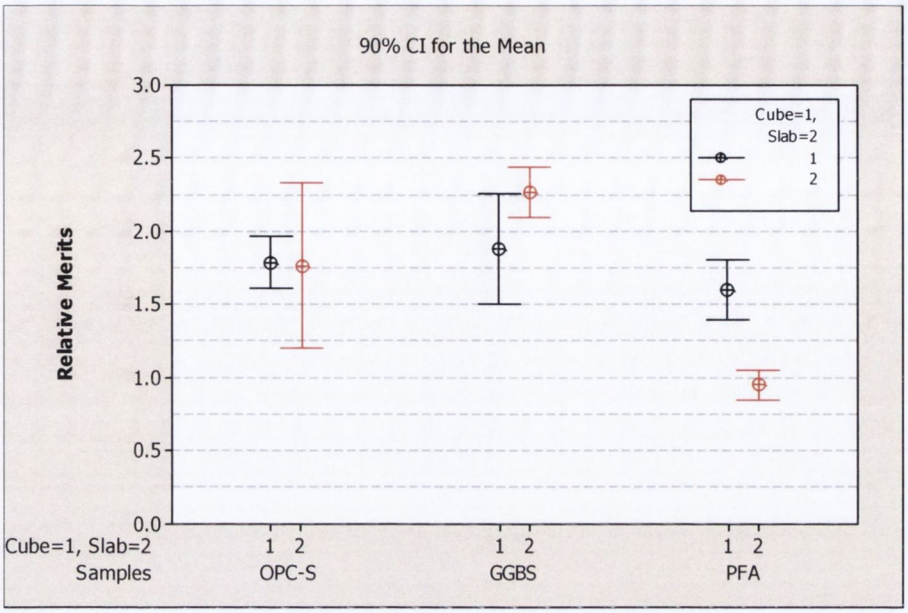


Figure 4.36 Relative merits for slabs and cubes

As can be seen from Figure 4.36 no statistically significant difference can be detected between the sets of slab and cube long run mean relative merits for GGBS and OPC-S. The mean slab and cube relative merits for the OPC-S are almost identical. For the GGBS SCC the cube mean relative merit is 19% smaller than the slab mean value. As can be seen from Figure 4.36 this difference in mean relative merit is well within the confidence intervals for GGBS. Given this relatively consistent performance across the GGBS and OPC-S slabs when compared to the OPC SCC, it is interesting to note the large difference between the slab and cube relative merits for the PFA durability option. The relative performance of the PFA SCC is 60% better for the cube specimens than for the slab specimens. As can be seen from Figure 4.36 this can be said to represent a statistically significant difference between the long run mean relative merit for the PFA slab and cube specimens with 90% confidence. The reduction in the average PFA relative merits from cube sample to slab sample can be attributed to an elevated average D_{app} value found in the PFA slab specimen. This is illustrated in Figure 4.37 which presents the average D_{app} values together with 90% confidence interval bars for both the slab and cube specimens.

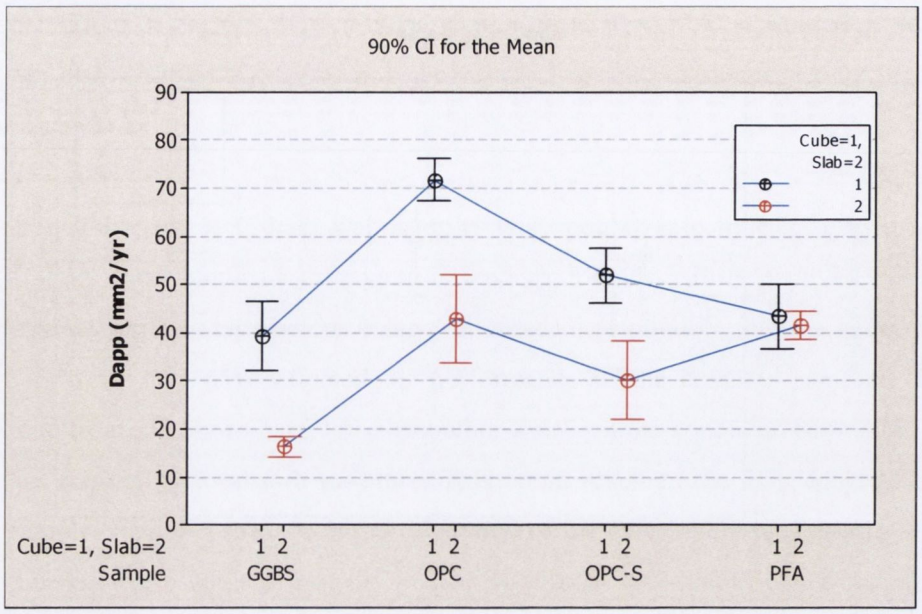


Figure 4.37 D_{app} mean values with interval bars for both slab and cube specimens

The blue lines in Figure 4.37 connect the mean D_{app} values across the four SCC options for both slab and cube specimens. As can be seen from the blue lines all of the slab and cube sample mean D_{app} values follow almost identical patterns with the exception of the PFA SCC durability option. It can be noted that each of the slab mean D_{app} values is statistically significantly lower than the corresponding mean D_{app} cube value with the exception of the PFA durability option. The reason for the lower D_{app} values in slabs for GGBS, OPC-S and OPC will be discussed later in this section. The PFA D_{app} mean values for the slab specimens and cube specimens are within 10% of one another with values of 40mm^2/yr and 43mm^2/yr, respectively. In the context of the consistent D_{app} patterns observed for the OPC, GGBS and OPC-S options, and the 90% confidence interval bars shown in Figure 4.37, the similar average D_{app} value for the PFA slabs and cubes would seem represent a systematic shift, not caused by random variation.

If the change in the PFA performance from cubes to slab specimens is systematic it is likely that there is an external influence which caused the change in performance. The differences between the slabs and cube preparation and testing have previously been highlighted. Of the two possible known causes it would not seem unlikely that the different mould types would be the cause of the poorer PFA performance in the slab samples. The change in mould types, which could lead to a difference in the quality of the skin concrete, is more likely to affect the C_s values than the D_{app} values. In addition the mould effect should affect all the SCC durability options, not just the PFA option.

Examination of the available literature provides a possible reason for the change in PFA performance due to the difference in RH in the two chambers. This possible cause relates to the absorption characteristics of OPC and OPC with PFA utilised as a partial cement replacement. Gonen and Yazicioglu (2007) examined the absorption characteristics of OPC and OPC + PFA concretes and found that the capillary absorption of concrete samples containing 15% PFA was increased by as much as 47% over OPC concrete. Yildirim et al. (2011) identified a similar effect finding that for the C40/50 strength class concrete containing PFA had a greater capillary absorption coefficient than OPC. Chan et al. (1998) however found that the replacement of OPC with 30% of PFA had the opposite effect, reducing the initial surface absorption of concrete. Guneyisi et al. (2011) found that direct replacement of OPC with PFA in the proportion of 20% & 40% resulted in a reduction in the sorptivity coefficient of approximately 2% and 15% respectively. Elahi et al. (2010) examined the water absorption characteristics of OPC concrete and OPC with 20% and 40% replacement level at 44 days and 91 days after pouring. For the 20% PFA the sorptivity index was slightly increased over OPC at 44 days but was lower than the OPC at 91 days. For the 40% PFA replacement however it was found that the sorptivity index for the PFA was 43% greater than OPC at 44 days and 16 % greater than OPC at 91 days. Bai et al. (2002) also considered the sorptivity index at a number of different concrete ages for OPC concrete and concrete containing PFA. It was found that water cured concrete with 30% PFA had a greater sorptivity index than water cured OPC concrete up to a period of 4 months. From 4 to 18 months however the 30% PFA concrete had a slightly reduced sorptivity index. This was however contradicted by the findings of McPolin et al. (2005) who found that concrete containing 30% PFA had a sorptivity index 30% greater than OPC at 28 days and 115% greater than OPC at 11 months. Thus overall it can be stated that there are mixed findings in terms of the relative absorption characteristics for OPC and PFA, however, the majority of studies seem to suggest that concrete containing PFA has a greater capacity for absorption, at least in the first number months after curing.

In the context of the Phase 1 experimental results, Figure 4.10 & Figure 4.14 in Section 4.4.2, showed that the cube samples in the SC1000 spray chamber were subjected to the equivalent of almost four 100% humidity salt fog days per week while the slab samples in the SKWT4300 were subjected to just over two 100% humidity salt fog humidity days per week. Consequently, the slab samples would have been subjected to greater amounts of drying. This would lead to absorption playing a greater role in chloride ingress in slab samples than in the cube samples.

Although the absorption characteristics of concrete are often assessed when examining concrete durability in marine environments the drying capacity of various concretes are rarely investigated.

It is true however to say that in order for a concrete to absorb chlorides it must first dry out to some degree. Consequently, relative ability of a concrete to absorb chlorides in marine environments will also be effected by the relative ability of the concretes to dry out between wetting cycles. It is stated by Nevile (1995) that the ingress of chlorides into concrete in the marine environment is strongly influenced by the sequence of wetting and drying, with the extent of movement of chlorides depending on the length of the wetting and drying periods. By the same rational, when comparing two concretes in the same wetting and drying cycle the ability of the concrete to dry out will have an influence on chloride ingress. Kanna et al. (1998) examined the drying of an OPC mortar and 35% PFA mortar in an experimental study investigating shrinkage and moisture content. It was found that after 21 days drying at 50% RH the OPC concrete had a relative moisture content of 72% while the PFA had a relative moisture content of 52%. This indicates that the PFA mortar had a greater drying capacity.

In light of the above discussion it would seem possible that the poorer relative merit of the PFA slab samples could be due to the fact that a) absorption played a bigger role in the slab testing environment, b) PFA concrete has been shown to be result in greater levels of absorption than OPC concrete and c) PFA concrete may dry more quickly than OPC concrete meaning it may be more susceptible to absorption upon wetting, after a given drying period, when compared to OPC concrete. As will be discussed in detail in Chapter 5, Phase 2 testing was designed to assess all three of these possibilities. Point a) was investigated by amendment to the fog chamber wetting and drying cycles in Phase 2 testing to induce a wetter exposure environment than either of the Phase 1 exposure conditions. Point b) was examined through a series of water absorption and sorptivity tests. Point c) was investigated in the Phase 2 testing utilising a simple drying test. In addition the slab and cube samples were tested in a single chamber in the Phase 2 testing to investigate the combined effect of the difference in moulds and the effect of specimen size.

The change in PFA performance in the two different exposure conditions in the Phase 1 test results is particularly interesting in the context of the discussions in Chapter 2 and 3 relating to the fact that the majority of chloride ingress resistance in concrete is examined in fully submerged saturated conditions. If the relative performance of PFA is particularly sensitive to the environmental conditions and the extent of drying the saturated sample tests, which make up the majority of tests in the literature and report high levels of improvement with the addition of PFA, must be treated with caution.

It should however be noted at this point that the drying process is concrete is very slow and it is acknowledged in the literature that the diffusion is the dominant means by which chlorides reach

the reinforcement depth (Nanukuttan et al. 2009; Thomas and Bamforth 1999). It is also true however to state that some areas on marine structures which are not in the tidal zone or at the edge of the splash zone will be subject to infrequent salt water exposure. These areas will experience a significant level of chloride ingress by absorption due to the fact chlorides are absorbed far quicker by absorption than by diffusion (Neville 2011).

As briefly mentioned above, another issue highlighted in Figure 4.37 which warrants further discussion is the fact that for the GGBS, OPC and OPC-S durability options, the slab specimen mean D_{app} values are significantly lower than the cube sample mean D_{app} values. Figure 4.38 and Figure 4.39 below present the slab and cube chloride profiles side by side for comparison.

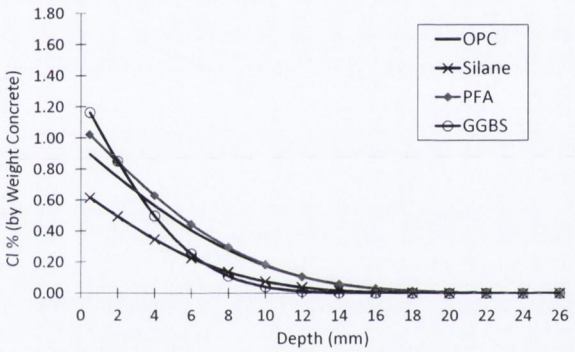


Figure 4.38: Slab specimen chloride profiles for OPC, OPC-S PFA and GGBS

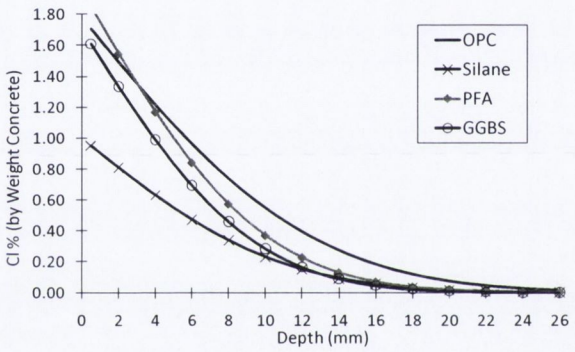


Figure 4.39 Cube specimen chloride profiles for OPC, OPC-S PFA and GGBS

It is clear from Figure 4.38 and Figure 4.39 that less chlorides have ingressed over the exposure period for the slab samples. It is likely that the greater quantities of chlorides ingressing into the cube samples are due to the increased chloride solution loading which occurs in the SC1000 chamber. As previously discussed, the cube samples in the SC1000 spray chamber were subjected to almost twice the duration of 100% salt fog humidity as the slab samples in the SKWT4300. The results of the Hong and Hooton (1999) study, presented in Chapter 3, showed that the wetting and drying cycle adopted by the Hong and Hooton increased ingress over pure diffusion, but further increase in the drying duration for a given testing time reduced the ingress of chloride due to reduced contact with the chlorides. This is in agreement with points made by Zhang and Jin (2011), also presented in Chapter 3, which alluded to the fact that when considering the changes in marine exposure with distance above low tide, as the point considered is moved away from the low tide level the accumulation of chloride on the surface of the concrete is at first aggravated due to the

increasing drying time proportion, and then weakened due to the reduction in contact time with chlorides.

When considering the slab and cube results comparison as a whole it can be said that a good level of consistency has been found between the patterns observed in the slab results and the cube results with the exception of the PFA SCC durability option. The otherwise consistent patterns provide confidence that the anomaly relating to PFA performance from SC1000 to SKT4300 chamber exposure was not due to random variation. Consequently, the further examination of this finding constituted an important part of the Phase 2 experimental programme, as will be discussed in detail in Chapter 5. This finding from the Phase 1 testing, combined with examination of the literature presented in Section 3.3 of Chapter 3, also lead to the development of Research Question 2, presented in the introduction and listed again below for clarity;

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

4.4.7 Comparison of Phase 1 Results & Results from the Existing Literature

There is a large amount of data available in the literature which compares the performance of different durability options. As discussed in Chapter 3 there is however, very little literature available which examines durability options in the context of SCC. To facilitate a broad comparison the Phase 1 results will be compared to both traditional vibrated concrete and the limited number of SCC durability studies published to date. Studies examining concrete durability options in the literature come from a variety of different sources ranging from data collected from 50 year old marine structures (Pack et al. 2010), to data from laboratory tests carried out over a number of hours which examine the charge passed through concrete samples under impressed current (Al-Amoudi et al. 2011). The relative worth of the various test set-ups has been discussed in Chapter 2 and Chapter 3.

One cannot directly compare the C_s, D_{app} or T_i values obtained from the various studies due to the variety of exposure situations utilised. For instance the times to initiation of corrosion resulting from exposure in the salt fog chambers cannot be directly compared to the T_i values found in marine structures or the T_i values found at marine exposure sites. It is true however to state that the exposure conditions experienced at each marine site are also different. Consequently, if two exposure sites were set up to test durability options the results from the two sites could not be directly compared to one another. It is instead the relative performance of the durability options at

each site which would be of interest. Consequently the comparative study between the Phase 1 experimental results and results available in the literature will be conducted utilising the relative merits of the alternative durability options when compared to the OPC option, often termed the control concrete in the material comparative literature (Bai et al. 2003; Stanish and Thomas 2003; Thomas and Bamforth 1999).\ This will allow performances from the range of testing set-ups to be examined, facilitating the broadest comparison of the Phase 1 tests with the existing literature.

The relative merits comparison will be conducted using both tables and figures. The tables provide the essential details for each published study together with the relative merits obtained. The figures present only the relative merits in graphical form facilitating easy comparison. Due to the large volume of studies carried out across the range of testing formats the Phase 1 results will be compared to results from each form of assessment method separately. The first comparison will be between the Phase 1 testing and the studies in the literature which utilised information from real marine structures and from concrete specimens tested at marine exposure sites. Table 4.7 presents background details for the studies including binder types, w/b ratios, exposure conditions, exposure durations and whether SCC or VC was examined. Figure 3.42 presents a dot plot of the relative merits obtained for each study. The nature of the analysis means that the OPC relative merit is always 1 and thus is not included in the dot plot. The x-axis of the dot plot presents the source study and letters indicating the type of exposure. The (a) group denotes the Phase 1 tests, the (b) group denotes the data from marine structures (MS), the (c) group denotes data from marine exposure sites in the spray zone (MES-S) and the (d) group denotes data from the marine exposure site specimens which were subjected to either tidal conditions or submerged conditions (MES-T, MES-Sub).

Table 4.7 Relative Merit comparison for marine structures and marine exposure sites

Exposure	Exposure time (years)	Concrete mix			Relative merit		Notes	Reference
		Binder	w/b	Concrete Type				
					Cubes	Slabs		
Phase 1 Results: Salt Spray	0.7	OPC	0.44	SCC	1.0	1.0		Current Study
	0.7	30% PFA	0.44	SCC	1.6	1.0		
	0.7	60% GGBS	0.44	SCC	1.9	2.3		
	0.7	OPC-S	0.44	SCC	1.8	1.8		
Marine structures (Tidal Zone)	2.2	OPC	0.39	VC	1.0			(Pack et al. 2010)
	2.8	49% GGBS	0.40	CVC	1.8			
	9.0	OPC	0.43	VC	1.0			
	8.3	45% GGBS	0.44	VC	1.9			
Marine structures	1-60	OPC	varied	unknown	1.0			(Melchers and Li 2009)
	1-15	GGBS	varied	unknown	2-3+			
Marine structures (Tidal and Spray Zone)	5,12	OPC	not given	VC	1.0, 1.0		Based on chloride profiles obtained from quay walls. Note: Silane treated area is in a different location to untreated area	(Schueremans et al. 2007)
	5,12	Silane	not given	VC	8.3, 6.5			
Marine exposure site specimens (Splash Zone)	1,3,8	OPC	0.66	VC	1.0, 1.0, 1.0		Specimens exposed in the splash zone	(Thomas and Bamforth 1999)
	1,3,8	30% PFA	0.54	VC	1.7, 4.5, 10.8			
	1,3,8	70% GGBS	0.48	VC	1.0, 5.0, 10.9			
Marine exposure site specimens (Tidal Zone)	25	OPC	0.40	VC	1.0		No duplicate samples	(Thomas et al. 2008)
		65% GGBS	0.40	VC	14.9			
				VC				
Marine exposure site specimens (Tidal zone)	15	OPC	0.45	VC	1.0		Tidal cycles, 7.5 hrs wet, 4.5 hrs dry	(Mohammed et al. 2002)
	15	10-20% PFA	0.45	VC	2.5			
	15	60-70% GGBS	0.45	VC	7.1			
Marine exposure site specimens (Submerged samples)	4	OPC	0.4	VC	1.0		Submerged Samples	(Bentz et al. 1996)
	4	30% PFA	0.4	VC	4.0			
Marine exposure site specimens (Tidal Zone)	2,4	OPC	0.49	VC	1.0, 1.0			(Thomas and Matthews 2004)
	2,4	30% PFA	0.39	VC	3.1, 10.9			

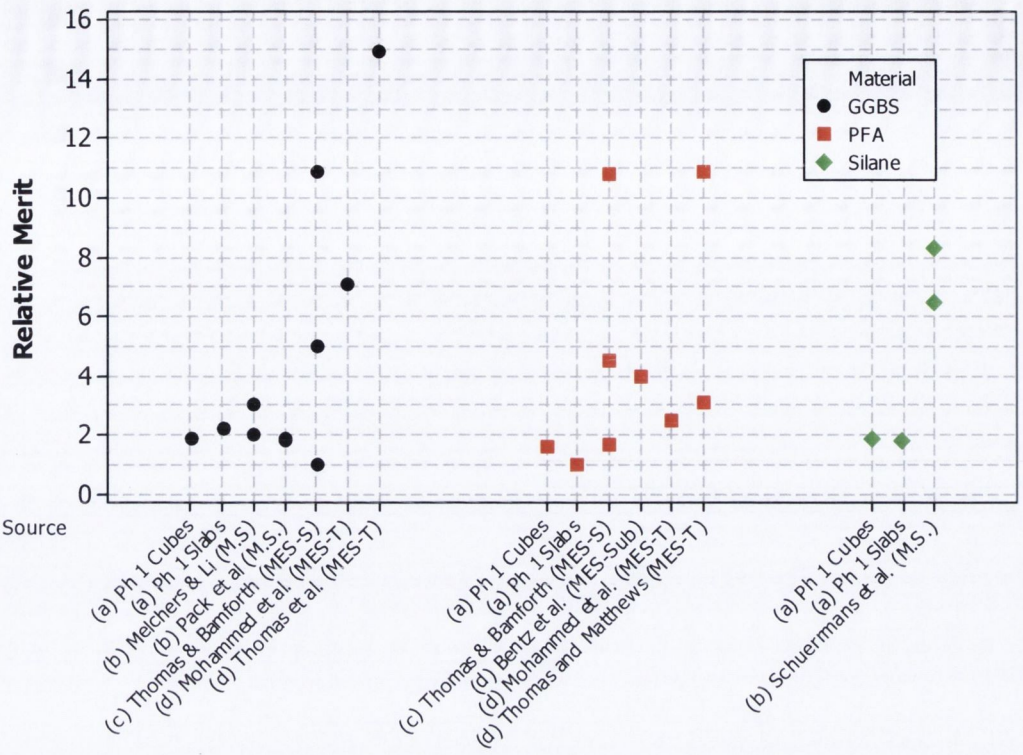


Figure 4.40 Dot plot of relative merits for Phase 1 testing, marine structures and marine exposure sites

Firstly, considering the GGBS relative merits in Figure 4.40, it can be noted the values range from 1.0 to 14.9. The results obtained by Thomas and Bamforth (1999) indicate that the duration of exposure has a high influence on the relative performance of GGBS. This time influence is likely to be due to the time dependant nature of the diffusion coefficient which has been found to be greater in GGBS concrete than OPC concrete (Audenaert et al. 2010; Pack et al. 2010; Stanish and Thomas 2003), as discussed in detail in Chapter 6. The largest relative merit for GGBS was obtained from the study carried out by Thomas et al. (2008) which is based on specimens exposed in the marine environment for 25 years. The study by Pack et al. (2010) however, which utilises real marine structures, shows a similar GGBS relative merit at 2.8 years and 9.0 years, with the two dot plot points almost overlaid. It should be noted that the Pack et al. relative merits were obtained by comparing results from different OPC and GGBS structures. Thus variation of exposure condition from one site to the next is likely to have influenced results.

In order to facilitate a more sensitive comparison of the GGBS results Figure 4.41 shows the relative merits over the ranges 0 to 8.5. This means that the Thomas et al. GGBS relative merit result is not plotted in Figure 4.41. The Thomas and Bamforth results at one and three years are plotted, however, the 8 year results for both GGBS and PFA are not plotted. The Thomas and

Matthews PFA relative merit at two years is plotted however the four year result is not plotted in Figure 4.41.

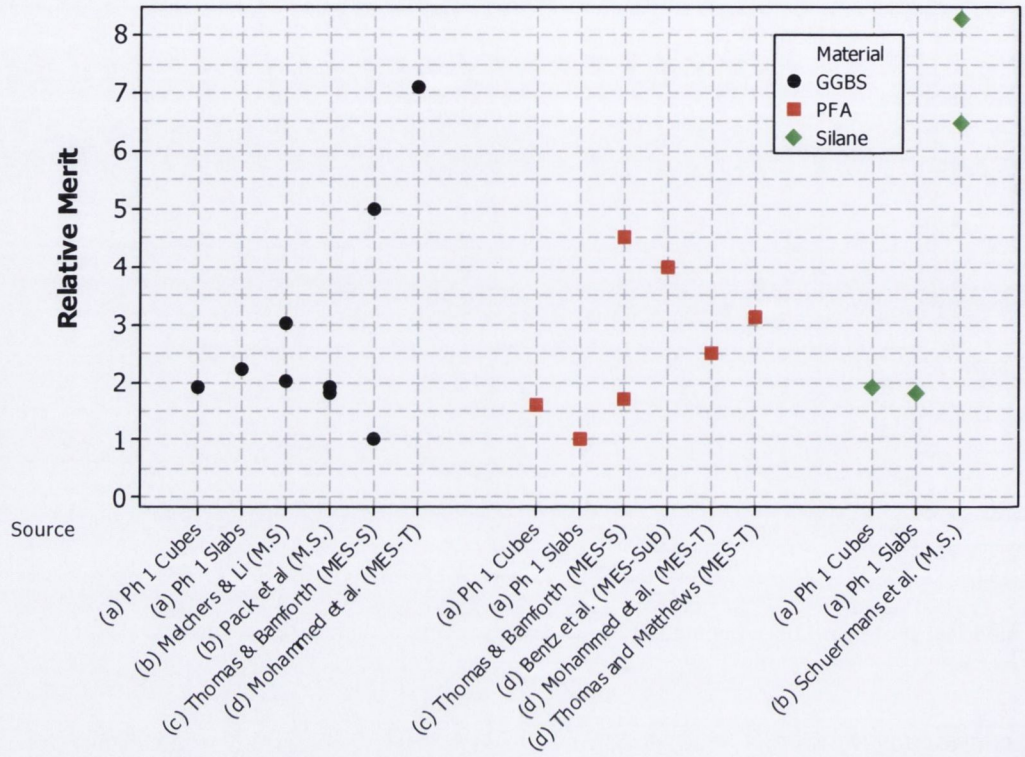


Figure 4.41 Relative merits of Phase 1 results, marine structures and marine exposure sites from values of 0 to 8.5

From Figure 4.41 it can be seen that the Phase 1 GGBS relative merits are in a similar range to a number of the relative merit values plotted. The 1 year value obtained by Thomas and Bamforth is the lowest GGBS relative merit observed, however it is noted that the relative merit of the GGBS concrete in this study increased to 5.0 at three years, and 10.9 at 8 years. The Mohammed at al. (2002) GGBS relative merit is notably higher than the Phase 1 GGBS relative merits. This difference is again likely to be due to age effects with the exposure duration in the Mohammed study at 15 years. It is noted at this point that the time dependant nature of the chloride diffusion process will be incorporated into the probabilistic modelling when utilising the Phase 1 test results to make service life predictions in Chapter 7.

As with the Pack et al. results, the Melchers & Li relative merits values were obtained from collecting data from a range of structures, however it is noted that the Melchers and Li study considered a greater number of structures than the Pack et al. study. These relative merits values are in the same range of the two Phase 1 relative merits values, however the data from the real

structures was obtained from older concretes. Thus it might be expected that the relative merits from these structures would be greater than the Phase 1 relative merits. It is also noteworthy however that the relative performance of GGBS may be reduced in real structures due to load induced cracking. As highlighted by the work of Li (2000), chloride movement in concrete with crack widths greater than 0.2mm is dominated by transport through the cracks. Thus the relative performance of GGBS concretes will be reduced in cracked members.

At this point it is noted that there is a possible effect or issue in the Phase 1 testing methodology which may have reduced their relative merits over other studies in the literature. This is the effect of the initial chloride absorption into the concrete samples upon first exposure to the salt spray fog. As discussed in Section 4.3.1 the test samples were removed from the curing tank at 28 days and painted before being exposed in the salt fog cambers. The painting procedure, however, dictated that the samples were in the ambient laboratory environment for 3 weeks during the painting process. Thomas and Matthews (2004) cited initial absorption as a considerable error in determining the relative diffusion characteristics of OPC concrete and concretes containing PFA. It was found by Thomas and Matthews that the initial absorption accounted for a significant amount of the chlorides in the PFA sample after 1 year while the initial absorbed chlorides only accounted for a small portion of the chlorides in the OPC samples after 1 year. Dhir et al. (1993; 1994) pre-soaked air cured OPC and OPC + PFA concrete prior to exposure in a salt spray chamber to rule out rapid absorption of chlorides upon commencement of the exposure period. The majority of researchers however do not cite a need to pre-soak concrete specimens prior to testing (Li 2001; Loser et al. 2010; McPolin et al. 2005; Stanish and Thomas 2003). This is likely to be due to the fact that wet curing techniques are more common in the literature than the dry curing adopted by Dhir et al. (1994) as previously discussed. Wet curing limits pre-exposure drying time to the time taken for sample preparation before the test commences. In addition concrete elements are subject to long drying times during exposure if located in splash or atmospheric zones and before exposure if the elements are of pre-cast construction. Given that the Phase 1 samples were saturated when they were removed from the curing tanks and dried for three weeks it is not anticipated that the error induced will be extremely high. However, this was investigated in the Phase 2 testing as will be discussed in Chapter 5, through testing both soaked and unsoaked specimens for OPC, PFA and GGBS.

Considering Figure 4.40 and Figure 4.41 in terms of the PFA relative merits comparison it can be seen that the Phase 1 PFA relative merits are less than those found in the literature for marine exposure sites. The PFA relative merit values plotted range from 1.0 to 10.9. The Phase 1 slab and cube relative merits are the two lowest values at 1.0 and 1.6, respectively. It is worth nothing

however that the Thomas and Bamforth one year result is close to that of the Phase 1 cube samples at 1.7. It is also interesting to note that the three other studies plotted examine samples which were exposed in either tidal zone conditions or fully saturated conditions. These conditions would represent less drying than that experienced in the spray zone and in the salt fog chambers, an interesting point in the context of the discussion presented in Section 4.4.6.

It is also possible that the poor relative performance of the PFA concrete in the Phase 1 testing is related to the SCC nature of the concrete. All of the other studies plotted in Figure 4.41 examine normal concrete which typically has a larger coarse aggregate content and a lower cement paste volume per volume of concrete, as discussed in Chapter 3. Furthermore it has been established that the pore structure of SCC might be somewhat different from that of traditional concrete (Boel et al. 2002; Rilem Technical Committee et al. 2008). The Phase 2 testing regime will examine the chloride resistance properties of OPC, PFA and GGBS for SCC and VC of the same strength grade to further investigate this issue. The time dependent nature of the diffusion coefficient is also likely to have reduced the relative merit of the Phase 1 PFA SCC durability option in comparison with the long term studies. The results presented from Thomas and Bamforth (1999) in Figure 4.41 indicate the longer the exposure period the greater the PFA relative merit. The Phase 1 test duration is the shortest of the studies plotted in Figure 4.40 and Figure 4.41. Again it is noted that the time dependant diffusion effects will be incorporated in the probabilistic model in Chapter 6 and Chapter 7.

Figure 4.41 illustrates that the performance of the OPC-S durability option was far poorer in the Phase 1 testing than was observed on site by Schueremans et al. (2007). There are however a number of important differences between the two studies. The Schueremans et al. study involved the application of two coats of silane one week apart while the current study only utilised a single coating. In addition the Schueremans et al. study involved sampling of treated concrete at one pier location facing West while sampling of untreated concrete was carried out at a different quay wall location which faced North. The location and orientation differences would have induced considerable variation in the study due to spatial variability. It must also be noted that the COV value associated with the treated and untreated results was 0.85 and 1.55 respectively signifying a high level of variation in the results. The difference in the efficiency of silane surface treatment between the two studies could also be due to differing moisture conditions at the time of silane application. It has been found that the degree of saturation of the concrete sub-strata upon application effects the depth of penetration of the silane with a near saturated condition reducing silane performance (Basheer et al. 1998). This would appear to be a drawback associated with this

durability option as the moisture condition on the day of silane application can only be controlled to a limited extent due to the weather effects.

The next analysis, summarised below in Table 4.8 and Figure 4.42, compares the Phase 1 results to laboratory based experiments published in the literature which do not utilise a current to accelerate the movement of chlorides during testing. As discussed in Chapter 2 and 3 such studies are less common than studies which utilise an impressed current to accelerate the chloride movement. Again the x-axis of the dot plot presents the source study with letters indicating the type of exposure. The (a) group denotes the Phase 1 tests, the (b) group denotes the data from salt fog/spray chambers (SSC), the (c) group denotes data from ponding tests (P) the (d) group denotes bulk diffusion testing of SCC (BD), and the (e) group denotes data from bulk diffusion test and submerged sample testing for VC (BD, Sub).

Table 4.8 Relative merit comparison of Phase 1 tests and published laboratory studies examining natural chloride movement (no impressed current applied)

Exposure	Exposure time (years)	Concrete mix			Relative merit		Notes	Reference
		Binder	w/b	Concrete Type				
					Cubes	Slabs		
Phase 1 testing: Salt fog chambers	0.7	OPC	0.44	SCC	1.0	1.0		Current Study
	0.7	30% PFA	0.44	SCC	1.6	1.0		
	0.7	60% GGBS	0.44	SCC	1.9	2.3		
	0.7	OPC-S	0.44	SCC	1.8	1.8		
Salt Spray	0.4	OPC	0.58	VC	1.0			(Mangat and Gurusamy 1987)
	0.4	30% PFA	0.58	VC	0.5			
				VC				
Salt Spray	1.0	OPC	0.45	VC	1.0		k value used w/(c+k*PFA)=0.45 . Water soluble chlorides examined.	(Dhir et al. 1994)
	1.0	30% PFA	0.35	VC	1.5			
Salt Spray	0.1	OPC		VC	1		Results expressed only as mass ratio of chloride to concrete, short duration tests	(Leung et al. 2008)
	0.1	Silane		VC	12.5			
				VC				
Ponding	0.9	OPC	0.5	VC	1			(McPolin et al. 2005)
	0.9	30% PFA	0.5	VC	2.1			
	0.9	50% GGBS	0.5	VC	3.0			
Bulk Diffusion Tests	0.14	OPC	0.40	SCC	1.0		50 day bulk diffusion is a short duration test. SCC and VC mixes have different w/b ratio. Unknown % of GGBS, 36-65%.	(Loser et al. 2010)
	0.14	20% PFA	0.40	SCC	1.3			
	0.14	36-65% GGBS	0.40	SCC	3.7			
	0.14	OPC	0.45	VC	1.0			
	0.14	20% PFA	0.45	VC	1.2			
	0.14	36-65% GGBS	0.45	VC	2.6			
Submerged Samples	0.5	OPC	0.57	VC	1.0		Samples submerged in 5M NaCl for 6 months. K value used	(Dhir et al. 1993)
	0.5	35% PFA	0.45	VC	3.75			
Bulk Diffusion Tests	0.5, 1.0, 4.0	OPC	0.5	VC	1.0, 1.0, 1.0		Submerged for varying exposure times 3 months after curing	(Stanish and Thomas 2003)
	0.5, 1.0,	25% PFA	0.5	VC	4.0, 6.2, 5.7			

	4.0						
Bulk Diffusion Tests		OPC	0.30	VC	1.0		(Elahi et al. 2010)
		20% PFA	0.30	VC	2.4		
		40% PFA	0.30	VC	2.1		
		50% GGBS	0.30	VC	3.1		
		70% GGBS	0.30	VC	3.4		
Ponding test	1.0	OPC	0.50	VC	1.0	Based on Cl content at 18mm	(de Vries and Polder 1995)
	1.0	OPC-S	0.50	VC	5.6		
Saline absorption test with intermittent oven drying	0.12	OPC	0.4	VC	1.0	Based on depth of chloride penetration. Tests	(McCarter 1996)
	0.12	OPC-S	0.4	VC	4.0		
Natural Diffusion cell test	_	OPC	0.45	VC	1.0	Tests carried out on Mortars. RM based on steady state diffusion coefficients	(Buenfeld and Zhang 1998)
	_	OPC-S	0.45	VC	5.2		
Ponding test	0.85	OPC	0.45	VC	1.0	Based on chloride content at 25mm (CR2 pre-conditioning: see paper)	(Basheer et al. 1998)
	0.85	OPC-S	0.45	VC	14.8		
Ponding test	up to 1. 23	OPC	_	_	1.0	40% silane applied. Results obtained from Basheer et al. full testing details not available	(Basheer et al. 1997; Smith 1986)
	up to 1.23	OPC-S	_	_	4.5	RM based on % reduction in chloride content at depths, RM range depends on depth considered.	
	up to 1.23	OPC-S	_	_	12.5		

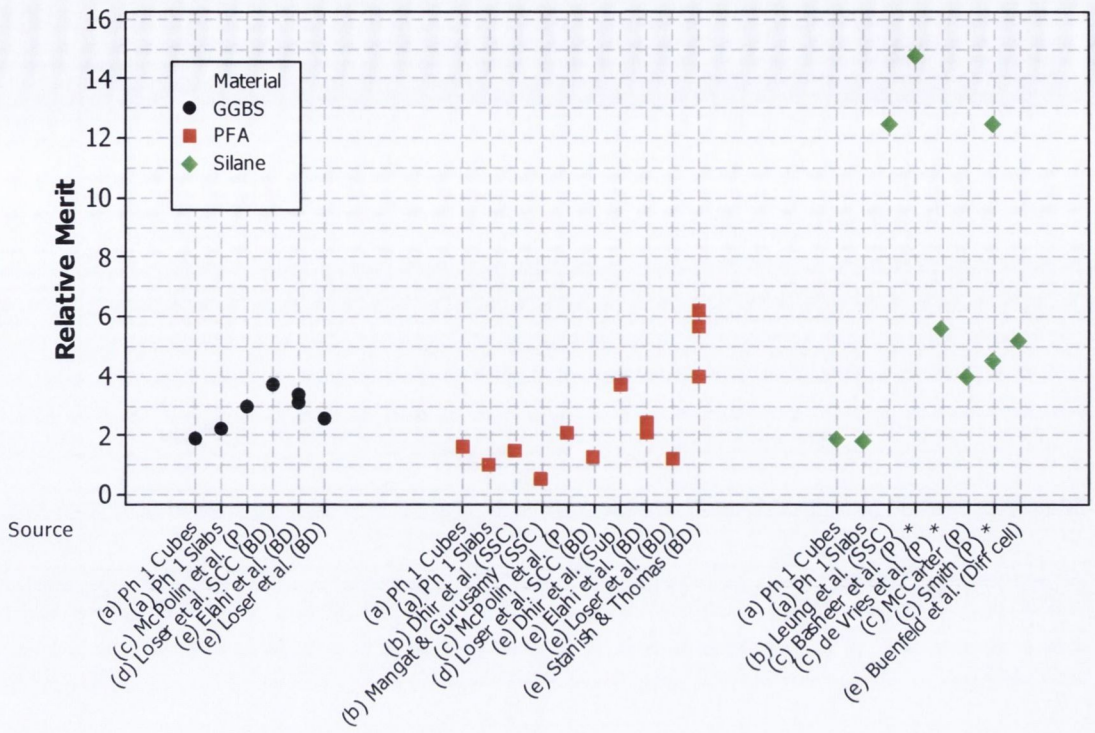


Figure 4.42 Dot plot of relative merits for Phase 1 testing and laboratory test results without current accelerated chloride movement

Upon examination of Figure 4.42 the most notable point is the highly variable performance of OPC-S in the literature, with relative merit values ranging from 1.8 to 14.8. As discussed in the context of Figure 4.41 above differences in the moisture content of the sub strata from study to study, differences in the nature of the silane application and the number of coats of silane utilised from study to study would all have contributed to variation in OPC-S relative performance across the literature. Some of the variation associated with silane performance may also be due to the fact that information on chloride ingress performance is limited in a number of papers to chloride content at a given depth. These studies are marked with an * in Figure 4.42.

From the plot it is also notable that the Phase 1 OPC-S relative merits are the lowest points plotted. However, it must be noted that the other plotted studies consider VCs only, with little or no information available in the literature relating to the effectiveness of silane treatment in SCCs. Differences in the pore structure and chemistry of SCC and VC are likely to affect the performance of the silane surface treatment in some way.

Having discussed the OPC-S results from the literature Figure 4.42 is plotted again without OPC-S relative merits greater than 7.0 in Figure 4.43 in order to facilitate examination of the OPC + PFA and OPC + GGBS results on a more appropriate y-axis scale.

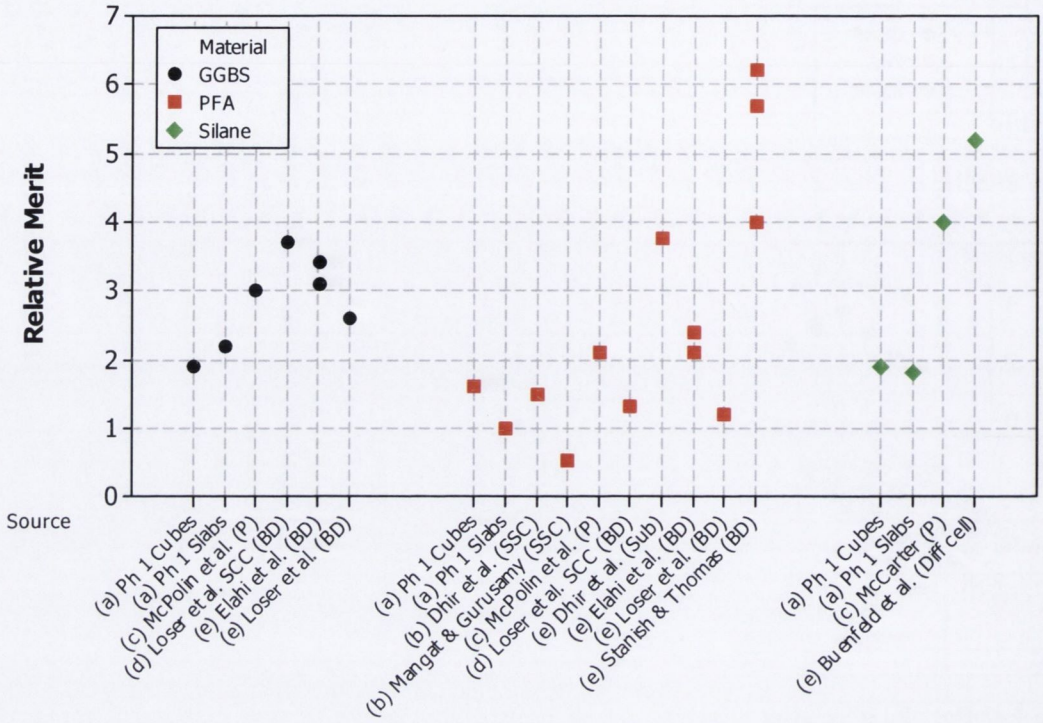


Figure 4.43 Dot plot of relative merits for Phase 1 testing and laboratory test results without current accelerated chloride movement in relative merit range 0-7.0

The GGBS relative merits values in Figure 4.43 range from 1.9 to 3.7. The relative merits of the Phase 1 GGBS samples are lower than the relative merits identified in three other studies plotted above. It is noted however that the three of these studies examine VCs in a fully saturated state. As discussed in Chapter 3, the work of Ben Fraj et al. (2012) indicated that the relative performance of OPC + GGBS concrete reduced with increased drying during exposure. This is the likely reason for the Phase 1 OPC + GGBS results being the lowest of those plotted. This issue is explored in detail in Chapter 5. The SCC tested by Loser et al. (2010) represents the highest relative merit. This result was obtained from short duration bulk diffusion test. The percentage of GGBS replacement was somewhere between 36% and 64% for this study. Finally in terms of GGBS it is noted that the relative merits obtained from the laboratory based experiments are lower than those obtained from the long term studies examining marine exposure sites and real structures. This is likely to be due to the time dependant diffusion effects, further highlighting the need to incorporate time dependant diffusion effects in the probabilistic modelling in Chapter 6 and Chapter 7.

When considering the PFA laboratory results comparison it can be noted that the reported performances are somewhat varied. The relative merits obtained from the literature range from 0.5 to 6.2. The Phase 1 laboratory test results can be seen to sit at the mid to lower end of the PFA

relative merit range. The Phase 1 PFA slab performance represents the second lowest relative merit plotted. The Phase 1 100mm cube PFA performance is of the same order of magnitude as were found by Dhir et al. (1994), Loser et al. (2010) and McPolin et al. (2005). The relative merits found by Stanish and Thomas (2003) seem high when compared to the other laboratory studies with values of 4.0 to 6.2. This heightened performance of the PFA concrete cannot be explained by the time dependant diffusion coefficient factor as the Stanish and Thomas relative merit values of 4.0, 6.2 and 5.7 were found by the authors after 0.5, 1.0 and 4 years of exposure respectively.

It is interesting to note that the relative merit obtained by Dhir et al. in 1994 using spray chamber testing was 1.5 while the relative merit obtained by Dhir et al. for PFA using submerged tests in 1993 was 3.75. It can also be seen from Table 4.8 that different k-values were used in the two studies. The 1994 study employed a k-value of 0.4 while the 1993 study employed a k-value of 0.7. It would thus be expected that the performance of the PFA concrete might be better in the 1994 study however this is not the case. This comparison would seem to highlight the influence of the testing method utilised, and specifically the reduction of PFA relative performance in testing incorporating a level of drying.

It is also interesting to note that the performance of the PFA concrete in the SCC and VC concrete in the Loser et al. study is similar with relative merit values of 1.2 and 1.3. This indicates that there is little difference between PFA performance for VC and SCC. This is however the only study which could be found in the published literature which compares the performance of OPC and concrete containing PFA as a direct cement replacement under natural chloride migration conditions. As highlighted in Table 4.8 however the w/b ratios for the VC and SCC are not the same in the Loser et al. study. As previously mentioned the issue of comparison of VCs and SCCs was further investigated in the Phase 2 testing and in the probabilistic modelling in Chapter 7.

The majority of laboratory based studies published in the literature deal with tests carried out with impressed current. As previously mentioned the popularity of the test is due to the ability to obtain results in under 48 hours, reducing experimental cost and the required time investment. As discussed in Chapter 2 and 3, there are a number of problems associated with these methods, however, the practice is widely utilised due to the speed of the tests. The two most popular forms of impressed current tests are the Rapid Chloride Permeability Test (RCPT) and the Rapid Chloride Migration Test (RCMT). For reasons already discussed the RCPT is not suitable for the comparison of SCC or concretes containing SCMs (Scanlon and Sherman 1995; Shi et al. 1998). Consequently, the RCPT results published in the literature will not be compared to the Phase 1

relative merits. Results from the RCMT type testing in the existing body of published literature will however be compared to the Phase 1 results in Table 4.9 and Figure 4.44.

Table 4.9: Relative merit comparison of Phase 1 tests and published laboratory studies utilising RCMT method or closely aligned methods

Exposure	Exposure time	Concrete mix			Relative merit		Notes	Reference
		Binder	w/b	Concrete Type				
					Cubes	Slabs		
Salt Spray	0.7 yrs	OPC	0.44	SCC	1.0	1.0	As discussed notably different relative merit obtained for PFA slabs and cubes	Current Study
	0.7 yrs	30% PFA	0.44	SCC	1.6	1.0		
	0.7 yrs	60% GGBS	0.44	SCC	1.9	2.3		
	0.7 yrs	OPC-S	0.44	SCC	1.8	1.8		
RCMT	24 hrs	OPC	0.40	SCC	1.0		SCC and VC mixes have different w/b ratio. Unknown % of GGBS, 36-65%.	(Loser et al. 2010)
	24 hrs	20% PFA	0.40	SCC	1.5			
	24 hrs	36-65% GGBS	0.40	SCC	14.7			
	24 hrs	OPC	0.45	VC	1.0			
	24 hrs	20% PFA	0.45	VC	1.0			
	24 hrs	36-65% GGBS	0.45	VC	5.6			
RCMT		OPC + 6-20% L	0.45	VC	1.0		CEM II-AL= OPC + 6-20% Limestone. Taken as control for this study.	(McNally and Sheils 2012)
		OPC + 6-20% PFA	0.45	VC	1.7			
		50% GGBS	0.45	VC	8.7		50% GGBS, 3-10% L	
		70% GGBS	0.45	VC	7.0		70% GGBS 2-6% L	
RCMT		OPC		VC	1.0			(Medeiros and Helene 2009)
		OPC + Silane		VC	1.1			
RCMT	24 hrs	OPC	0.46	SCC	1.0			(De Schutter and Audenaert 2004)
		36-64% GGBS	0.46	SCC	2.5			
		OPC	0.46	VC	1.0			
		36-64% GGBS	0.46	VC	1.21			
RCMT		OPC	0.46	SCC	1.0			(Audenaert et al. 2010)
		36-64% GGBS	0.46	SCC	3.3			
		OPC	0.46	VC	1.0			
		36-64% GGBS	0.46	VC	1.7			
RCMT	7 days	OPC	0.45	VC	1.0			(Yang and Cho 2003)
	11 days	20% PFA	0.45	VC	5.6			

L= Portland Limestone Cement

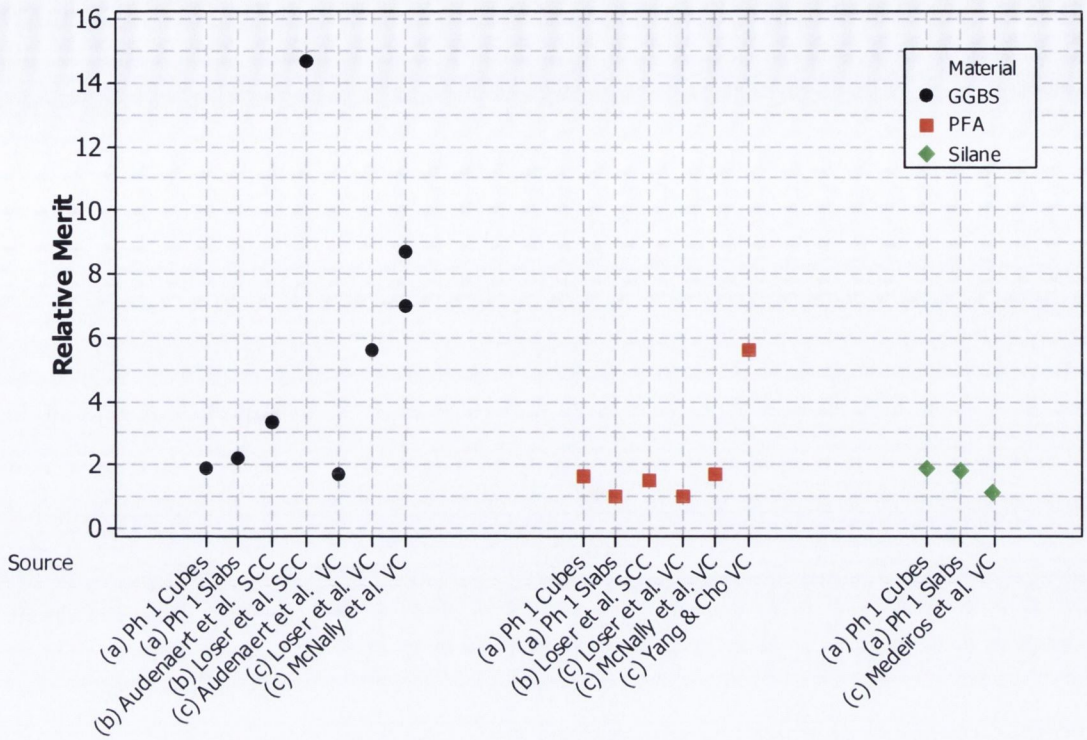


Figure 4.44 Dot plot of relative merits for Phase 1 testing and laboratory test results obtained utilising RCMT

As can be seen from Figure 4.44 the relative merits obtained for GGBS seem somewhat high with the exception of the VC tested by Audenaert et al. (2010), with values ranging from 3.3 to 14.7. This tendency to obtain higher relative merits for GGBS using the RCMT is highlighted by the Loser et al. (2010) study. Loser et al. tested SCCs and VCs using both natural diffusion testing and the RCMT, with the results reported in Table 4.1 and Table 4.9 respectively. For the GGBS SCC a relative merit of 14.7 was obtained using the RCMT while a relative merit of 3.7 was obtained using natural diffusion conditions. For the VC relative merits of 5.6 and 2.6 were obtained using the RCMT and the natural diffusion test respectively. In general, the RCMT GGBS relative merits appear greater than the relative merits obtained from natural chloride migration testing. The RCMT data is also illustrates higher GGBS relative merits than much of the real marine site data. In this comparison it must be noted that the RCMT data results presented have yet to have the time dependent diffusion coefficient incorporated. Thus comparison of the RCMT and real marine exposure GGBS relative merits on the same time scales would result in far greater RCMT GGBS relative merits. These findings are interesting in the context of the discussion in Section 3.4 of the literature review which showed that GGBS and OPC have only been compared to date using probabilistic methods which incorporate testing which utilising an impressed current (i.e. RCMT).

When considering the PFA relative merits it can be noted that the Phase 1 test results are in close agreement with the RCMT PFA results. The relative merits range from 1.0 to 5.6. Only one study could be found in the literature which examined a silane surface treatment utilising the RCMT. As can be seen from Figure 4.44 this study reported an OPC-S relative merit of 1.0.

4.4.8 Areas Highlighted for Further Research in Phase 2 Testing

As is the case with most experimental studies which undergo rigorous analysis, a number of issues have been raised from the Phase 1 testing. It was not possible to investigate all issues highlighted by the Phase 1 testing in the second phase of testing in this thesis due to environmental chamber space constraints and PhD time constraints. The issues raised in the Phase 1 testing which fall under one of the key PhD research questions, and thus were further examined in the Phase 2 testing programme are briefly discussed below. The basis for the Phase 2 testing is discussed in detail in Chapter 5.

Research Question 1:

- The effect of the silane application system used on the variability of the silane results will be examined by applying silane using a highly controlled method, designed to be consistent from sample to sample

- The effect of initial absorption on the experimental results will be investigated

Research Question 2:

- The effect of exposure conditions on the relative performance of concretes containing PFA and GGBS will be further investigated through testing OPC, PFA and GGBS slab and cube samples in the same salt fog chamber, under exposure conditions which differ from the two exposure conditions utilised in the Phase 1 testing

- The possibility of the effects of specimen size and casting mould type on this kind of testing will be further investigated through testing slab samples and cube samples for OPC, PFA and GGBS concretes in the same salt spray chamber

4.5 CONCLUSIONS

As discussed in detail in the literature review in Chapter 3, it is recognised that SCC is, at present being used in industry without insight into the intrinsic durability of the material itself (Rilem Technical Committee et al. 2008). Chapter 3 also presented the shortage of literature examining various SCC options under natural chloride migration conditions, and concerns in the literature surrounding the use of accelerated test conditions which utilise an impressed current (Loser et al. 2010; Rilem Technical Committee et al. 2008). These two factors combined to result in an urgent need for information on SCC options obtained from natural chloride migration testing. Based on this need Research Question 1 was developed as follows;

Research Question 1: How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

As stated in the introduction this key research question is addressed in this thesis through the following;

e) A comparison was carried out between the chloride transport properties of a number of different SCC options through natural chloride migration laboratory testing which incorporated wetting and drying cycles
f) The same experimental techniques were used to compare SCC to the more established traditional vibrated concrete type for both OPC concretes and concretes with SCMs used as partial cement replacements.

The work presented in this chapter has partially addressed Research Question 1 by comparing the chloride transport properties OPC SCC, OPC + PFA SCC, OPC + GGBS SCC and OPC + silane surface treatment i.e. achieving part a) listed above.

Comparison of the chloride transport properties from two different salt fog chambers identified the GGBS SCC as the most durable SCC option, with a chloride resistance performance of between 1.9 and 2.3 times that of the OPC SCC. The OPC + silane SCC was the second best performing option with chloride resistance 1.8 times better than the OPC SCC. It was also found that the addition of corrosion inhibitor had little effect on the chloride transport properties of the OPC concrete, contradicting the findings of Lou and Schutter (2008).

A subsidiary Phase 1 experimental aim was to carry out an explorative study to assess the possible effects of utilising salt fog chamber testing on different scales. It was found that, even though the chambers were programed identically, a far greater level of drying took place in the larger SKWT 4300 chamber. Comparison of the results from the two salt fog chambers, and consideration of the existing literature relating to absorption characteristics of different concretes, it would seem likely that the differences in the RH conditions had a significant influence on the chloride resistance performance of the PFA SCC.

This finding, combined with a) the review of the existing literature examining the effects exposure condition on relative performance discussed in Chapter 3 and b) the fact that the majority of information relating to the comparative performance of OPC concrete and OPC + PFA concrete is obtained from testing in fully saturated conditions, lead to the development of Research Question 2, presented in Chapter 1, and listed again below;

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

The work carried out to address this research question is discussed in detail in Chapter 5.

The Phase 1 salt fog chamber results were compared to the concrete durability comparative studies in the published literature. Reasonably good agreement between the relative merits obtained in testing and those published in the literature were identified however the following points were highlighted;

- Studies of longer durations tended to have increased relative performance for concretes containing GGBS and PFA. This is thought to be primarily due to the effects of the time dependent diffusion coefficient.
- Natural chloride migration studies by Dhir et al. (Dhir et al. 1993; Dhir et al. 1994) showed improved performance of PFA concrete tested in a fully saturated state, when compared to results from a salt spray chamber test. This is in agreement with reasoning presented for the observed change in the PFA SCC relative performance from slabs to cube specimens in the Phase 1 testing.
- Literature comparisons indicated that the RCMT method may elevate the relative performance of concretes containing GGBS.

- Examination of the OPC-S results in the literature highlighted the fact that the performance of silane is highly variable and is likely to be dependent upon the moisture condition of the concrete upon application and the method of application of the silane.

5. PHASE 2 EXPERIMENTAL METHODOLOGY AND RESULTS

5.1 INTRODUCTION

The phase 2 experimental programme was primarily concerned with addressing Research Questions 1 and 2. In addition experimental work was undertaken which will allow Research Question 3 to be addressed in Chapter 7, through probabilistic modelling. As discussed in detail in Chapter 3 and in Chapter 1, the three key PhD research questions were developed based on shortages in the existing literature and are presented below again for the sake of clarity;

Research Question 1: How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

Research Question 3: How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

As discussed in Chapter 3, addressing Research Question 1 requires that; a) different SCCs are compared to one another, and b) the performance of SCCs and equivalent vibrated concretes are compared. The work described in Chapter 4, focused on part a) by examining the chloride transport properties of five different SCC options. The Phase 2 testing further investigates Research Question 1 part a) by examining the following areas which were highlighted for further investigation in Section 4.4.8 of Chapter 4;

- Examine the effect of initial absorption on the results of salt fog/spray chamber tests
- Investigate the effect of the silane application method adopted on the variability of the silane performance results i.e. how susceptible is silane performance to the manner in which it is applied

Part a) of Research Question 1 is also further addressed in this chapter by examining a PFA SCC with the *k*-value factor applied. As previously discussed, the *k*-value concept, specified in EN 206-1:2000, specifies that only 40% of the PFA content utilised as an OPC 42.5 strength class replacement in a structural element, can be used in the calculation of the w/b ratio of the concrete.

This parameter was primarily estimated based on compressive strength characteristics (Bentur and Mitchell 2008; Papadakis and Tsimas 2002; Sanjuan et al. 2011). Examination of the effect of the *k*-value concept under natural diffusion conditions are uncommon in the literature (Dhir and Byars 1993; Dhir et al. 1993; Dhir et al. 1994; Thomas and Bamforth 1999). As shown in Chapter 3, examination of the *k*-value concept, with $k = 0.40$ for 42.5 OPC strength class as per the relevant standard, has not been investigated in terms of SCC marine durability using appropriate testing methods. However, given that OPC + PFA concrete is one of the SCCs which makes up the exploration of SCC under Research Question 1, and the *k*-value's specification in EN 206-1:2000, the effects of the *k*-value efficiency factor has been explored in the Phase 2 experimental testing in terms of compressive strength, absorption characteristics and chloride transportation characteristics.

Finally in terms of Research Question 1, the Phase 2 testing described in this chapter address part b) of Research Question 1 by comparing the transport properties of the SCC to equivalent VC mixes using natural chloride migration testing in the form of the salt fog chamber tests. The supplementary cementing materials, namely PFA and GGBS, were utilised as OPC replacements rather than filler materials herein.

In order to address Research Question 2 in this chapter using both the Phase 1 and Phase 2 test results two separate issues had to be explored. From the Phase 1 results it was noted that the PFA SCC had a significantly lower relative performance in the SKWT 4300 chamber than in the SC1000 chamber. As discussed in Chapter 4, however, it was not possible to separate climactic effects from the effect of specimen sizing and the concrete mould type used. Having considered relevant studies in the literature it was, however, tentatively stated that the reduced PFA SCC performance in the SKWT 4300 was most likely due to the difference in climactic conditions, with a higher level of drying taking place in the SKWT 4300 chamber. The effect of exposure conditions on the relative durability performance of different concretes using both the Phase 1 and Phase 2 test results were investigated to address Research Question 2. Two experimental investigations were therefore carried out in this chapter; a) examine the combination of specimen size effect and casting mould effect on the relative performance of the OPC, OPC + PFA and OPC + GGBS SCCs, and b) investigate the effect of exposure conditions on the relative performance of OPC, OPC + PFA and OPC + GGBS SCCs. These areas were explored in this chapter utilising a combination of absorption and sorptivity tests, drying tests, and chloride transport properties from three different exposure conditions.

Finally, it should be noted that the work described in this chapter does not address Research Question 3 directly. Instead testing is carried out on specimens which facilitate investigation into calibration of salt fog chamber testing using real marine environment data in Chapter 7. A number of attempts have been made by researchers to calibrate and compare salt spray/fog chamber test results with results from the real marine environment (Li 2000; Mangat and Gurusamy 1987; Mangat and Gurusamy 1987; Mangat and Molloy 1995). However, each of these attempts has failed in one way or another for a variety of reasons. Consequently, after the most recent attempt at salt fog/spray chamber calibration by Li, the author stated that the applicability of data obtained from accelerated conditions in environmental chambers to conditions typical of service, is a significant issue that requires the continuous efforts of researchers (Li 2000). The calibration attempt in Chapter 7 utilises real marine data collected from the pre-repaired Ferrycarrig Bridge in 2007 by O'Connor and Kenshel (2013). The work described in this chapter which aids in this calibration involves the salt fog chamber testing of cube samples constructed to mimic the concrete used in the original Ferrycarrig Bridge structure in 1980. The details of this mix are provided in Section 5.2.2.

It is clear that the Phase 2 experimental programme had a more detailed and specific brief than the Phase 1 experimental programme, with a significant number of aims to be achieved in Phase 2, each of which falls one of the three core PhD research questions. To facilitate the clear illustration of how the Phase 2 experimental testing addressed the three core PhD research questions in the manner discussed above, seven separate Phase 2 experimental aims have been identified. These Phase 2 experimental aims are listed below under their relevant research questions;

Research Question 1

Part a):

Aim 1: Examine the effect of initial absorption on the results of salt fog/spray chamber tests

Aim 2: Investigate the effect of the silane application method adopted on the variability of the silane performance results i.e. how susceptible is silane performance to the manner in which it is applied

Aim 3: Investigate the chloride transport properties of OPC + PFA SCC with and without the *k*-value concept applied

Part b)

Aim 4: Compare the chloride transport properties of SCC options to equivalent VC options

Research Question 2:

Aim 5: Investigate the combination of specimen size effect and casting mould effect on the relative performance of the OPC, PFA and GGBS.

Aim 6: Investigate the effect of exposure conditions on the relative merits of alternative SCC durability options

Research Question 3:

Aim 7: Carry out salt fog chamber testing to facilitate a calibration of the test results with data from a real marine structure

Having carried out a comprehensive comparison of the different SCCs in Chapter 4, it is noted that the two most important Phase 2 experimental aims in the context of the core PhD research questions are Phase 2 experimental aim No. 4 and Phase 2 experimental aim 6.

The first section of this chapter presents the Phase 2 experimental design in the context of the established experimental aims. This discussion details the methodology utilised to achieve each experimental aim. This is done through the presentation of three separate sub-sections, each of which deals with a different aspect of the Phase 2 experimental design. The first sub-section describes the necessary changes implemented in the Phase 1 experimental programme and the additional testing methods adopted to facilitate achievement of specific experimental aims. The second sub-section presents the Phase 2 mix design details and explains how the concrete mixes examined allowed for the investigation of experimental aims. The third sub-section details the experimental aims addressed through changes to concrete specimen preparation process.

Having presented the details of the Phase 2 testing, the Phase 2 experimental results are presented in five separate sections. The first section is concerned with the classification of the concretes examined through presentation of workability results and compressive strength test results. The second section presents the results of a number of ancillary tests which were primarily carried out to aid in addressing Research Question 2 through examination of absorption characteristics, drying characteristics and humidity test data. The third results section presents the salt fog chamber results used to address Research Question 2. Subsequently, the next section presents the salt fog chamber results which address Research Question 1. The final results section discusses the salt fog chamber results which relate to Research Question 3. Each of the sections also includes comparison of the Phase 2 experimental results with results from the existing literature where possible. The last section of the chapter provides a final discussion on the most relevant research findings of the

Phase 2 experimental programme in the context of the Phase 2 experimental aims and the three key PhD research questions.

5.2 PHASE 2 EXPERIMENTAL DESIGN AND METHODOLOGY

5.2.1 Phase 2 Testing Methodology

Chapter 4 provided extensive details of many of the experimental testing methods utilised in this thesis. In order to avoid repetition, only details of the Phase 2 experimental programme which vary from the methods utilised in the Phase 1 testing will be provided in this chapter. These details will be presented in this sub-section with each change in procedure linked to its related Phase 2 experimental aim numbered 1 to 7 above.

Firstly, considering the salt fog chamber testing, it should be noted that for the Phase 2 experimental programme all test samples were exposed to the accelerated marine environment in the larger, SKWT 4300, salt fog chamber. This facilitated performance comparison for slab and cube samples for OPC, OPC + PFA and OPC + GGBS SCC without the influence of the environmental differences between chambers. This measure allowed Phase 2 testing aim No. 5 to be achieved. The SKWT 4300 chamber with both the slab and cube samples in position is shown before in Figure 5.1.

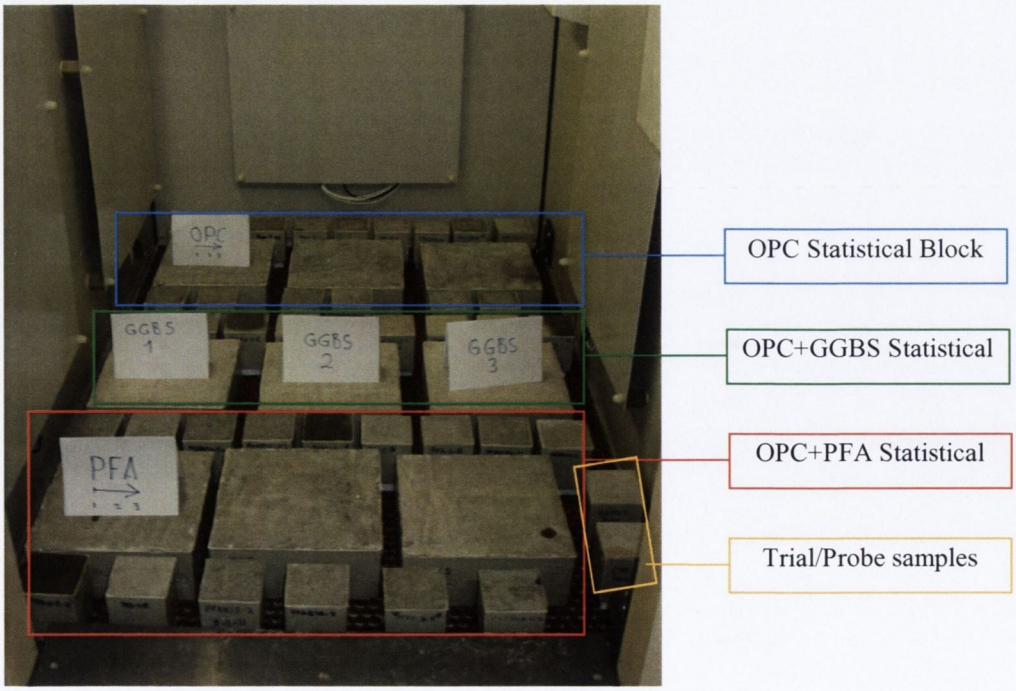


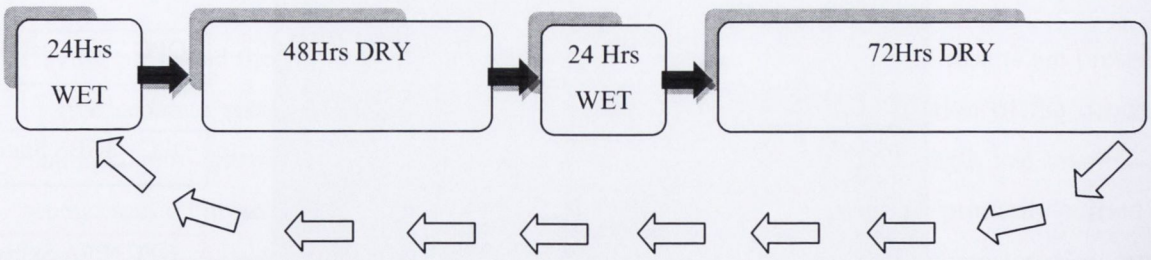
Figure 5.1 SKWT 4300 chamber with test specimens in position

The labels in Figure 5.1 mark the statistical blocks that were used in the Phase 2 testing. The OPC slabs and cubes, labelled in blue, consisted of one statistical block. The OPC + PFA slabs and cubes, labelled in red, consisted of another statistical block. The final statistical block comprised of the OPC + GGBS slabs and cubes. The probe samples were included in the chamber to assess the depth of chloride penetration at an intermediate point in the testing programme. They remained in the same position throughout tests and thus do not constitute a statistical block. The statistical block rotation system described in Chapter 4 was again utilised for the Phase 2 testing to reduce the effects of any spatial variability on the experimental results.

In order to achieve Phase 2 Experimental Aim No. 6, which is concerned with examination of the effect of varying climactic conditions on relative performance, the cyclic exposure in the Phase 2 testing was altered from that used in the Phase 1 testing. The analysis in Chapter 4 identified the effect of exposure conditions as the most likely cause of change in the PFA SCC relative performance. In order to further investigate this via salt fog chamber testing the level of drying in the SKWT 4300 chamber was reduced in the Phase 2 testing. This would allow the relative performance of the durability options to be investigated in three different sets of climactic conditions; Phase 1 SKWT 4300 chamber, Phase 1 SC 1000 chamber and Phase 2 SKWT 4300 chamber.

The level of drying in the SKWT 4300 chamber was reduced in two ways. Firstly, the cyclic wetting and drying was changed as per Figure 5.2 below:

Phase 1 Wetting and Drying Cycle:



Phase 2 Wetting and Drying Cycle:

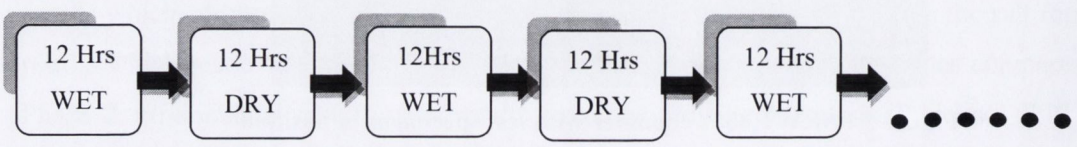
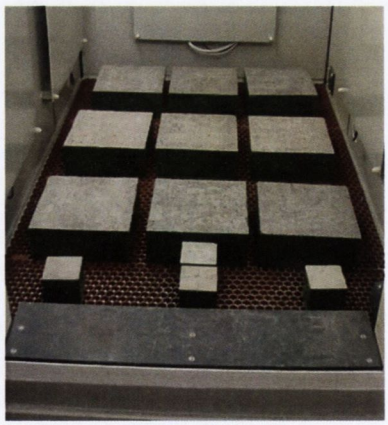


Figure 5.2: Schematic of Phase 1 and Phase 2 drying cycles

The implementation of this shorter, 12 hours on 12 hours off salt solution fog emission meant that less drying would occur over the duration of the exposure period. This created climactic conditions which resulted in less drying than both the chambers in the Phase 1 testing. The level of drying in the Phase 2 tests was also reduced by changing the fan type used to blow air through the chamber during the drying period. The fan was changed from the 75 watt fan used in the Phase 1 testing, to a smaller 12 watt computer fan.

The extent to which the chamber drying power was altered by the fan change was visually observed by placing the Phase 1 samples in the salt fog chamber once the Phase 1 sample analysis was complete. The samples were subjected to a 12 hour wetting cycle followed by a drying cycle, using different fans. The series of pictures in Figure 5.3 serve as a quick and easy qualitative reference as to the effect of the fan change. It should be noted that due to the fact that each of the tested samples had a hole in the top face from profile grinding, the bottom face of the samples faced up during this test. Prior to this investigation this bottom face was angle grinded to remove the sealant paint. The process resulted in the surface being somewhat uneven which caused some ponding during this drying test. This is particularly evident in the slab sample in the bottom left corner of the photographs. Consequently, the drying of each sample can only be compared to the equivalent sample for both fan conditions. Comparison of the drying capacity of the different samples in each photograph should not be considered. Yet, a different test was devised to assess the drying ability of the various concrete types, as will be described later in this sub-section.



(a) Samples prior to wet cycle

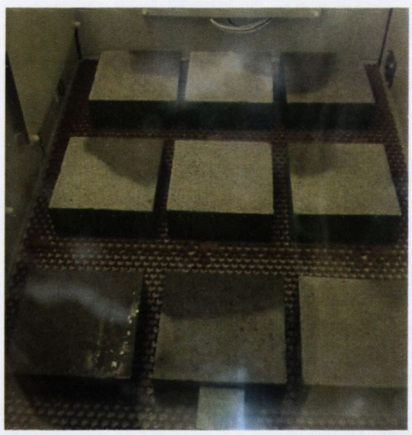
(b) 75 Watt fan 15 minutes drying

(c) 12 watt fan 18 minutes drying (chamber not clear @ 15 mins)

(d) 75 Watt fan 1 hour drying

(e) 12 watt fan 1 hour

(f) 75 watt fan 2 hours



(g) 12 watt fan 2 hours

Figure 5.3 Effect of fan change for Phase 2 testing

Changing the strength of the air exchange system in the Phase 2 testing was expected to have two effects. Firstly, as shown in the figures above, the level of drying in the chamber was significantly reduced. Secondly, any spatial variability associated with the sample drying will have been reduced somewhat. A less powerful fan should mean that there is a smaller difference between the air flow

over the sample under the fan inlet, and the sample furthest from the fan inlet. As described above however, in the context of Figure 5.1, statistical blocking implemented to reduce any spatial variability that remained. It is noted that in the context of the examination of climactic effects using the Phase 1 & 2 results, the size effect and casting mould type effect would remain unknown from the Phase 1 results. The examination of the combined size effect and casting mould effect will be discussed in Section 5.2.3.

In addition to the changes to the salt fog chamber testing procedure, three additional tests were also introduced in the Phase 2 experimental programme in order to assess the properties of hardened concrete. These new tests were selected to help address Research Question 2 through achieving Phase 2 Experimental Aim No. 6 (effect of exposure conditions on relative performance). The first of the additional tests was the concrete absorption test which was carried out in accordance with BS 1881-122:1983 and ASTM C 642-06 (ASTM 2006). This test involves oven drying concrete samples for 72 hours or to a constant mass of (< 0.5% weight change in 24 hours), and then cooling the samples in a desiccator for 24 hours. The samples are then weighed and immersed in water. The weight of water absorbed by each sample is then recorded at 30 minutes and 24 hours by weighing the samples. The test set-up for the absorption test can be seen below in Figure 5.4. This test was carried out for each of the eight concrete mixes which were utilised in the Phase 2 testing after the 28 sample day curing period.

Figure 5.4: Absorption test set-up

The second additional test utilised in the Phase 2 experimental programme was the sorptivity test. This test determines the rate of water absorption through the concrete surface (Zhu and Bartos 2003). The test procedure was adapted from the RILEM testing recommendations for testing the characteristics of stone (RILEM TC 1980). The test has been used by a number of researchers to investigate the sorptivity coefficient for VCs, (Chan and Ji 1998) and more recently, for SCCs

(Gesoglu et al. 2009; Guneyisi et al. 2011; Sahmaran et al. 2009; Zhu and Bartos 2003). The test involved drying 100mm cube samples to a constant mass and then cooling the samples in a desiccator for 24 hours. The samples were then painted with a silicone type sealant on four faces with two opposite mould faces remaining unpainted. After the silicone dried in the desiccator the samples were placed in a container sitting on steel angle sections with one of the unpainted faces immersed in water up to a depth of 2mm. The container was covered and the water level was kept constant in the tank. The samples were weighed at intervals to obtain the water absorbed by the immersed surface. Measurements were taken at 1, 3, 5, 10, 15, 30, 45, 60, 90 and 120 minutes. The recorded change in mass of the sample is then converted to the unit of kg/m^2 giving the absorption per m^2 of surface area immersed in the water. These figures are then plotted against the square root of time to obtain the absorption coefficient in accordance with the relevant standard as will be discussed in more detail in Section 5.5.2 (RILEM TC 1980). The test set-up for the sorptivity test can be seen below in Figure 5.5. The sorptivity test was carried out for each of the concrete mixes utilised in the Phase 2 testing after the 28 sample day curing period.

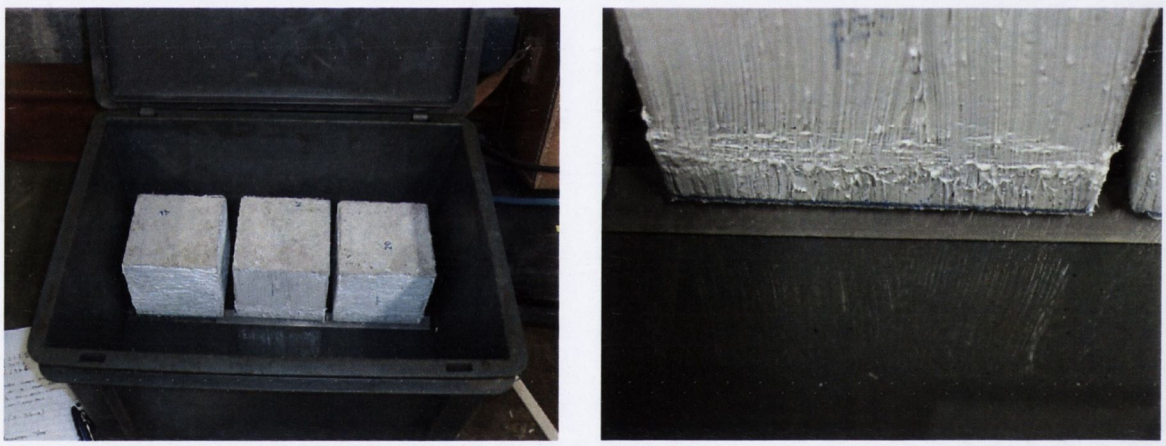


Figure 5.5: Sorptivity test set-up

The third additional test which was used in the Phase 2 programme was a concrete drying test. This test was only carried out for concretes tested in the Phase 1 experimental programme namely, OPC SCC, OPC + PFA SCC, and OPC + GGBS SCC. The test was designed to help address Research Question 2 by aiding in identifying the cause of the poorer OPC + PFA SCC performance in the SKWT 4300 chamber from the first phase of testing. It is recognised that structures subject to wetting and drying cycles are particularly vulnerable to high chloride uptake (Richardson 2002). Neville (1995) stated that the ingress of chlorides into concrete in the marine environment is strongly influenced by the sequence of wetting and drying, with the extent of movement of chlorides depending on the length of the wetting and drying periods. It is clear therefore, that the

ability of concrete to absorb water rich in chlorides will depend on the moisture condition of the concrete. Consequently, for a given wetting and drying cycle, concretes which dry out faster between chloride solution wetting cycles will be more prone to the absorption of chlorides in the outer millimetres of concrete cover upon commencement of a wetting cycle. This wet cycle could be brought about by high winds carrying splash water or by rising tide on a bridge pier. The absorption characteristics of different concrete types have been investigated in numerous publications in the literature which examine the durability characteristics of different concretes (Assié et al. 2007; Guneyisi et al. 2011; McPolin et al. 2005; Sahmaran et al. 2009; Sonebi and Nanukuttan 2009; Zhu and Bartos 2003). The drying characteristics of various concrete options are, however, rarely considered in the context of the marine durability of different concretes. In order to address this point, and gather information which would aid the assessment of the performance of concretes in different exposure conditions, a drying test was carried out herein.

The procedure for the drying tests carried out as part of the Phase 2 testing was quite simple. The samples were soaked in a curing tank for 3 days, then dried with a paper towel and weighed. Three duplicate specimens were tested for each of the three SCC concretes investigated. The samples were placed on timber sheeting which had a 100mm grid marked out. This grid allowed for 100mm between each of the cubes in every direction to facilitate uniform drying. The order of the samples on the grid was arranged so that each row and column contained one of each sample type tested. The timber sheeting was then placed in the centre of the salt fog chamber floor with the chamber programed to run on a dry cycle. The samples were removed at intervals and weighed. The test set-up can be seen below in Figure 5.6.

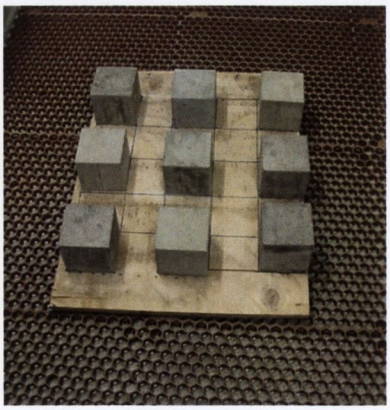


Figure 5.6 Drying test set-up

The last change in testing of hardened concrete from Phase 1 to in the Phase 2 relates to the concrete compressive strength testing. Again the tests were carried out in accordance with BS EN

12390-3:2009, however, for the Phase 2 experimental programme compressive strength testing was carried out at 7, 28 and 56 days maturity (Centre Europeen de Normalisation CEN 2009).

Finally, it is noted that in addition to the changes in the procedures of testing hardened concrete a number of additional fresh concrete tests were carried out in the Phase 2 experimental programme to characterise the SCCs investigated herein in line with the new suite of EN Standards relating to SCC, which were released in 2010 (Centre Europeen de Normalisation CEN 2010; Centre Europeen de Normalisation CEN 2010; Centre Europeen de Normalisation CEN 2010; Centre Europeen de Normalisation CEN 2010).The testing procedures and results are presented in Section 5.4.

5.2.2 Details of Concrete Mixes Examined

The Phase 1 experimental programme saw the investigation of four different SCC mixes and an OPC SCC with silane surface treatment. The Phase 2 experimental programme investigated 8 different concrete mixes consisting of both VC and SCC in addition to the OPC SCC with silane surface treatment durability option. Each mix investigated contributed to addressing the core PhD research questions though the achievement of the Phase 2 experimental aims outlined in Section 5.1. This sub section will discuss each of the concrete mixes, or set of mixes, under the heading of the Phase 2 experimental aim which they primarily contributed to.

It should be noted at this point that the only concrete mix which was examined in the Phase 1 tests, which was not examined in the Phase 2 experimental programme, was the OPC SCC with mixed-in corrosion inhibitor. It was omitted from the Phase 2 testing as; a) the lack of reinforcement in the concrete specimens limited the ability to fully assess the durability option and b) the Phase 1 test results indicated that the addition of the mixed in corrosion inhibitor did not notably effect the chloride transport properties of the concrete.

Phase 2 Experimental Aim No. 6: Effect of different exposure conditions (Research Question 2)

Group: SCC mixes

Mixes: OPC SCC, PFA SCC and GGBS SCC

As previously mentioned this experimental aim is achieved through testing the SCC mixes which were examined in the Phase 1 experimental programme using a different wetting and drying sequencing. The Phase 1 and Phase 2 results will then be compared with particular attention paid to the PFA performance. The SCC mixes utilised for this purpose are OPC (OPC SCC), OPC + 60%

GGBS (GGBS SCC) and OPC + 30% PFA (PFA SCC). The terms in brackets will be used to refer to the mixes from this point forward. For each of these mixes salt fog chamber testing was conducted on three 300mm x 300mm x 120mm slab samples and three 100mm cube samples. The mix-design details and materials used in the Phase 2 testing for these mixes were identical to those used in the Phase 1 testing. The mix designs are included in Table 5.1 at the end of this sub-section together with all the Phase 2 mix-designs for ease of reference. Details of the workability tests and results for the SCCs, together with workability testing details for all the concretes, are provided in Section 5.3.

Phase 2 Experimental Aim No. 4: SCC vs. VC (Research Question 1)

Group: VC mixes and SCC mixes

Mixes: OPC VC, PFA (k) VC and GGBS VC and SCC mixes from above

As discussed in Section 5.1, the Phase 2 experimental programme seeks to address Research Question 1, part (b) by comparing the transport properties of the SCC's detailed above to equivalent VC mixes, using natural chloride migration testing in the form of the salt fog chamber tests. The supplementary cementing materials, namely PFA and GGBS, are utilised as OPC replacements herein, rather than filler materials. The nature of the replacement is in accordance with EN 206-1: 2000 meaning that GGBS is used as a direct replacement for cement while PFA is used with the relevant *k*-value efficiency factor provided in the code.

Having discussed the mix designs for the SCC in detail in Chapter 4, the VC mix designs will now be discussed. The VC mix designs utilised for the Phase 2 testing were again based on a real marine structure. The VC mix design was provided by CEMEX Ireland who used the concrete mix in the construction of permanently submerged reinforced concrete bridge piers on the east coast of Ireland between February 2009 and February 2011. Hence, the bridge piers fall within the XS2 exposure class specified in EN 206-1:2000. CEMEX were not at liberty to state which bridges the mix was used on or cite the exact dates of construction.

The mix design provided was for an OPC + 70% GGBS with GGBS used as a direct OPC replacement. The design was for a 50N/mm^2 strength class, the same as the SCC mixes. The w/b ratio was 0.44, identical to the SCC w/b ratio used at Ferrycarrig Bridge. The mix had a target slump of 150mm. To facilitate comparison with the SCC the mix was adjusted to produce an OPC mix, an OPC + 60% GGBS mix and an OPC + PFA mix with the *k*-value incorporated. Due to space limitations in the SKWT 4300 chamber no more than 48 test specimens could be accommodated in the Phase 2 testing. This space limitation meant that the PFA vibrated concrete,

with, and without the *k*-value concept applied, could not both be accommodated in the Phase 2 testing. Thus, only one of these parameters could be examined and subsequently used in the SCC vs. VC comparison. As the implementation of the *k*-value is required in order to comply with EN 206-1:2000 the PFA (k) vibrated concrete was examined. The PFA (k) mix designs for both the SCC and VC are provided in Table 5.1, while the workability details are provided in Section 5.3.

Phase 2 Experimental Aim No. 3: *k*-Value Concept (Research Question 1)

Group: *k*-value mixes

Mixes: PFA (k) SCC & PFA (k) VC (compared to mixes above as appropriate)

The effect of the *k*-value concept on SCC chloride transport properties under natural diffusion conditions were assessed in the Phase 2 experimental programme. A CEM 1 42.5 strength class concrete was used for all mixes examined in this thesis and thus the appropriate *k*-value, or efficiency factor, was 0.4 in accordance with EN206-1:2000 (Centre Europeen de Normalisation CEN 2000). The fact that a OPC + PFA VC without the *k*-value implemented was not examined in Phase 2 means that the marine durability effects of the *k*-value efficiency factor was examined in the Phase 2 experimental testing in the context of SCCs only. This examination involves comparing PFA (k) SCC mix will to OPC SCC and OPC + PFA SCC without the *k*-value concept employed, in terms of compressive strength, absorption characteristics, and most importantly in the context of this thesis, in terms of chloride transport properties.

In terms of the mix design itself, EN 206-1:2000 specifies that the maximum amount of PFA to be taken into account in the *k*-value calculation shall meet the requirement of PFA/OPC < 0.33. Consequently, increasing the PFA content above this ratio would have no impact on the effective w/c ratio. Thus in order to implement the *k*-value approach, both the OPC content and the PFA content had to be increased or the water content reduced. Discussions were held with Quinn Cement and the Admixtures Department of the BASF Chemical Company to ascertain the most popular, appropriate approach used in the concrete industry to satisfy the *k*-value requirements. These discussions resulted in a combined approach being adopted whereby the binder content was increased, the water content slightly reduced, and the superplasticiser content slightly increased to preserve workability. The aggregate content was then reduced in line with the relative densities of the binder, the water and the aggregate to ensure the mix design details represented 1 kg/m^3 of concrete. The PFA: OPC ratio was maintained at the maximum of 0.33 for both PFA concretes calculated. Details of this approach are provided in Table 5.1.

Phase 2 Experimental Aim No. 7: Calibration Attempt of Test Results (Research Question 3)

Group: Ferrycarrig Original Mix

Mixes: Ferr. Orig. VC (compared to mixes above as appropriate)

As discussed in the Section 5.1 the Phase 2 testing did not seek to address Research Question 3 directly, but instead aided in addressing Research Question 3 by examining the chloride transport properties of a concrete mix which is used to examine the effect of calibration of salt fog chamber results in a probabilistic framework in Chapter 7. The concrete mix in question was designed to mimic the concrete utilised in the original Ferrycarrig Bridge construction. The C_s and D_{app} results obtained for this mix are used with the results obtained from a site investigation of Ferrycarrig Bridge before the crossheads were extensively repaired to developed calibration factors in Chapter 7. The details of the comprehensive site investigation are provided in a paper published by O'Connor and Kenshel (2013).

As discussed in Chapter 2 Ferrycarrig Bridge was constructed in 1980 and consequently, sufficient details were not available to facilitate an exact replica of the original Ferrycarrig Bridge concrete. Therefore, a mix design was developed to attempt to imitate the Ferrycarrig original concrete using the information available. The Ferrycarrig Bridge original construction drawings were available in the 2005 NRA Eirspan Special Inspection Report (National Roads Authority 2005). These drawings specified that the crosshead beams were constructed of Grade 30 concrete. Kenshel (2009) stated the compressive strength of concrete cores extracted from the crossheads in 2007 were in line with a 28 day concrete strength specification of 30N. Inspection of the concrete cores obtained from the structure in 2007, as part of the work carried out by Kenshel (2009), indicated that 10mm and 20mm aggregate were used in the original mix. With this information a mix design was completed in accordance with the Design of Normal Concrete Mixes manual (Teychenné et al. 1997). The mix design returned a water content of 210 L and a cement content of 381kg/m^3. At this point, a research document which recorded the cement content and w/c ratios in use in Ireland in the 1990s was consulted (West and Keating 1999). This publication considered data collected from 18 ready-mix concrete plants and presented the change in cement content with concrete grade and the change in w/c ratio with concrete grade. The average w/c ratio for 30N concrete at the time the report was published was 0.58, while the average cement content at the time for the 30N mix was 320kg. The mix design was adjusted accordingly to give the Ferr. Orig. mix design presented in Table 5.1.

It is recognised that this approach by no means creates an identical concrete to that used on Ferrycarrig Bridge in 1980. It was thought, however, that the testing of this concrete mix would

give a rough estimate of the performance of a Ferrycarrig Bridge type mix in the salt fog chamber conditions. It will thus, facilitate some form of calibration of the accelerated test results using Ferrycarrig bridge data collected in 2007. A more accurate calibration of the results from the salt fog chamber will be available in time once information is available from Ferrycarrig Bridge.

The mix design details for each of the concretes utilised in the Phase 2 testing are provided below in Table 5.1.

Table 5.1 Mix design details per m^3 of concrete

Mix Type	OPC (kg)	GGBS (kg)	PFA (kg)	Total Binder (kg)	Water (Ltrs)	w/b ratio (kg)	20mm Agg[2] (kg)	10mm Agg.[2] (kg)	Sand (kg)	Plast.[3] (kg)	S. Plast.[4] (ltrs)
OPC SCC	500	_	_	500	220	0.44	_	500	1140	2.7	2.5
GGBS SCC	200	300	_	500	220	0.44	_	500	1140	2.7	2.2
PFA SCC	350	_	150	500	220	0.44	_	500	1140	2.7	2.5
PFA (k) SCC	400	_	132	500	198	0.37 k(0.44)	_	500	1140	2.7	3.0
OPC VC	420	_	_	420	185	0.44	_	1040	720	2.3	1.2
GGBS VC	168	252		420	185	0.44	_	1040	760	2.3	1.2
PFA (k) VC	334	_	111	445	166	0.37 k(0.44)	_	1040	760	2.3	1.2
Ferr. Orig. VC	320	_	_	320	186	0.58	825	412	666	_	_

1, C.I.; corrosion inhibitor; 2, Agg, aggregate; 3, Plast, plastisiser; 4, S. Plast, superplasticiser

5.2.3 Sample Preparation and Laboratory Procedures

Three of the Phase 2 experimental aims are not addressed via changes in experimental testing, or via comparison of concrete mix types, and thus the means by which these aims are addressed have not been presented. Instead these three experimental aims were achieved through sample preparation procedures as detailed below.

Phase 2 Experimental Aim No. 1: Examination of the effect of initial absorption (Research Question 1)

The discussion in Section 4.4.7 in the previous chapter highlighted initial rapid chloride absorption as a possible cause for error in the Phase 1 test results. As previously discussed in Chapter 4 this concern was based on the findings of Thomas and Matthews (2004) who cited initial absorption as a considerable error in determining the relative diffusion characteristics of OPC concrete and

concretes containing PFA. While some publications address the issue of initial absorption through pre-soaking samples prior to exposure (Basheer et al. 2008; Cheng et al. 2011; Dhir and Byars 1993; Dhir et al. 1994), the majority of publications do not account for initial absorption (Li 2001; Loser et al. 2010; McPolin et al. 2005; Stanish and Thomas 2003). No publications could be found in the literature which examined the effect of pre-soaking samples on chloride ingress by testing both pre-soaked and unsoaked samples. This is likely due to the fact that the effect of pre-soaking will vary between studies, depending on the environment in which the samples dry out and the duration over which they dry out. In addition, examining pre-soaked samples and unsoaked samples under natural migration testing is a costly and time consuming exercise. However, as there was no literature available on the effect of initial absorption on performance, and as it was thought initial absorption could have reduced the relative performance of the GGBS and PFA concretes in the Phase 1 experimental programme, the Phase 2 testing programme examined both pre-soaked and un-soaked samples. Since the primary reason for testing the effect of initial absorption was to assess its influence on the Phase 1 testing, the three Phase 1 SCCs were examined in both a pre-soaked state and an un-soaked state. The PFA(k) SCC option was also examined in both the soaked and unsoaked state to facilitate comparison across a different spectrum. Thus the specimens used in the initial absorption study were as follows;

- OPC SCC; 3 100mm cubes pre-soaked, 3 100mm cubes not pre-soaked
- PFA SCC; 3 100mm cubes pre-soaked, 3 100mm cubes not pre-soaked
- GGBS SCC; 3 100mm cubes pre-soaked, 3 100mm cubes not pre-soaked
- PFA (k) SCC; 3 100mm cubes pre-soaked, 3 100mm cubes not pre-soaked

The slab samples were not pre-soaked prior to testing in order to preserve the testing conditions across the Phase 1 and Phase 2 testing, with the exception of changes to the cyclic exposure. The remainder of the test samples investigated in the Phase 2 testing programme were pre-soaked prior to testing. This meant that if errors due to pre-soaking were found to be substantial, a full experimental comparison could be performed across all mixes without the influence of pre-soaking.

The pre-soaked samples were prepared in an identical manner to the un-soaked samples, up to the point when the sample sealant painting was complete. At this point the pre-soaked samples were placed back in the curing tank for 13 days. This pre-soaking duration was selected by monitoring the change in weight of one of each of the mix types at each day over the pre-soaking period. Once the change in weight in a 24 hour period was less than 1% for all the samples, the specimens were judged to be adequately saturated. After the 13 day period the samples were placed directly into the chamber after weighing and the Phase 2 salt fog testing commenced with a 12 hour wet cycle. It

should be noted that for reasons detailed later in this chapter the silane samples were placed in the salt fog chamber 5 days after the remainder of the test specimens.

Phase 2 Experimental Aim No. 2: Investigate effect of silane application method on variability of silane results (Research Question 1)

As discussed in Chapter 4, the silane results in the Phase 1 testing showed high levels of variation when compared to the other sample sets. A possible reason put forward for these high levels of variation was the manner in which the silane surface treatment was applied to the Phase 1 specimens. For Phase 1 the silane was applied by spraying the specimen surfaces until the concrete was visibly saturated as per the manufacturer's guidelines. As noted in Chapter 4, application of silane would vary far more in an on-site situation than in a laboratory study, due to access problems on site and human error. Consequently, in order to further investigate OPC SCC with silane, an OPC option examined under Research Question 1, part (a), it is important to identify if silane performance is greatly affected by the uniformity of application.

To investigate if the Phase 1 silane variation was due to application procedure or to concrete variability, a highly uniform silane application method was adopted for the Phase 2 testing. This highly controlled spraying method allowed all samples to receive the same quantity of silane treatment. Thus if the variability of the silane in the second phase of testing was still high it could be stated that the variation was not due to the application method utilised. If however, the performance variation in Phase 2 was substantially reduced, this would indicate that the cause of the Phase 1 variation was indeed the nature of the silane application. The test set-up for the silane application is shown below in Figure 5.7.

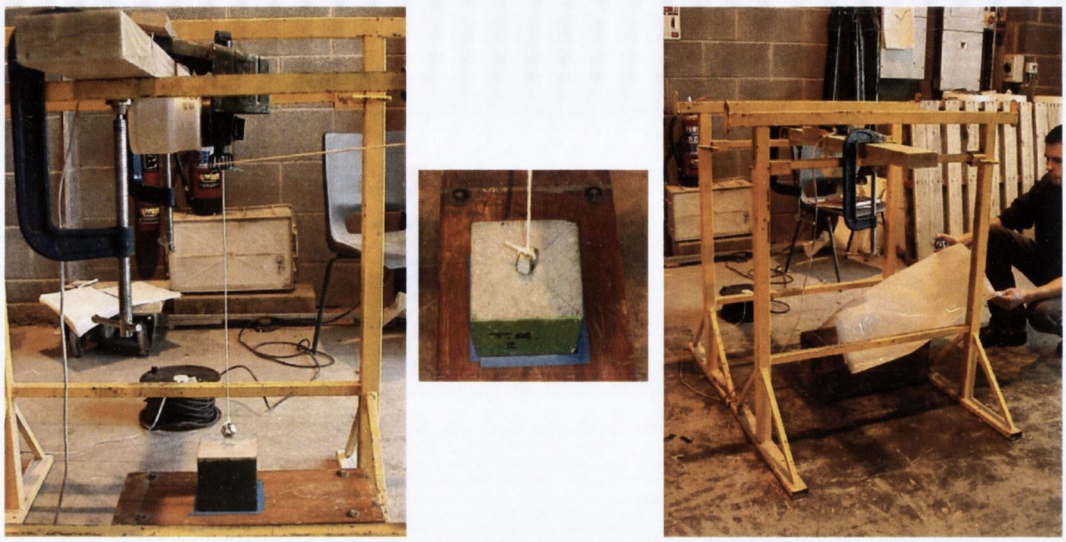


Figure 5.7 Silane application set-up

The manufacturers state that samples should be flow coated to saturation point with a dosage of 150g/m^2. The spray gun utilised was found to have a spray rate of 175g/min. The area of the spread of the spray was measured in the test set-up above. The proportion of this area to the 100mm x 100mm cube area was used, with a safety factor of 1.8, to calculate a suitable spraying duration, ensuring the manufacturers recommended dosage was achieved.

The silane application for each cube included the following steps

- Place samples in position directly under the airless spray gun nozzle using the plumb bob shown in Figure 5.7.
- Cover the samples with plastic sheeting and engage the airless spray gun for 3 seconds to ensure uniform spraying had been established.
- Remove the plastic sheet in one quick motion and spray the cube for 5 seconds, after which time the spray gun trigger is disengaged.
- Allow samples to dry for 24 hours before placing in the salt fog chamber environment.

It should be noted at this point that the experimental design for the silane sample preparation involved a substantial error. In order to compare the silane results with those of the other cube samples the silane underwent the same sample preparation as the majority of the 100mm cube specimens and were thus pre-soaked before exposure. Pre-soaking was carried out prior to silane spraying as the hydrophobic nature of the silane spray would have prevented efficient soaking after silane application. The error however, was in the drying time allowed before the application of the silane. After soaking, the samples were allowed to dry for six hours before the silane was applied by airless spray gun. The applications instruction detailed in the silane information pack states

simply that the silane should not be applied to a wet or damp substrate (Evonik Industries 2011). This information is presented in the section which provides details on curing requirement, preparation of substrate, dosage etc. Drying the samples for 6 hours was ample time for the cube specimens to become surface dry and thus comply with the manufacturers guidelines in this regard. It was noticed, however, after the Phase 2 testing had been completed, that under limitations of the silane treatment, the manufacturer states, 'If rain has preceded the application, the surface should be allowed to dry for at least 48 hours'. Consequently, the application of the silane for the Phase 2 testing, six hours after pre-soaking the samples does not comply with the manufacturers guidelines. The practical nature of requiring 48 hours drying time for concrete on a marine bridge is questionable, especially for elements in splash and tidal zones. It is however, recognised that the relative performance of the Phase 2 OPC SCC + silane cannot be fairly compared to the other SCC durability options due to this experimental design error. Despite this experimental design error the results will still facilitate an investigation into the effect of silane application variability on performance variation. The results will also be used as an indicator of the change in silane performance expected from a tidal zone area to a dryer splash zone area.

Phase 2 Experimental Aim No. 5: Investigate combination of specimen size effect and concrete casting mould effect

As discussed in detail in Chapter 4 and reiterated in the introduction section of this chapter, the Phase 1 testing investigated the effect of salt fog chamber testing on different scales, using the SC 1000 chamber and the SKWT4 300 chamber. It was noted that despite both chambers being programed identically the humidity conditions were significantly different. This was tentatively put forward as the most likely cause of difference between the slab and cube results. In addition however, it was noted that the difference in slab and cube tests could also have been due to an unknown size effect or due to the effect of the different casting mould types used in the preparation of both sets of samples. Thus in order to address Research Question 2 in this chapter using the Phase 1 and Phase 2 testing results, the effect of mould type and sample size must be quantified.

Due to the space and time constraints already discussed in this chapter, the three possible influencing factors could not be separately investigated experimentally in the Phase 2 testing. Having identified the difference in environmental conditions as the most likely cause of difference in Phase 1, this parameter was investigated separately as previously discussed. The casting mould effect and sample size effect were examined together by testing both slab and cube specimens for OPC SCC, PFA SCC and GGBS SCC together in the SKWT 4300 chamber. The sample preparation and materials were identical to those employed in the Phase 1 testing. As previously

mentioned the 3 slab samples and 3 cube samples for each mix were not pre-soaked to conserve the experimental procedures utilised in the first phase of testing.

Phase 2: General experimental procedures

Aside from the targeted areas of change highlighted in this sub-section and in sub-sections 5.2.1 & 5.2.2, the experimental preparation and analysis practices adopted in the Phase 2 testing were consistent with those used in the Phase 1 testing, described in detail in Chapter 4. Some of the more important procedural consistencies relating to materials, specimen preparation and analysis are listed below for clarity's sake.

- All materials used in the Phase 1 and Phase 2 testing were identical with the exception of the 20mm aggregate used for the Ferrycarrig mix which was crushed 20mm limestone
- All admixtures, sealant paint and silane used in Phase 2 testing were identical to those products used in the Phase 1 testing
- The concrete mixing procedures for Phase 1 and Phase 2 sample preparation were identical regardless of whether SCC or VC was being mixed
- Concrete for the SCC was poured as per the Phase 1 testing without vibration to aid compaction. The VC samples were poured in 2 layers using a concrete scoop and each layer was compacted for a duration of 15 seconds using a vibration table
- Again 28 day wet curing was adopted for the Phase 2 as with Phase 1 for the reasons outlined in Chapter 4

The Phase 2 profile grinding procedures and titration analysis procedures were also identical to those used in the Phase 1 testing.

5.3 INTRODUCTION TO PHASE 2 RESULTS

Having discussed the details of the Phase 2 experimental design and methodology, the Phase 2 results will now be presented and analysed. The results section is split up into five different sub-sections as follows;

- 5.4 Concrete Characterisation
- 5.5 Ancillary Tests Relating to Research Question 2
- 5.6 Salt Fog Chamber Results: Research Question 2
- 5.7 Salt Fog Chamber Results: Research Question 1

- 5.8 Salt Fog Chamber Results: Research Question 3

As the above list shows, the characterisation of the concretes in terms of the workability and compressive strength is presented first. The next section contains the results and analysis of the ancillary testing, which was carried out to aid in examining the effect of chloride exposure conditions on relative durability performance. The results discussed in this section are absorption and sorptivity test results, drying test results and humidity and temperature monitoring results. Presenting these results before the salt fog chamber test results are introduced allows the salt fog chamber results discussion to draw on all information available from ancillary testing. The third results section presents a discussion on the salt fog chamber test results which relate to Research Question 2.

Once the salt fog chamber results from Phase 1 and Phase 2 testing have been discussed in the context of Research Question 2, the Phase 2 test results concerned with addressing Research Question 1 will be presented. These results include work which addresses Research Question 1 a) through examining issues raised in the Phase 1 testing, and importantly, work which addresses Research Question 1 b), through comparison of the chloride transport properties for SCCs and VCs.

The last set of salt fog chamber test results which are presented and briefly analysed are the Ferrycarrig Bridge Orig. mix results, which relate to Research Question 3.

5.4 CONCRETE CHARACTERISATION

5.4.1 Workability Tests and Results

The Phase 2 experimental design saw the introduction of a number of workability tests which were not utilised in the Phase 1 testing programme. The additional tests were concerned with characterising the fresh properties of the SCCs. These tests were added to the Phase 2 testing programme to reflect the release of a suite of EN Standards relating to SCC in 2010 (Centre Europeen de Normalisation CEN 2010; Centre Europeen de Normalisation CEN 2010; Centre Europeen de Normalisation CEN 2010; Centre Europeen de Normalisation CEN 2010). The release of EN 206-2:2010 was particularly important as it represented the first European Standard which provided rules and guidelines for the workability of SCC (Centre Europeen de Normalisation CEN 2010). This code states that as a basic requirement the slump flow test must be carried out in accordance with EN 12350-8:2010 (Centre Europeen de Normalisation CEN 2010). The code also

states that where necessary, to achieve intended performance of concrete, the following list of specifications should be selected from:

- Viscosity class
- Passing ability
- Segregation resistance
- Other technical requirements

EN 206-9-2010 defines SCC as concrete that is able to flow and compact under its own weight, fill formwork with its reinforcement, duct box houses etc. whilst maintaining homogeneity (Centre Europeen de Normalisation CEN 2010). Thus the main properties of SCC are its ability to flow and compact under its own weight, its stability and its ability to pass reinforcement. In order to investigate each of these properties for the SCC mixes examined in this thesis the following tests were carried out in accordance with the suite of European Standards released in 2010:

- Slump flow test in accordance with EN 12350-9:2010 (Centre Europeen de Normalisation CEN 2010)
  - Examines ability of concrete to flow and compact under its own weight.

- V-funnel test in accordance with EN 12350-9:2010 (Centre Europeen de Normalisation CEN 2010)
  - Examines viscosity of concrete which partially dictates ability to flow and compact under its own weight

- J-ring test in accordance with EN 12350-12:2010 (Centre Europeen de Normalisation CEN 2010)
  - Examines passing ability of SCC in congested reinforcement type situation

- Sieve segregation test in accordance with EN 12350-11:2010 (Centre Europeen de Normalisation CEN 2010)
  - Examines the stability of the SCC

Photographs of each of these tests being carried out can be seen in Figure 5.8 to Figure 5.11. Table 5.2 presents the workability test results for the four SCC mixes examined in the Phase 2 testing. The table also presents the classification limits specified in EN 206-9:2010 (Centre Europeen de Normalisation CEN 2010).

Figure 5.8 OPC slump flow test

Figure 5.9 GGBS V-funnel test

Figure 5.10 PFA J-ring test

Figure 5.11 PFA(k) sieve segregation test

Table 5.2: SCC workability test results

SCC mix	Slump Flow (mm)	T 500 (sec)	V-Funnel (sec)	PJ (16 Bar) (mm)	% Segregation
OPC- SCC	670 (SF2)	2.1 (VS2)	4.6 (VF1)	9.8	3.3
PFA SCC	795 (SF3)	3.0 (VS2)	8.4 (VF1)	9.9	1.6
GGBS SCC	805 (SF3)	2.5 (VS2)	7.1 (VF1)	6.3	8.1
PFA (k) SCC	660 (SF2)	4.7 (VS2)	10.1 (VF2)	16.3	1.5
EN 206-9:2010 Limits	SF1: 550-650 SF2: 660-750 SF3: 760-850	VS1 $\leq$ 2.0 VS2 $\geq$ 2.0	VF1 < 9.0 VF2 9.0-25.0	PJ2 $\leq$ 10	SR1 $\leq$ 20% SR2 $\leq$ 15%

As can be seen from the table the OPC SCC and the PFA (k) SCC concretes fall into the SF2 category while the PFA and GGBS SCC fall into the SF3 slump flow category. It should be noted that in order to satisfy the *k*-value requirement in EN 206-1:2000, the water content in the PFA (k)

mix design had to be reduced slightly and the powder content increased slightly as previously discussed. This is likely the cause of the reduction in the PFA slump flow when compared to the PFA SCC. As previously stated there is little examination into the effect of the implementation of the *k*-value concept in the SCC literature. It was found by Gesoglu that the introduction of PFA and GGBS as direct cement replacements, in similar proportions to those used herein, increases the slump flow diameter of SCC. Sahmaran (2009) and Petal (2004) also found that the use of PFA as a direct replacement increased slump flow diameter. It was however, found by Nehdi et (2004) that the addition of PFA as a direct replacement for OPC resulted in a reduction in slump diameter.

When considering the T_{500} and V-funnel times it can be noted that the use of PFA and GGBS as cement replacements increase the viscosity of the SCC. It is noted that the PFA (k) SCC results in the greatest viscosity test times with the only V-funnel time which falls into the VF2 category. The three other SCC all fall into the VF1 category. The above results are in good agreement with the existing literature which shows that in general, the replacement of cement with PFA and GGBS increases viscosity of SCC (Gesoglu et al. 2009; Sahmaran et al. 2009).

When considering the passing ability it is noted that the OPC and PFA have very similar J-ring results which are just within the EN 206-9:2010 PJ2 classification limit of 10mm. This indicates these SCC are suitable for heavily congested formwork. The GGBS has a PJ value of just 6.3 mm meaning it is well within the PJ2 classification limit. The PFA (k) J-ring result however, falls outside of the EN 206-9-2010 PJ2 limit meaning that if PFA were to be used in accordance with EN 206-1:2000 for a structural member for the SCC studied herein it would not be suitable for a member with heavily congested reinforcement in accordance with EN 206-9:2010. In terms of comparison with the literature, it was found by Uysal (Uysal et al. 2012) and Gesoglu et al. (Gesoglu et al. 2009) that the use of PFA and GGBS as direct cement replacements increased the passing ability of SCC. Thus the results herein are in agreement with publications in the literature in terms GGBS but not in terms of PFA.

Finally, when considering the segregation resistance of the SCC examined in this thesis, it can be noted that the SCCs have excellent segregation resistance with the OPC, PFA and PFA (k) SCCs having a segregation ratio which is a fraction of that required to satisfy the lower requirement of EN 206-9:2010. The GGBS SCC ratio also falls well below the EN 206-9-2010 SR2 limit. This is likely due to the high proportion of fine aggregate used in the SCC mix designs in this thesis, which is almost 69% of the total aggregate.

It is noted that the use of this high fine aggregate content results in a very stable mix without the need for a viscosity modifying admixture. In general a slightly lower fine aggregate content is more common in the literature. The EFNARC guidelines for SCC (EFNARC et al. 2005) provide a typical fines range by mass of 48% to 55% of the total weight of the aggregates. The document states, however, that the range given is by no means restrictive and many SCC will fall outside the ranges for the mix design constituents. The Spanish standard simply states that the proportion of coarse aggregate should be less than 50% of the aggregate (Rodriguez Viacava et al. 2012) while no guidelines are provided in EN 206-9:2010. In general, in the literature, a coarse/fine aggregate ratio of approximately one tends to be the most popular (Akram et al. 2009; Guneyisi et al. 2011; Hwang and Khayat 2009; Loser et al. 2010; Panesar and Shindman 2012; Patel et al. 2004; Sahmaran et al. 2008). However, as mentioned in the previous chapter, many researchers have employed greater proportions of fine aggregate with Yu et al. (2010) and Sahmaran (2009) utilising a mix with 60% of the total aggregate constituting of fines. Ouchi et al. (2003) stated that a mix with a fines proportion of 63% of the total aggregate was typical of SCC used in the United States while Corinaldesi et al. (2004) used a mix with the fine aggregate making up 72% of the total aggregate. It is noted that the higher fines aggregate content in the Ferrycarrig Bridge SCC mix is likely to result in increased shrinkage, however, as previously stated the examination of concrete shrinkage was outside the scope of the work carried out in this thesis, but is recommended as an area for future research in Chapter 8.

5.4.2 Compressive Strength Test Results

Compressive strength testing was carried out at 7, 28 and 56 days on 100mm concrete cube samples for each of the SCC mixes and VC mixes to assess the concrete strength characteristics. Three cubes were tested at each of the three maturity points in accordance with EN 12390-3:2000 (Centre Europeen de Normalisation CEN 2009). The mean compressive strength result and standard deviation associated with each mean result set are shown below for the eight concrete mixes in Table 5.3. As with the compressive strength results in the previous chapter, the t-test lower bound value for the 28 day compressive strength is provided with the test results. The first seven concrete mixes in the table had a characteristic strength of $50N/mm^2$. As can be seen from Table 5.3, based on the mean and standard deviations obtained from the compressive cube testing, it can be said with 95% confidence that these seven mixes have a long run mean compressive strength which is statistically significantly greater than $50N/mm^2$. The Ferrycarrig Bridge original imitation mix had a characteristic strength of $30N/mm^2$. As can be seen from the table the mean 28 day compressive strength is $47.1N/mm^2$. This is in line with the target mean strength based required for a $30N/mm^2$ mix in accordance with the Design of Normal Concrete Mixes manual

(Teychenné et al. 1997). It can also be noted from the table that the Ferrycarrig Bridge original imitation mix (Ferr. Orig.) t-test lower bound limit is greater than 30N/mm^2.

Table 5.3 7, 28 and 56 day compressive strength results

Concrete Mix	7 day strength (N/mm^2)	7 day std. dev. (N/mm^2)	28 day strength (N/mm^2)	28 day std. dev. (N/mm^2)	56 day strength (N/mm^2)	56 day std. dev. (N/mm^2)	t-test lower bound vale (28 Days)
OPC SCC	60.9	0.7	67.6	1.4	74.1	0.4	63.5
GGBS SCC	56.6	0.6	64.1	1.1	69.5	1.9	60.9
PFA SCC	36.2	0.8	54.6	0.8	63.5	1.4	52.3
PFA (k) SCC	37.3	0.8	73.5	1.9	80.6	2.5	68.0
OPC VC	59.5	1.6	72.9	4.7	79.1	2.3	59.2
PFA (k) VC	59.9	1.1	77.5	2.0	86.6	1.6	71.7
GGBS VC	43.1	2.4	65.0	4.2	70.1	2.4	52.7
Ferr. Orig VC	34.8	1.1	47.1	1.0	53.7	0.9	44.2

The strength gain over the period of 7 to 56 days is shown for the SCCs in Figure 5.12. From this figure it can be noted that the PFA SCC has a lower strength than the OPC SCC at each maturity level. This highlights the need for a *k*-value or efficiency factor when considering the compressive strength of SCCs containing PFA. However, the plot also shows that the PFA SCC which utilises a *k*-value of 0.4 leads to a higher compressive strength than the OPC concrete at 28 days and at 56 days. This indicates that when considering 28 day strength for this SCC the *k*-value of 0.4 is somewhat conservative. It is noted however, that this is a very small trial and assessing an appropriate *k*-value for compressive strength of SCC containing PFA is not the aim of this testing. These points are merely made as a discussion on the *k*-value in the context of chloride transport properties which will be presented later in this chapter.

It can also be noted from Figure 5.12 that the GGBS SCC has a slightly lower compressive strength than the OPC SCC at 28 and 56 days. This is interesting when it is considered that no *k*-value is required at present for the use of GGBS as a cement replacement in accordance with EN 206-1:2000 (Centre Europeen de Normalisation CEN 2000). According to the latest literature, however a *k*-value for GGBS is currently being considered and is likely to be implemented in the next version of EN 206-1 (Sanjuan et al. 2011). It is also notable that, as with the Phase 1 results, both the GGBS and the PFA SCCs gain strength on the OPC SCC from 7 to 28 days as one would expect. Again as with the Phase 1 test results the GGBS SCC gain on OPC is greater than that of the PFA SCC.

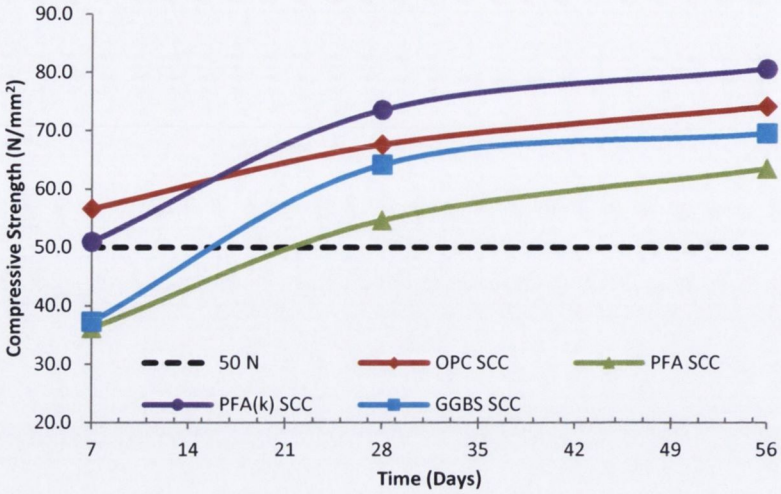


Figure 5.12 SCC compressive strength results

Figure 5.13 below presents the strength gain from 7 to 56 days for the VC. It can be noted from the plot that the PKA (k) VC results in a greater compressive strength than the OPC at 28 and 56 days. It is also noted that the GGBS VC results in a lower compressive strength than the OPC VC at each maturity level plotted. This again indicates that from the period of 28 to 56 days the GGBS VC requires an efficiency factor to reach the same strengths as an equivalent OPC mix while the PFA (k) efficiency factor appears conservative. Therefore, in order to properly analyse the *k*-value in terms of compressive strength, a far more comprehensive analysis would be required, using PFA and GGBS from different sources with different fineness as well as examining strength across a range of w/b ratios and aggregate types. It should be noted that while such a study is feasible for concrete strength, carrying out such a study for chloride transport characteristics using natural diffusion would be extremely costly and would take a huge investment of time, equipment and man hours.

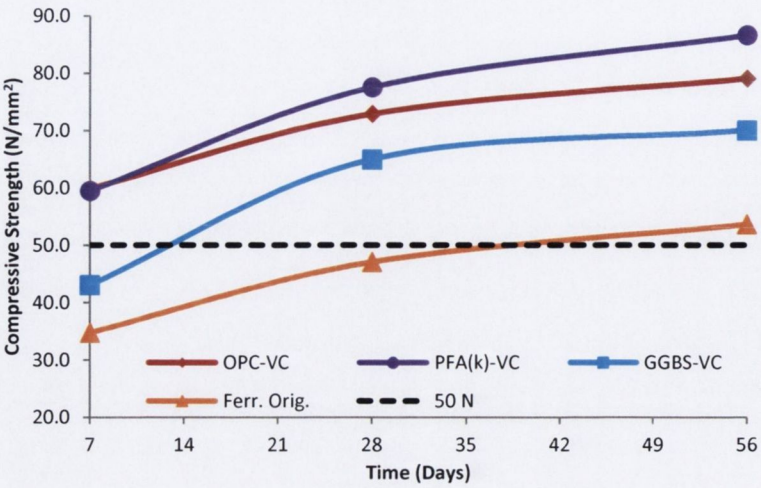
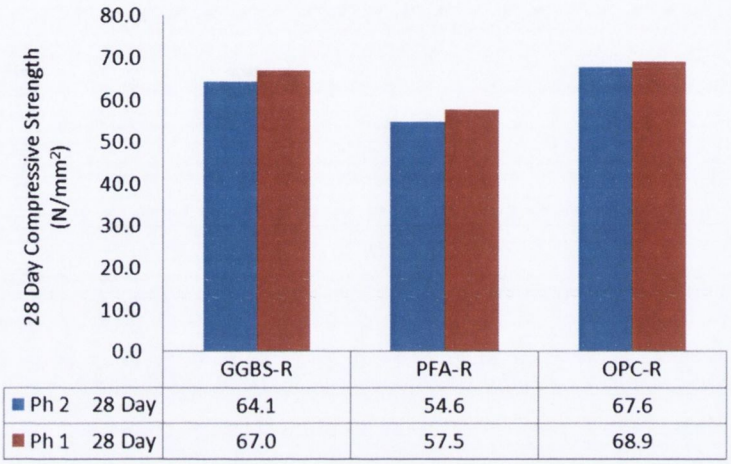


Figure 5.13 VC compressive strength results

Figure 5.14 below presents the Phase 1 and Phase 2 compressive strength results for the OPC, PFA and GGBS SCCs. As can be seen from the chart, very good agreement was identified between the two sets of 28 day compressive strength results.

Ancillary Tests Relating to Research Question 2



	GGBS-R	PFA-R	OPC-R
Ph 2 28 Day	64.1	54.6	67.6
Ph 1 28 Day	67.0	57.5	68.9

Figure 5.14 Phase 1 and Phase 2 compressive strength results comparison

In the context of the chloride resistance performance comparison of the SCCs and VCs, which is presented in Section 5.7.4, Figure 5.15 below compares the SCC and VC compressive strength results at both 28 days and 56 days. As previously established these concretes had equal w/b ratios but different mix proportions.

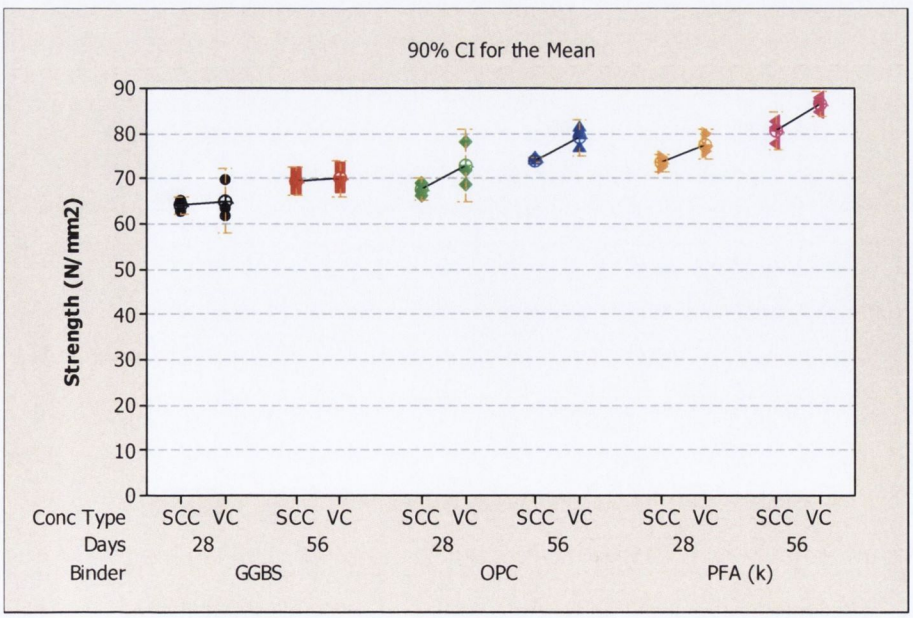


Figure 5.15 SCC and VC strength comparison at 28 and 56 days

As can be seen from the plot the GGBS SCC and VC have almost identical mean compressive strengths. For the OPC concretes and the PFA(k) concretes, the VCs have higher mean compressive strength values at both 28 and 56 days. It is noted, however, that the difference between the compressive strengths are relatively small with the OPC 28 day strength illustrating the maximum reduction from VC to SCC at a value of 7%. It is notable from the plot that none of the differences between the SCC and VC sample mean compressive strengths can be said to represent a statistically significant difference in long run mean values with 90% confidence.

5.5 ANCILLARY TESTS RELATING TO RESEARCH QUESTION 2

The three sets of ancillary test results presented in this section were obtained to aid in the interpretation of the salt fog chamber results relating to Research Question 2 (investigate effect of exposure condition on the relative performance of SCCs). This section simply presents the drying, absorption, and humidity monitoring results. The implications of the ancillary tests' findings in the context of Research Question 2 are primarily discussed in the salt fog chamber results Section 5.6.

5.5.1 Humidity Monitoring Results

As previously discussed in Section 5.2.1 two measures were taken to reduce the level of drying in the SKWT4300 chamber to a level below that experienced in either the SC1000 or the SKWT4300 in the Phase 1 testing. In order to quantify the effect of these measures the temperature and Relative Humidity (RH) were monitored during the Phase 2 testing programme using the Testo monitoring system detailed in Chapter 4.

Figure 5.16 shows RH data for a weekly cycle for the Phase 2 testing programme plotted together with the Phase 1 RH data from the SC1000 chamber and the SKWT4300 chamber. As can be seen from the plot, the Phase 2 RH does not drop below 80% in the 8 day period. In addition the drying periods are far shorter for the samples drying out for 12 hour periods compared with 72 and 48 hour periods in the SKWT4300 chamber in the Phase 1 testing. In the Phase 1 testing a total of 120 hours of drying time was programed into the chambers for a weekly cycle as opposed to 84 hours in the Phase 2 testing. All of these factors will lead to a reduced level of drying in the Phase 2 experimental programme compared to that experienced in either of the salt fog chambers in the Phase 1 testing. It should also be noted that the average temperature in the SKWT4300 chamber over the 8 day period shown below is 21.8°C. The average temperatures in the SC1000 and SKWT4300 chambers for the same period in the Phase 1 testing were 22.2°C and 22.8°C

respectively. Thus the maximum change in temperature across the three environmental conditions is 1 °C. This was the limit of the temperature control system in the salt fog chambers.

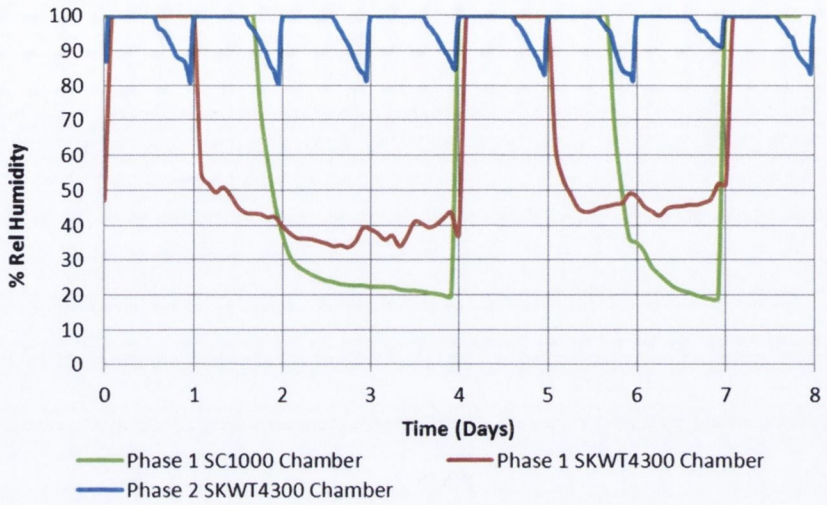


Figure 5.16 One week of relative humidity data from the Phase 1 and Phase 2 testing

Figure 5.17 below shows the Phase 2 RH levels over a 60 day monitoring period. As can be seen from the plot, the relative humidity in the chamber drops below 70% only once and, in general, lies between 100% and 80%. The average RH in the chamber over the monitoring period was 96% as shown on the plot below.

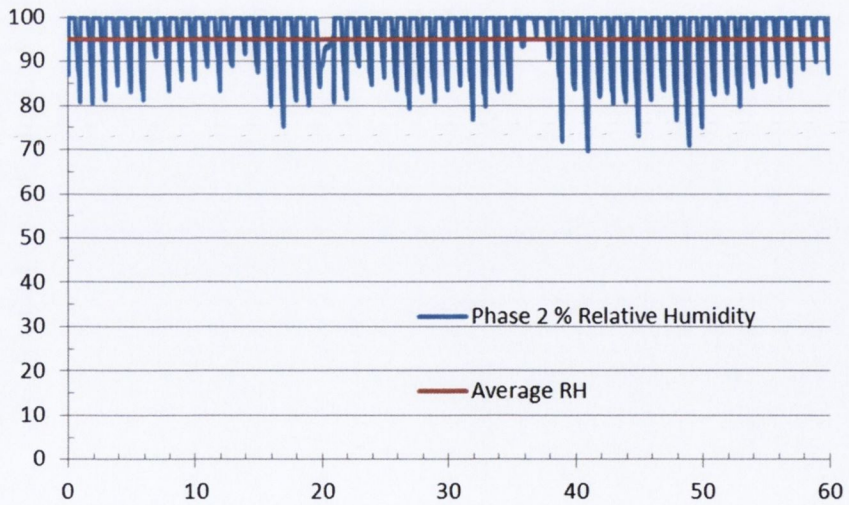


Figure 5.17 Phase 2 relative humidity data over 60 day monitoring period

Overall it can be stated that the environmental conditions in the Phase 2 testing are stable and differ from the Phase 1 experimental conditions, with less sample drying facilitated in the Phase 2 testing regime.

5.5.2 Absorption Test & Sorptivity Test Results

As previously mentioned, two types of absorption tests were carried out in the Phase 2 testing programme to aid in the understanding of the performance of the alternative concrete mixes in a chloride exposure environment which allows for wetting and drying cycles. Although the absorption tests were carried out for all the concrete mixes in the Phase 2 programme, it is the results for the mixes which were tested in both the Phase 1 programme and the Phase 2 programme which are of primary interest (i.e. OPC SCC, PFA SCC and GGBS SCC). This is due to the fact that the Phase 1 and Phase 2 salt fog chamber data for these three SCCs offer an opportunity to examine performance across three different exposure conditions. Consequently, this section will first examine and compare the absorption properties of the OPC SCC, PFA SCC and GGBS SCC. Subsequently the SCC and VC absorption properties will be compared in slightly less detail to investigate whether there is a notable difference in absorption characteristics of the SCC and VC tested herein.

Figure 5.18 below presents a sorptivity test plot which was obtained for one of the OPC SCC samples. This plot is typical of the results obtained in the Phase 2 testing programme. As can be seen from the plot the relationship between the water absorbed per unit of surface area, and the square root of time is very well represented by a linear relationship with a R^2 value of 0.993 obtained. The slope of this line is termed the sorptivity coefficient in accordance with the relevant standard (RILEM TC 1980). Three of these plots were produced for each of the 8 concrete mixes investigated in the Phase 2 testing. The R^2 values obtained in the study ranged from 0.928 to 0.998.

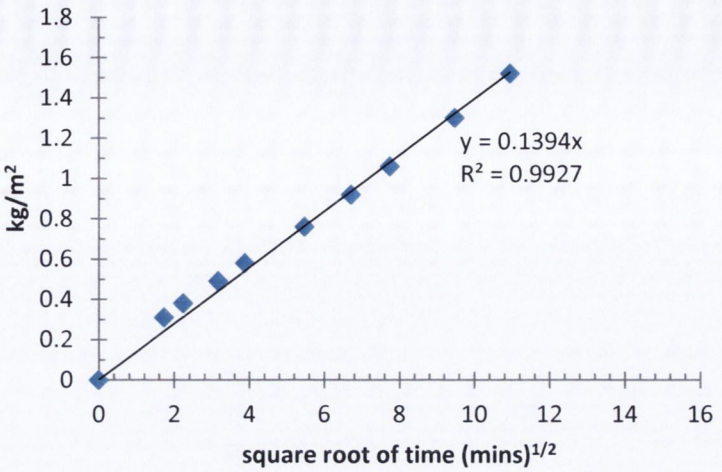


Figure 5.18 Plot to obtain water absorption plot for OPC-SCC sample

The sorptivity coefficient values obtained for each test sample for the OPC, PFA and GGBS SCCs are plotted below in Figure 5.19 together with the mean values and the 90% confidence intervals for the long run mean sorptivity coefficients.

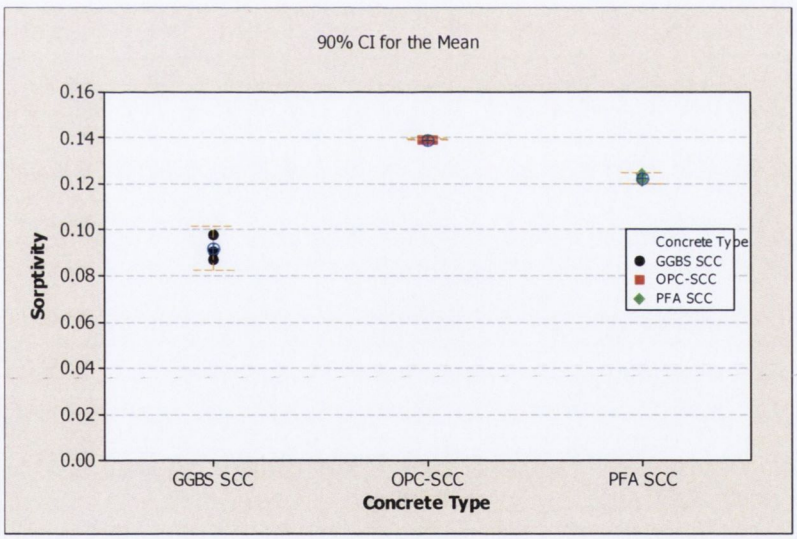


Figure 5.19 Sorptivity Indices for OPC SCC, PFA SCC & GGBS SCC at 28 days

As can be seen from the plot the OPC SCC results in the highest sorptivity index with a mean value of 0.139 mm/min$^{1/2}$ (kg/m^2/min$^{1/2}$). The utilisation of 30% PFA as a direct cement replacement results in a 12% reduction in the mean sorptivity coefficient to a value of 0.122 mm/min$^{1/2}$. The GGBS SCC results in the lowest sorptivity index with a mean value of 0.092 mm/min$^{1/2}$, a reduction of 34% when compared to the OPC SCC. From the plot it can also be noted that the results obtained indicate with 90% confidence that each of the long run mean sorptivity coefficients

for the OPC SCC, PFA SCC and GGBS SCC are statistically significantly different from one another.

The findings above compare well with some of the studies in the existing SCC literature. Guneyisi et al. (2011) found that direct replacement of OPC with PFA in the proportion of 20 & 40% resulted in a reduction in the sorptivity coefficient of approximately 2% and 15% respectively. Guneyisi et al. also found that the replacement of OPC with 60% GGBS resulted in a reduction of the sorptivity coefficient of approximately 32%. Sahmaran et al (2009) found that the replacement of OPC with 30% PFA reduced the sorptivity by 13%.

Figure 5.20 & Figure 5.21 below present the absorption test results for the OPC, PFA and GGBS SCCs. The plots show the % change in weight of the test specimens at 30 minutes and 24 hours. This is simply calculated using Equation 5.1 where W_t is the weight at time, t, in the soaking tank and W_{dry} is the dry weight of the specimen.

$$\frac{W_t - W_{dry}}{W_{dry}} \times 100\% \qquad \textbf{Equation 5.1}$$

As can be seen from the plots there is good agreement between the 30 minute absorption results and the 24 hour absorption results. Both sets of data indicate that OPC SCC results in the greatest amount of absorption with a mean absorption percentage of 2.8% and 5.7% for 30 minutes and 24 hours respectively. The replacement of OPC with 30 % PFA reduces the mean water absorption by 13% and 7% for 30 minutes and 24 hours respectively. The GGBS SCC has the lowest water absorption with a mean value of 2.2% and 4.8% for 30 minutes and 24 hours. This represents a 21% reduction over the OPC SCC at 30 minutes and a 16% reduction over the OPC SCC at 24 hours. These results indicate that OPC absorbs water at a slightly faster initial rate than concretes containing GGBS and PFA, although it is noted that there is more variation associated in the 24 hour data set as shown by the interval bars in Figure 5.21. The absorption test data is in good agreement with the sorptivity test data, however, the reduction in absorption characteristics in the concretes containing the SCM is less in the absorption test results than in the sorptivity test results.

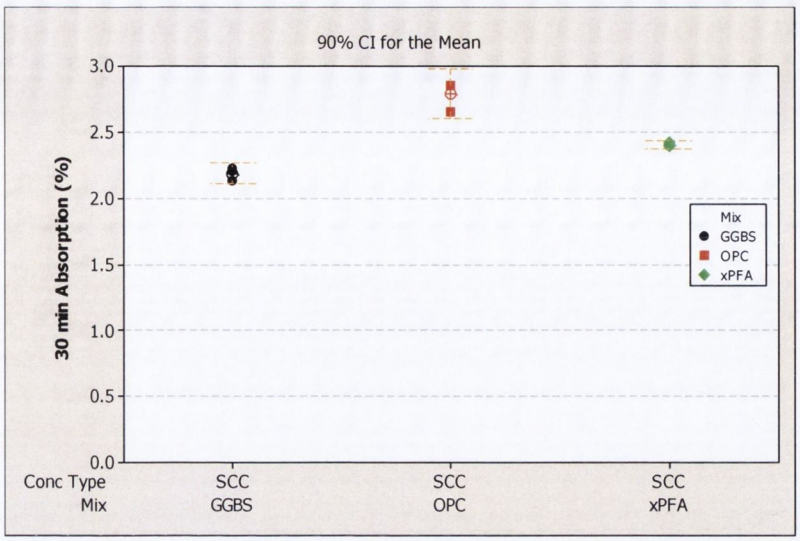


Figure 5.20 30 min OPC, PFA and GGBS SCC absorption results

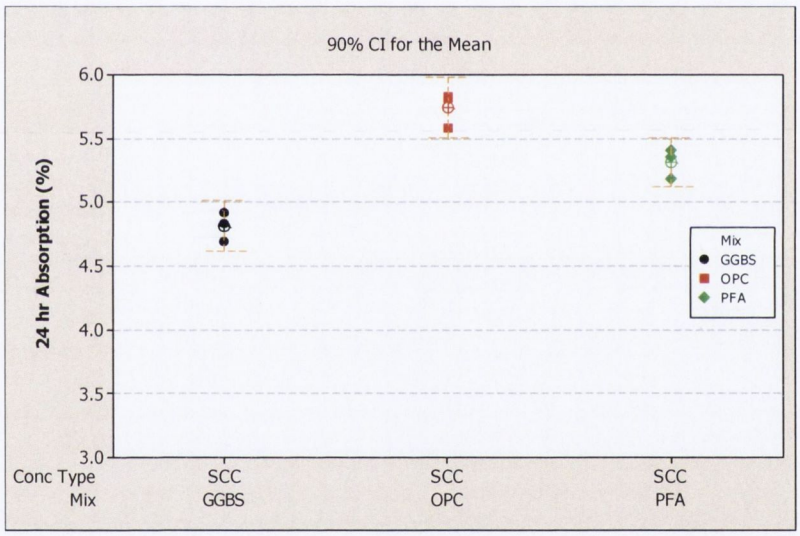


Figure 5.21 24 hr OPC, PFA and GGBS SCC absorption results

A summary of the OPC, PFA and GGBS SCC sorptivity and absorption results is presented in Table 5.4 below. The results are presented as percentages of the OPC SCC results to facilitate easy comparison. The 24 hour absorption result is particularly interesting in the context of the Phase 1 results discussed in Chapter 4. This result indicates that there is only a 7% difference between the absorption characteristics of the OPC and PFA SCC when exposed to saturated conditions for 24 hours. In relative absorption performance terms PFA therefore has a 24 hour value of 1.08. Studies in the literature examining pure diffusion in saturated concrete suggest that concrete with PFA used as a 30% direct replacement for OPC has relative durability characteristics considerably higher than 1 (Bentz et al. 1996; Elahi et al. 2010; Stanish and Thomas 2003). Consequently, the results

presented in the table below suggest that the greater the portion of chloride ingress by absorption the closer OPC + PFA as a direct replacement will be to 1.08.

Table 5.4 Summary of absorption and sorptivity results for OPC, PFA and GGBS SCCs

Mix	Sorptivity (% of OPC)	30 min Absorption (% of OPC)	24 hour Absorption (% of OPC)
OPC SCC	100	100	100
PFA SCC	78	87	93
GGBS SCC	66	79	84

In the Phase 1 testing both chambers exposed specimens to two 24 hour periods/week of saturation with chloride solution. From the Phase 1 humidity data it was established that a greater amount of drying occurred in the slab testing environment. Consequently, absorption will have played a bigger role in the slab environment as the dryer the sample the more chlorides will be absorbed. Thus, the results in Table 5.4 indicate a possible reason for a drop in PFA performance in the Phase 1 slab samples when compared to the Phase 1 cube samples. It is noted, however, that the drop in relative performance from 1.6 to 0.95 cannot be explained by absorption alone based on the results in the table as this would require that the PFA SCC showed greater absorption than the OPC concrete for a given concrete moisture content. This will be discussed further in the context of the examination of the effect of exposure conditions on salt fog chamber results in Section 5.6.3.

Figure 5.22 below shows a comparison between the sorptivity results for the VCs and the SCCs for OPC, GGBS and PFA(k). The utilisation of the *k*-value with PFA for the SCC has resulted in a reduction in mean sorptivity when compared the PFA SCC mix. The VCs illustrate a similar pattern to the SCCs with OPC having the largest mean sorptivity coefficient and GGBS having the lowest sorptivity coefficient. It is interesting to note that the VC have a higher mean sorptivity than the SCC for PFA (k) and GGBS, however, the opposite is the case for the OPC concretes. There is, however, some variation associated with these results. Consequently, a statistically significant difference between the VC and SCC long run mean sorptivity coefficients can only be identified with 90% confidence for the OPC concretes.

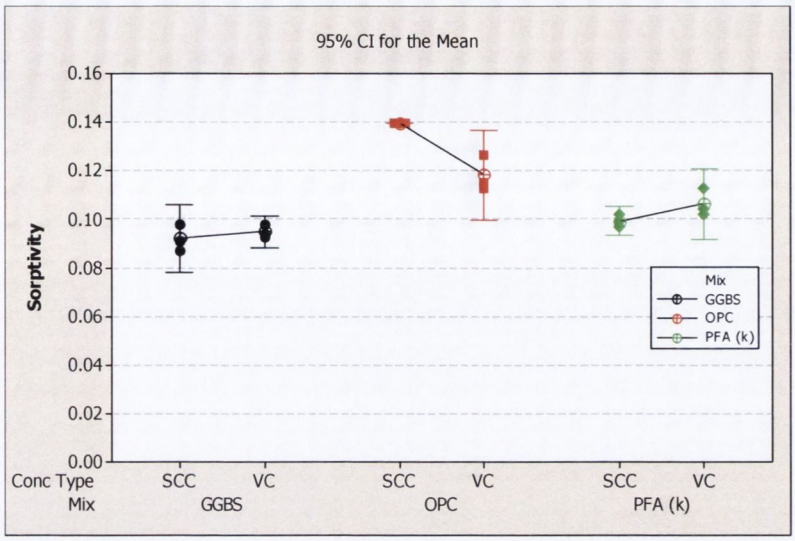


Figure 5.22 Comparison of SCC and VC sorptivity coefficients

A more uniform pattern was identified in the absorption test results as can be seen from Figure 5.23 and Figure 5.24 below. First considering Figure 5.23 can be seen that the water absorption is lower for the VC for OPC, PFA(k) and GGBS. The variation associated with the results is comparatively quite low. Consequently, the results indicate that the VC had statistically significantly lower water absorption than the SCC with 90% concrete for all the concretes compared. This reduction in water absorption in the VC becomes more pronounced in the 24 hour tests results shown in Figure 5.24.

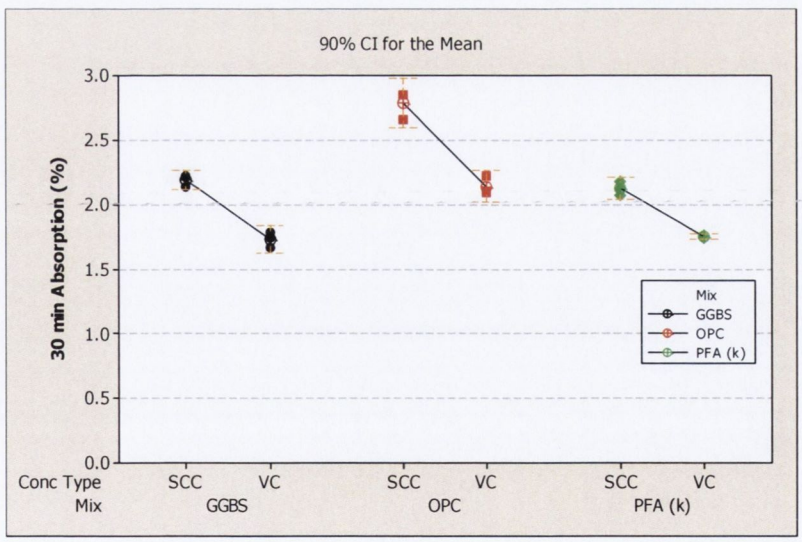


Figure 5.23 SCC and VC 30 min absorption results comparison

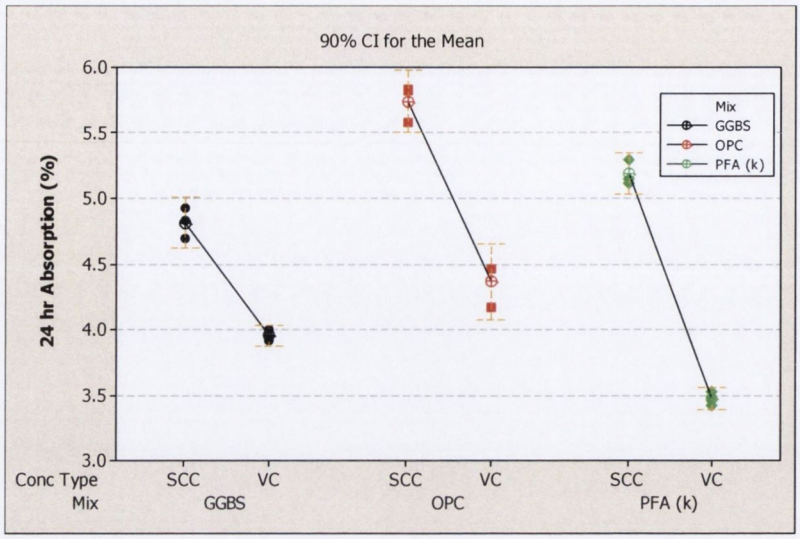


Figure 5.24 SCC and VC 24 hour absorption results comparison

It is not known why the sorptivity test results and the 30 minute water absorption test results disagree. However, it is noted that the sorptivity test results seem to have been more affected by variation, and in addition the patterns observed in both the 30 minute results and the 24 hour results for the absorption test seemed to be beyond chance variation. Consequently, the results and analysis indicate that, for the concretes considered herein, the SCCs would appear to have a greater capacity to absorb water than equivalent VCs.

5.5.3 Drying Test Results

The drying test was carried out for the SCC mixes which were examined in both the first and second phase of experimental testing, namely OPC SCC, PFA SCC and GGBS SCC. Three test samples were tested for each of the concretes and the 75 Watt fan was utilised in the chamber to mimic the Phase 1 conditions and promote faster drying over the monitoring period of 10 hours. The relative humidity levels in the chamber during the drying test can be seen below in Figure 5.25. As can be seen from the plot, the relative humidity levels fluctuate little over the testing period with readings ranging from 51% to 56%.

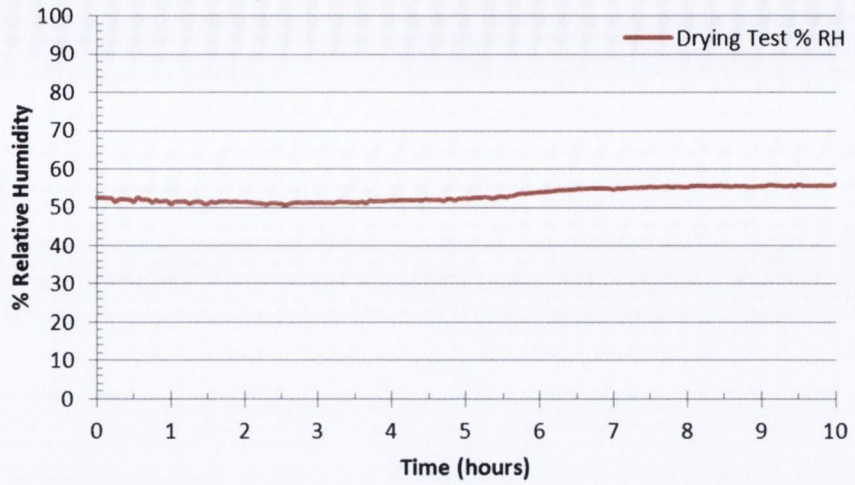


Figure 5.25 Relative humidity levels in the chamber during the drying test

Figure 5.26 presents the incremental drying results for the three SCCs over a period of 10 hours. As can be seen from the plot the most drying occurs in the first 30 minutes. This is likely due to surface drying and drying of the area immediately beneath the surface. The PFA SCC shows the highest initial drying, and maintains the highest rate of drying up to the point of 5 hours when OPC begins to dry at a slightly higher rate. The GGBS SCC can be seen to have a high initial drying rate, however, this quickly drops below the other concretes with the GGBS having the lowest rate of drying throughout the 10 hour monitoring period.

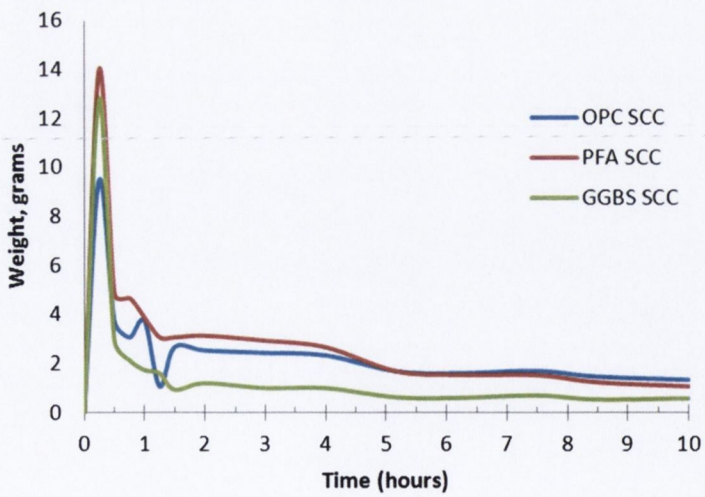


Figure 5.26 Incremental drying data for OPC, PFA and GGBS SCCs

Figure 5.27 below presents the total or cumulative drying of the three concretes over the same 10 hour monitoring period. As can be seen from the plot the PFA clearly results in the greatest

amount, of drying over the monitoring period with GGBS resulting in the least amount of drying over the 10 hour monitoring period.

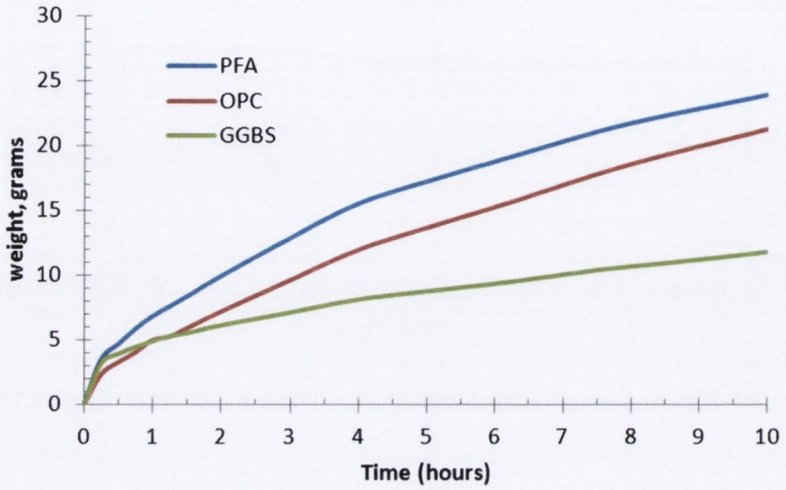


Figure 5.27 Total drying data for OPC, PFA and GGBS SCCs

In order to give an idea of the variation experienced in the drying test the total drying for all the test specimens is presented below in Figure 5.28. As can be seen from the plot, the PFA exhibited the most variation. However, overall the results are seen to be very consistant given the variable nature of concrete.

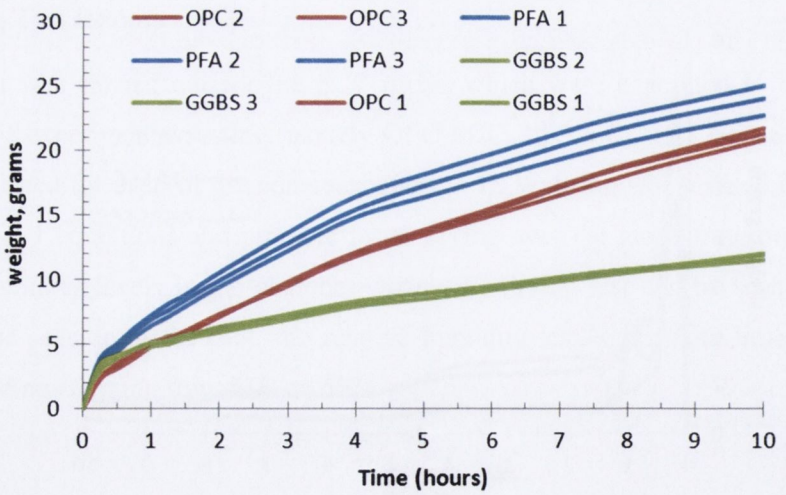


Figure 5.28 Total drying data for each of test specimens

As mentioned in Section 5.2.1 numerous publications examining concrete durability investigate the water absorption characteristics of OPC, OPC + PFA and OPC + GGBS, there is little information in the literature relating to the drying ability of these options. A study by Kanna et al. (1998) was,

however, mentioned in Chapter 4 which examined the effects of drying on OPC, OPC + 35% PFA and OPC + 35% GGBS mortars in the context of a study concerned with drying shrinkage. The samples, with a w/b ratio of 0.45, were cured for 7 days and then subjected to four different moisture conditions. The four conditions were; a) saturated for 21 days, b) 50% humidity for 7 days, c) saturated for 21 days followed by 6 days in an oven and d) 21 days of 50% RH followed by 6 days in the oven. In terms of natural drying, and comparison with the results herein, the 50% RH condition is most relevant. Kanna et al. found that over the 21 day period the OPC + 35% PFA mortar dried out the most of the three mortars, with a 20% greater reduction in moisture content than the OPC mortar. The OPC + GGBS mortar and the OPC mortar dried out to a similar extent with the GGBS moisture content reducing by 5% more than the OPC mortar over the 21 days. Thus, as with the study conducted herein, Kanna et al. found that the use of PFA as a direct replacement for OPC resulted in a greater level of drying than observed in the plain OPC option.

When the findings presented above are considered in the context of the wetting and drying cycles in marine exposure it would seem that for a drying period in the region of 10 hours, concrete containing 30% PFA as a direct OPC replacement will dry out faster than OPC concrete. Thus the OPC + PFA concrete will have the capacity to absorb more chloride laden water upon commencement of a wetting cycle. This, combined with the absorption results discussed, indicates that the relative performance of 30% PFA concrete compared to OPC concrete will be reduced if testing is carried out in an environment which incorporates an element of drying. The influence of these findings on the salt fog chamber results will be discussed further in Section 5.6.3 which presents the salt fog chamber results for the three different exposure conditions.

5.6 SALT FOG CHAMBER RESULTS: RESEARCH QUESTION 2

5.6.1 Introduction

As discussed in Section 5.1 there are two Phase 2 experimental aims which need to be achieved to address Research Question 2 as follows;

5. Investigate the combination of specimen size effect and casting mould effect on the relative performance of the OPC, PFA and GGBS SCC options
6. Investigate the effect of exposure conditions on the relative merits of alternative SCC durability options

These experimental aims will be presented in turn in this section. In general the basis for the salt fog chamber results analysis has been developed in the previous chapter. Consequently, details such as the relevance of confidence intervals, and the means by which transport properties were obtained will not be discussed again. In addition, an effort has been made to further condense the presentation of results in this chapter due to the large volume of different research areas being investigated in the Phase 2 testing programme. The rigorous nature of the results analysis will, however, be maintained and comparisons will be made with results from the literature within sub-sections where possible.

5.6.2 Examination of Size Effect & Mould Type Effect

As previously discussed the combined effect of the mould type and specimen size were examined together in the Phase 2 experimental programme by testing both slabs and cube specimens for OPC SCC, PFA SCC and GGBS SCC in the SKWT 4300 chamber. This sub-section will focus on the comparison of the slab and cube results for these concretes. As with the Chapter 4 results discussion, the C_s and D_{app} values will first be presented to facilitate graphical observation of the individual specimen results and variation associated with the results for each sample set. Following this the mean chloride profiles will be compared. The estimated T_i values and relative merits will then be presented. It should be noted that the plots and discussion presented in this sub-section are concerned with the comparison of the slab and cube performances in the Phase 2 testing and thus the performance of the PFA and GGBS SCCs relative to the OPC SCCs will not be discussed in this sub-section.

Figure 5.29 below presents a comparison of the slab and cube C_s values obtained for the Phase 2 test specimens for the OPC SCC, PFA SCC and GGBS SCC. As can be seen from the plot, the slab specimens result in lower mean C_s values than the cube specimens for each of the concrete types investigated. The difference in mean C_s values is very similar for the OPC SCC and the GGBS SCC with mean differences of 0.11% by weight of concrete. This represents an approximate reduction in C_s of 10% from cubes to slabs. There is a larger difference between the slab and cube C_s values for the PFA SCC with the slab C_s value 23% smaller than the cube C_s value. It is noted that the variation associated with the PFA C_s values is larger than the variation associated with the other concrete types. Given this larger PFA variation the difference of 23% between the PFA slab and cube mean C_s values cannot be said to represent a statistically significant difference with 90% confidence. The interval bars in Figure 5.29 show that a statistically significant difference between the slab and cube C_s long run means values cannot be detected for any of the concrete types with 90% confidence. However, as discussed and illustrated in Chapter 4, the small sample size of three

samples make a statistically significant difference difficult to detect. The plot does, however, suggest that the C_s value is lower for the slabs than the cubes for SCCs examined.

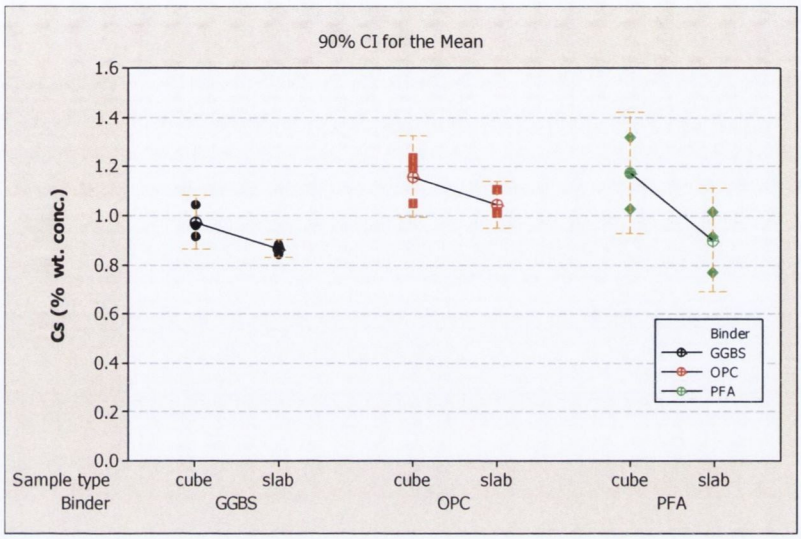


Figure 5.29: C_s value comparison for SCC slabs and cubes

The precise reason for the apparent mould/size effect on C_s is unclear. It is possible that the timber formwork used in the slab specimens lead to a greater quality of surface concrete than that of the cube specimens, which were cast in plastic moulds. This theory originated from considering the comparison of normal formwork and of controlled permeability formwork. Controlled permeability formwork is specially designed formwork which reduces the near surface water binder ratio by allowing water and air to escape while containing the cement and aggregate particles (Figueiras et al. 2009). This type of formwork has been shown to significantly enhance the durability of concrete when compared to impermeable formwork in terms of D_{app} and absorption characteristics for both VC and SCC (Basheer et al. 2008; Figueiras et al. 2009). It is noted that the differences between impermeable formwork and controlled permeability formwork will be far greater than the difference between timber formwork and the plastic formwork. It is true, however, to state that the timber formwork would have absorbed some water at the concrete formwork interface. This will have led to a reduction in the w/b ratio in the surface layers of concrete. However, the plastic moulds would not have absorbed any water, leading to a higher w/b ratio in the concrete surface layers. It is also possible that the reduction in C_s was due to an unknown size effect in the specimens. Given the variation identified in the C_s value data it is also possible that the reduction in C_s from cube specimens to slab specimens was the result of random variation.

Figure 5.30 below presents the D_{app} value comparison for the three concrete types. As can be seen from the plot the PFA and GGBS D_{app} values are almost identical across the slab and cube specimens. There is, however, a notable difference between the OPC slab and cube mean D_{app} values with the slab mean D_{app} value 20% smaller than the cube mean D_{app} value. Again the precise reason for this difference is unclear, but, it is noted that the OPC mix was the first concrete mix to be poured in the Phase 2 experimental programme and in the Phase 1 experimental programme. The timber formwork would, therefore, have been driest for the OPC SCC pours, meaning the formwork would have been capable of absorbing more surface moisture for the OPC SCC slabs than the GGBS and PFA SCC slabs.

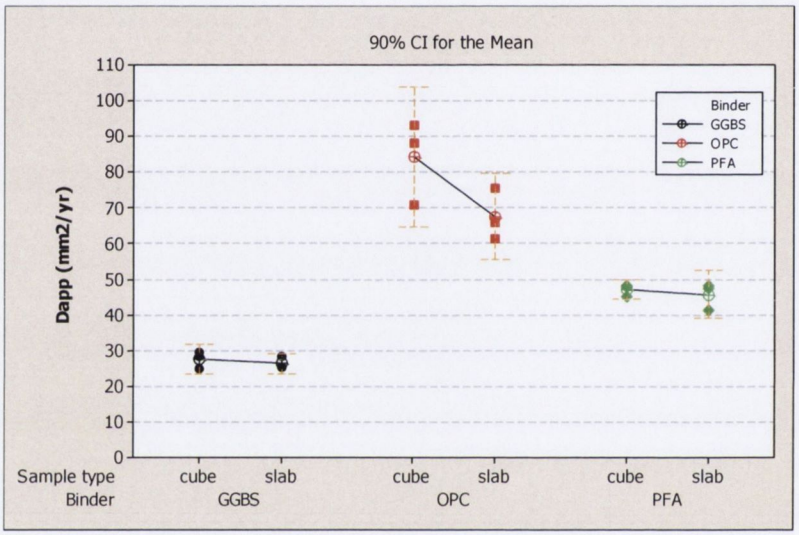


Figure 5.30: D_{app} value comparison for SCC slabs and cubes

Figure 5.30 also shows that the variation associated with the OPC D_{app} values is far greater than the variation associated with the GGBS and PFA D_{app} values. Thus, it is possible that the difference in the OPC SCC D_{app} values is due to experimental and material variation. The interval bars in Figure 5.30 show that, based on the experimental results a statistically significant difference with 90% confidence could not be detected between the cube and slab long run mean D_{app} values for any of the concrete types investigated. Finally it is noted that, as with the C_s values, any possible shift in D_{app} values for the OPC concretes could also be due to the size effect of the specimens.

Figure 5.31 to Figure 5.33 below present the mean slab and cube chloride profiles obtained for the OPC SCC, the PFA SCC and the GGBS SCC. In Figure 5.31 it is noted that a combination of a slightly reduced slab C_{cr} value and the reduced D_{app} values results in a lower chloride content in the OPC slab sample at each depth increment. The effect of considerable reduction in the mean PFA C_{cr} value from the cube to the slab specimens can be seen in Figure 5.32. It is noted, however, that

due to very similar cube and slab D_{app} values in the PFA SCC, the effect of the increased cube C_s reduces with depth into the concrete, resulting in a near convergence of the lines with depth. From Figure 5.33 it is noted that the slab and cube profiles for the GGBS SCC are very similar.

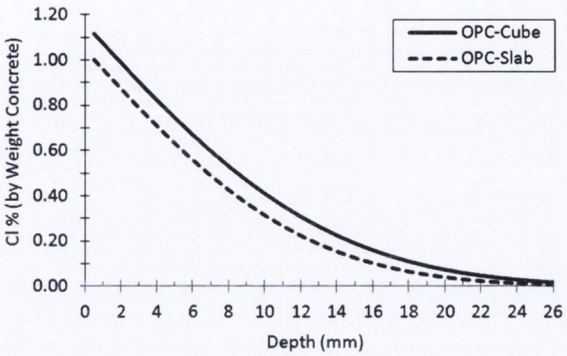


Figure 5.31: OPC SCC slab and cube chloride profiles

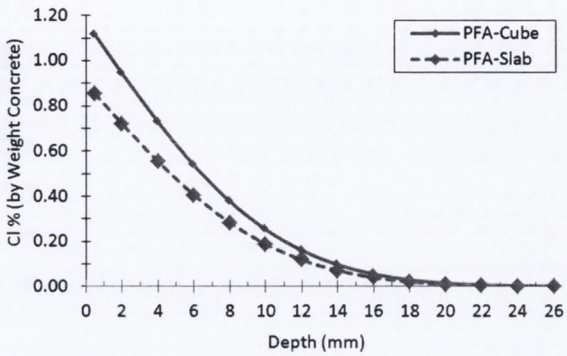


Figure 5.32: PFA SCC slab and cube chloride profiles

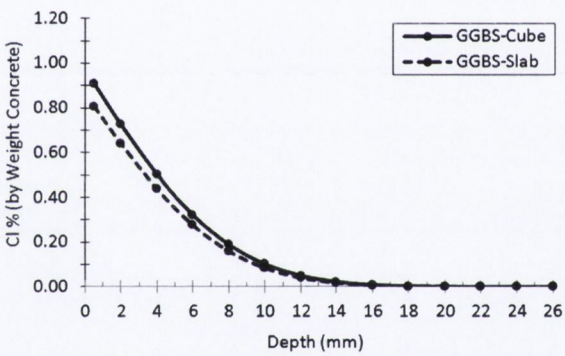


Figure 5.33: GGBS SCC slab and cube chloride profiles

As with the Phase 1 results, T_i values, calculated using the C_s and D_{app} values discussed above, will now be presented by way of an illustrative example. Again Fick's second law with Crank's error solution, presented in Chapter 4 as Equation 4.3, is used for the illustrative example with a C_{cr} value of 0.07% by weight of concrete and a cover value of 50mm. It is noted that detailed service life prediction modelling will be carried out using the results in Chapter 7. This calculation of T_i is merely used as a means of combining the C_s value and D_{app} value into one figure which quantifies performance.

The T_i values for the slab and cube comparison are shown in Figure 5.34 below. The pattern between the slab and cube values for the three concrete types is quite uniform as depicted by the slope of the mean connecting lines. The difference in the slab and cube samples for the OPC, PFA and GGBS SCCs are 1.2, 1.3 and 1.4 years respectively. Thus, from the results the apparent

combined slab effect/mould effect increases with the T_i value, from 1.2 to 1.4 years, across the different concretes. It should be noted that this difference results in a reduced percentage difference with increasing T_i value. The percentage increase in the mean T_i values from cube to slab specimens for the OPC, PFA and GGBS SCCs are 30%, 18 % and 11% respectively. This has implications for the comparison of relative merits of slabs and cubes.

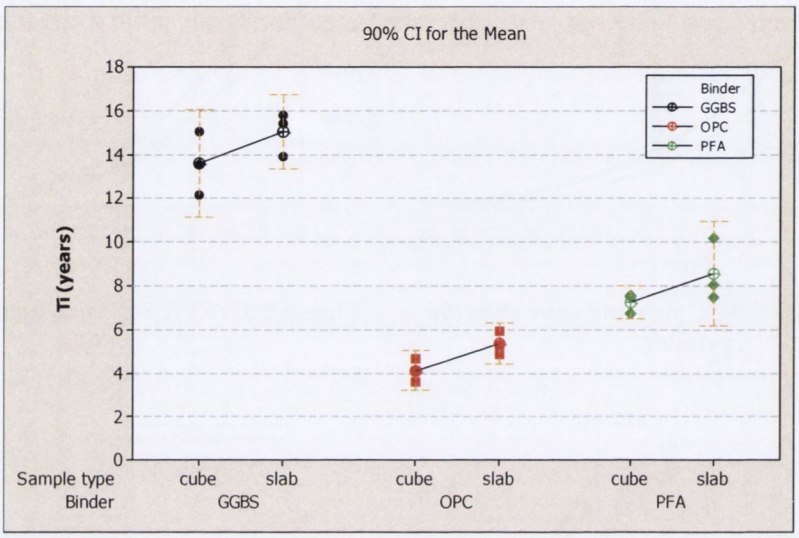


Figure 5.34: T_i value comparison for SCC slabs and cubes

Figure 5.35 below shows the relative merits of the GGBS SCC and PFA SCC for the cubes and slabs specimens. The OPC relative merits are also plotted to illustrate the spread of the OPC data. As can be seen from the plot the relative merit values for the PFA and GGBS slabs is smaller than that of the cube relative merits. This is due to the fact that the % increase in mean T_i value is greater for the OPC slabs than for the PFA slabs and GGBS slabs as shown above. This is highly relevant in the context of the difference in relative merit identified in the Phase 1 testing for the PFA SCC concrete.

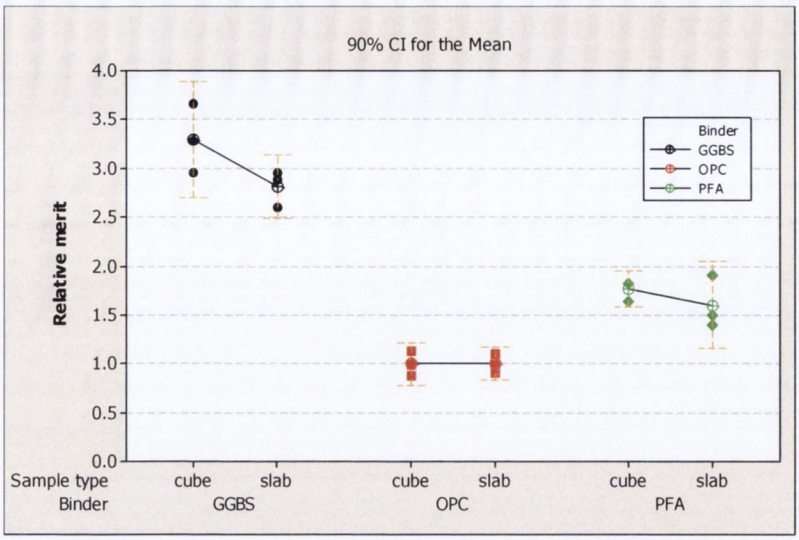


Figure 5.35: Relative merits of slabs and cube specimens

For clarity's sake the relative merit values for the PFA and GGBS SCCs obtained in the Phase 2 testing are presented below in Table 5.5. The Phase 1 relative merits are also included in the table to facilitate an assessment as to whether the combined size/mould effect could have been the reason for the change in relative performance of the PFA SCC from slabs to cubes in the Phase 1 testing.

Table 5.5 Slab Vs. cube relative merit: Phase 1 & 2

Durability Option	Phase 2 mean relative merit	Phase 1 mean relative merit
PFA Slabs	1.60	0.95
PFA Cubes	1.76	1.60
GGBS Slabs	2.81	2.27
GGBS Cubes	3.30	1.88

Firstly, considering the PFA SCC comparison, the Phase 2 analysis indicates that the size/mould type does reduce the relative merit value from cubes to slabs. As shown in the plots above, this is due to the fact the OPC concrete has the greatest percentage improvement from cube to slab specimens. It is noted from the table, however, that the reduction in relative merit from cubes to slabs for the PFA is 9%. The corresponding figure for the Phase 1 test results is 41%. This indicates that although the combined size/mould effect appears to reduce the performance of the PFA SCC in the slab specimens, based on the mean relative values in Table 5.5, this is very unlikely to be the sole reason for the change in PFA SCC performance identified in the Phase 1 testing. It is therefore likely that the PFA slab relative performance was reduced in the Phase 1 testing due to a

combination of the size/mould effect and the exposure condition effect, and possibly random variation. The effects of exposure condition effect will be discussed in detail in the next section.

Examining the GGBS results it is noted that the Phase 2 slab relative merits represents a 15% reduction on the Phase 2 cube relative merit. Again this is due to the greater percentage improvement of the OPC slab in the Phase 2 testing. Interestingly, the relative merit value from the Phase 1 testing increases from cube to slab specimen by a figure of 21%. This increase in relative merit in the Phase 1 slab sample is the opposite to that experienced in the PFA concrete and the opposite to the effect that the Phase 2 analysis suggests. This will be discussed in more detail in the next section.

Overall the assessment of the slabs and the cube samples in the Phase 2 testing would suggest that the combined size & mould effect does influence the slab and cube relative merits. It would however, seem unlikely that this effect is solely responsible for the change in the PFA performance from slab sample to cube sample in the Phase 1 testing programme. Given the results discussed herein, the effect of the environmental exposure conditions will be assessed in the following section by comparing Phase 2 slabs to Phase 1 slabs and Phase 2 cubes to Phase 1 cubes. Thus the effect of the exposure conditions can be assessed across three exposure environments without the influence of the size/mould effect.

Studies examining the effect of plastic and timber casting moulds on chloride diffusion could not be found in the existing literature. As previously stated, research into controlled permeability formwork is the only area which offers a similar, if not greatly increased effect. There is also a lack of suitable information in the existing literature assessing specimen size effect. Consequently, the findings presented herein could not be compared with the existing literature.

5.6.3 Effect of Exposure Condition on Relative Performance

As stated in the previous section, the examination of the effect of the exposure condition on the relative performance of the concretes will be assessed by comparing the Phase 1 slab results to Phase 2 slab results and Phase 1 cube results to Phase 2 cube results, facilitating analysis without the influence of the size/mould effect. The Phase 2 results presented in this sub-section are from concrete specimens which were not pre-soaked prior to testing. Thus continuity of sample preparation was preserved across the Phase 1 and Phase 2 testing regimes.

The variation between exposure conditions across the three sets of results means there is little to be learned from comparing C_s values, D_{app} values or chloride profiles across the three sets of data. Different exposure conditions will result in varying degrees of chloride ingress in the concrete. This is intuitive and does not need to be confirmed experimentally. However, as discussed in the literature review in the context of the development of Research Question 2, what is of interest is whether the relative performance of alternative SCCs, changes with alternating environmental conditions i.e. is the use of PFA SCC more suited to an environment or member location which experiences little drying or a lot of drying? Consequently, this sub section will examine the effect of the exposure condition through comparison of the performance of PFA SCC and GGBS SCC relative to that of the control OPC SCC through study of the relative merit values obtained from the three different environmental conditions examined.

As discussed in Section 5.5.2 the absorption tests indicated that the greater the role of absorption in the exposure condition the poorer the performance of GGBS SCC, and in particular PFA SCC, relative to OPC SCC. The drying test carried out on the OPC SCC, PFA SCC and GGBS SCC showed that the PFA SCC had the greatest capacity to dry out while the GGBS had the lowest drying capacity of the three concretes examined over 10 hours. This drying data is further evidence that the PFA SCC performance will be reduced with increased levels of drying in the exposure system. Thus based on these ancillary test results one would expect the PFA SCC to perform better in the Phase 2 testing than in either of the Phase 1 chamber tests. It is difficult to quantify how the interaction between drying and absorption will affect the performance of the GGBS SCC relative to OPC SCC across the different absorption conditions. The comparison of the Phase 1 and 2 results will, however, facilitate a quantification of this effect.

For ease of discussion the three different environmental test conditions will be numbered. As per the discussion in Section 5.5.1 the driest test condition was experienced by the slab samples in the SKWT 4300 chamber in the Phase 1 testing. This testing condition will be termed exposure condition 1. The test with the second highest level of drying was the Phase 1 SC1000 chamber test which was used to examine the Phase 1 cube samples. This will be termed exposure condition 2. Finally the Phase 2 test conditions, used to examine both slab and cube samples in an environment with the lowest level of drying will be termed exposure condition 3. As previously stated, slab specimens will be compared to slabs and cube specimens to cubes. The slab comparison will therefore facilitate comparison between exposure conditions 1 and 3. The relative humidity conditions for this comparison are shown below in Figure 5.36.

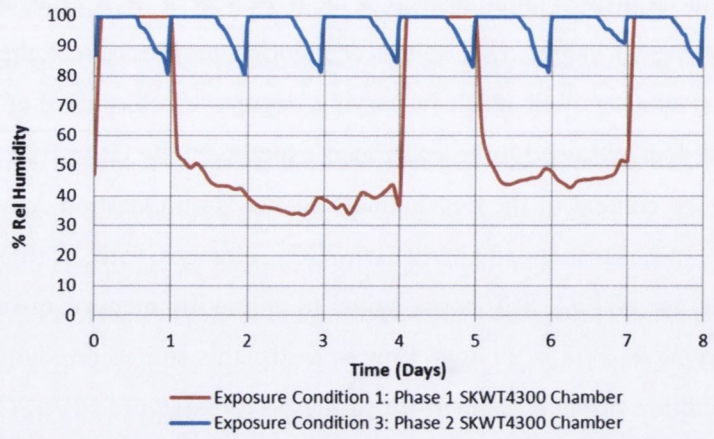


Figure 5.36: RH conditions for Phase 1 & 2 slab comparison

The cube comparison will facilitate comparison between exposure conditions 2 and 3. The relative humidity conditions for this comparison are shown below in Figure 5.37.

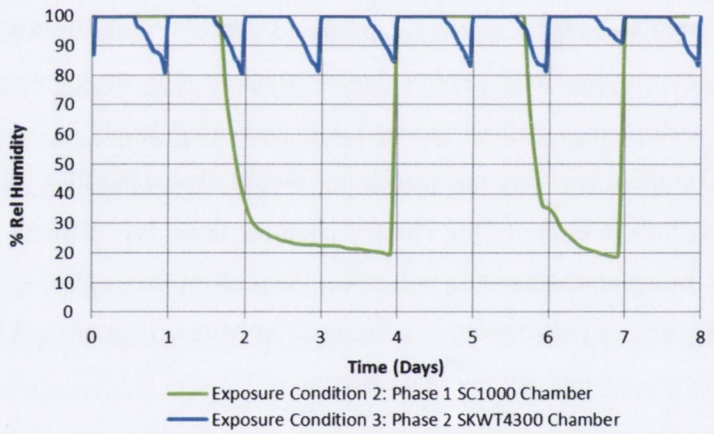


Figure 5.37: RH conditions for Phase 1 & 2 cube comparison

The effect of the change in exposure conditions on the PFA SCC relative performance will be discussed first. Figure 5.38 below presents the change in PFA relative merit from exposure condition 3 to 2 (cube samples) and from exposure condition 3 to 1 (slab samples).

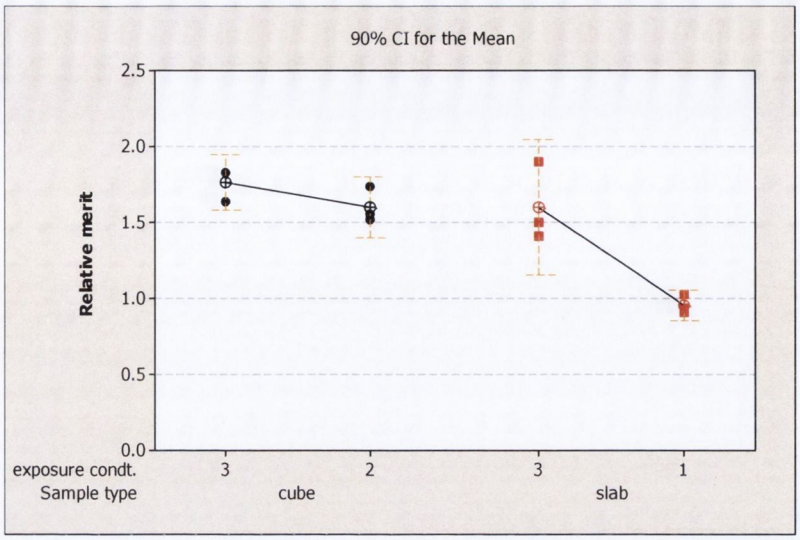


Figure 5.38: Change in PFA SCC performance across three exposure conditions

The first thing that can be noted from the plot is that for both exposure condition comparisons the relative merit of the PFA SCC reduces with increased levels of drying. The reduction in the PFA relative merit is greatest for the 3 to 1 exposure condition comparison. This slab comparison shows a reduction in the mean PFA SCC relative merit from 1.6 to 0.95. This represents a 41% reduction in the PFA relative performance from exposure condition 3 to 1. As can be seen from the interval bars in Figure 5.38 this reduction represents a statistically significant difference in the long run mean PFA relative merits with 90% confidence. The reduction in the relative performance from exposure condition 3 to 2 is smaller with the PFA relative merit falling from 1.76 to 1.60, representing a 9% reduction in relative merit. As can be seen from the plot, this reduction does not constitute a statistically significant difference between the long run mean PFA SCC relative merits with 90% confidence.

When Figure 5.38 is considered in the context of the Phase 1 results and the analysis of size/mould effect discussed in Section 5.6.2 , it seems likely that the reduction in the PFA SCC relative merit from Phase 1 cube specimens to Phase 1 slab specimens identified in Chapter 4 was primarily due to the change in exposure effects. This is based on the fact that the size/mould effect for the Phase 2 PFA SCC test was found to be the order of 9% while the exposure condition effect for PFA SCC above was found to be as high as 41%.

Figure 5.39 presents the change in the GGBS SCC relative merit from exposure condition 3 to 2 and from exposure condition 3 to 1. As with the PFA SCC, the GGBS SCC relative merit decreases as the level of drying in the testing increases for both exposure condition comparisons. Firstly, considering the slab comparison, the change in exposure condition from 3 to 1 results in a

reduction in GGBS SCC mean relative merit from 2.81 to 2.27. This represents a 19% reduction in relative merit. From the interval bars in Figure 5.39 it can be seen that this difference in relative merit represents a statistically significant difference in long run mean relative merits from exposure condition 3 to 1 with 90% confidence.

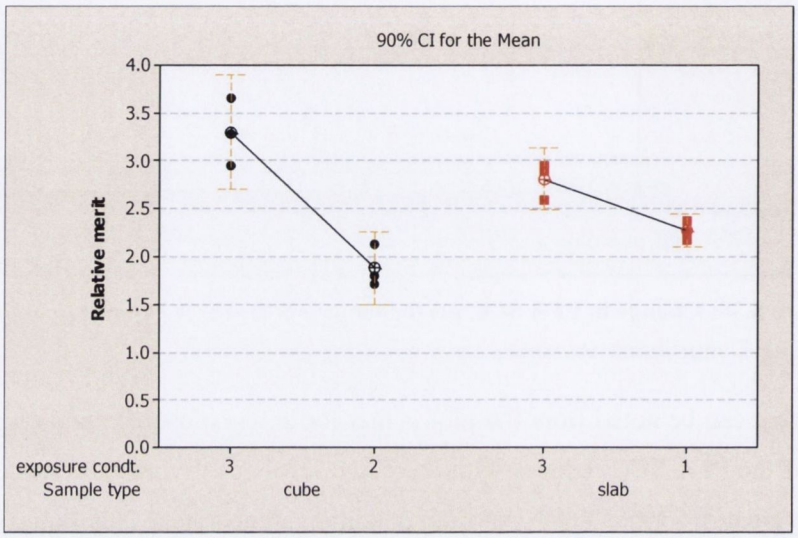


Figure 5.39: Change in GGBS SCC performance across three exposure conditions

There is an even greater reduction in GGBS relative merit from exposure condition 3 to 2. The comparison shows a drop in GGBS mean relative merit from 3.30 to 1.88; a 43% reduction. This is larger than expected, as one might have envisaged a greater reduction from exposure condition 3 to 1 than from exposure condition 3 to 2. It is noted that there is more variation associated with the GGBS cube samples relative merits than the GGBS slab samples relative merits. Despite this variation, the interval bars in the plot indicate that the difference in mean relative merit from exposure condition 3 to 2 easily represents a statistically significant difference in long run mean relative merits with 90 % confidence.

The novel nature of this investigation into the effect of exposure conditions on relative performance of different concretes, as highlighted by the literature review in Chapter 3, means comparison of the results presented above with the existing literature is limited to a single publication by Ben Fraj et al (2012). As detailed in Chapter 3 the Ben Fraj et al. study examined the change in D_{app} for OPC and GGBS vibrated concretes across three different RH levels using cyclic wetting and drying tests. The RH levels were controlled at 50%, 75% and 90% during the drying time of the wetting and drying cycles. These exposure conditions will be termed 1, 2 and 3 respectively to facilitate a comparison with the three exposure conditions presented herein. The average RH levels in the dry cycles of exposure conditions 1, 2 and 3 presented in this thesis were 45%, 50% and 92%

respectively. It should be noted that exposure condition 1 also had a greater degree of fan assisted drying as discussed in Chapter 4 and in Section 5.2.1 above.

Table 5.6 below presents the changes in the relative merit of GGBS concrete due to varying exposure conditions which were obtained by Ben Fraj et al. (2012), together with the results obtained from the experimental testing herein.

Table 5.6 Change in relative merit with exposure condition found by Ben Fraj et al. (2012) and found in the experimental programme herein

Concrete type	Change RM from exp. condt. 3 to 2	Change in RM from exp. condt. 3 to 1
(Ben Fraj et al. 2012) OPC + GGBS	4.0 - 1.2	4.0 - 1.3
GGBS SCC	3.3 - 1.9	2.8 - 2.3
PFA SCC	1.8 - 1.6	1.6 - 1.0

RM = relative merit, exp. condt. = exposure condition

As can be seen from the table, Ben Fraj et al. (2012) found that the performance of the GGBS concrete relative to the OPC concrete was reduced with reduced RH in the drying cycle. The patterns found by Ben Fraj et al. can be seen to be very similar to the patterns found in this experimental study for both the GGBS SCC and the PFA SCC.

The presentation of the relative merits across the three different exposure conditions presented above, combined with analysis of the absorption testing, sorptivity testing and drying test results, has allowed Research Question 2 to be addressed through the achievement of Phase 2 experimental aim No. 6. To recap Research Question 2 was presented in Chapter 1 as follows;

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

The combined Phase 1 and Phase 2 experimental testing indicates that, when compared to OPC SCC, the relative performance of both OPC + PFA SCC and OPC + GGBS SCC are reduced with increased levels of drying. This is a novel finding in the context of the discussion in Chapter 2 and 3, which highlighted that the majority of laboratory investigations in the literature examine the chloride transport properties of concretes in fully saturated conditions. Thus the work presented above, together with the work of Ben Fraj et al. (2012), is one of the few experimental studies in the available literature which attempts to quantify the amount by which the performance of OPC +

PFA and OPC + GGBS concretes are reduced with increasing levels of drying in cyclical chloride solution exposure.

The practical importance of such studies is clear when it is considered that corrosion of reinforcement does not in general occur in fully submerged (fully saturated) reinforced concrete elements due to the low level of oxygen available (Neville 2011; Richardson 2002). It is therefore, important to assess the effect of varying exposure conditions on concretes which have been established as being more durable based primarily on saturated experimental testing. As indicated by the results presented herein, OPC + GGBS concretes in particular are likely to be more durable than OPC concretes regardless of exposure conditions, however, it is important to investigate the scale of reduction in relative performance of these durable concrete options under various exposure conditions. This need is highlighted by the fact that the OPC + PFA performance was poorer than OPC in the Phase 1 SKWT 4300 testing (most drying) and 60% better than OPC in the Phase 2 SKWT4300 testing (least drying).

5.7 SALT FOG CHAMBER RESULTS: RESEARCH QUESTION 1

5.7.1 Introduction

This section discusses the Phase 2 experimental aims which seek to address Research Question 1 part a) and part b). Having addressed a large part of Research Question 1 part a) in Chapter 4, the most important part of this section is experimental aim No. 4, which compares the performance of SCC and VCs. This analysis is presented in Section 5.7.4. The other parts of this section relate to examining issues raised in the Phase 1 testing, namely; effect of initial absorption (Section 5.7.2) and Silane Variability (Section 5.7.3). The PFA SCC option is also further examined in this section through investigating the effect of the k-value efficiency factor in durability in Section 5.7.5.

5.7.2 Examination of Initial Absorption: Effect of Pre-Soaking Samples

As previously stated, the primary reason for analysing the effect of pre-soaking was to assess if initial absorption had a significant effect on the Phase 1 relative merits of the PFA SCC and the GGBS SCC, given that information on the effects of initial absorption on chloride ingress migration testing could be found in the existing literature. When considering the effect of initial absorption, one might expect the un-soaked samples tested herein to display a slightly higher C_s value and slightly higher D_{app} due to the additional chlorides which migrated into the specimens, as a result of the initial absorption of chlorides upon exposure. It is also noted that the effect of initial

absorption is likely to be greater in the Phase 2 testing than the Phase 1 testing due to the fact that the Phase 2 testing had less drying resulting in less absorption during the 36 week exposure period.

Figure 5.40 below shows the C_s values obtained for the soaked and un-soaked samples. As can be seen from the plot, the effect of pre-soaking on C_s is not consistent across the durability options. Unexpectedly, pre-soaking causes a slight increase in the mean GGBS SCC and PFA(k) SCC C_s. As previously discussed in Chapter 4, C_s values are subject to higher levels of variation than D_{app} values as they are highly influenced by the two to three shallowest chloride content results, D_{app} on the other hand is based on up to 15 chloride content readings. Consequently, the slight increase in GGBS and PFA(k) C_s for un-soaked to soaked specimens is likely due to random variation. The C_s value can be seen to increase from soaked to un-soaked for the OPC and PFA SCC options C_s. The biggest change is seen in the OPC durability option. This could be due to the fact that, as shown in Section 5.5.2, the OPC SCC has the greatest capacity for absorption followed by PFA SCC. It could however, also be due to random variation. As can be seen from the plot, none of the changes in the C_s values from soaked to un-soaked specimens can be said to represent a statistically significant difference in the long run mean values with 90% confidence.

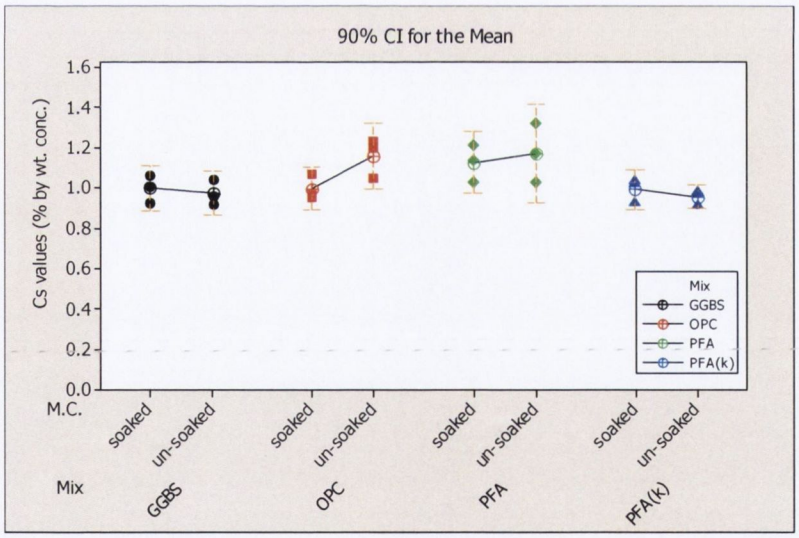


Figure 5.40 Soaked and un-soaked sample C_s values

Figure 5.41 below presents the D_{app} values for the soaked and un-soaked specimens. Upon examination of Figure 5.41 it is noted that the D_{app} value results in a more consistent apparent soaking effect. It is noted that for each of the mixes considered, the soaked samples illustrate a reduction in the D_{app} value over the un-soaked samples. Again the differences identified do not constitute a statistically significant difference in the long run mean D_{app} values with 90% confidence.

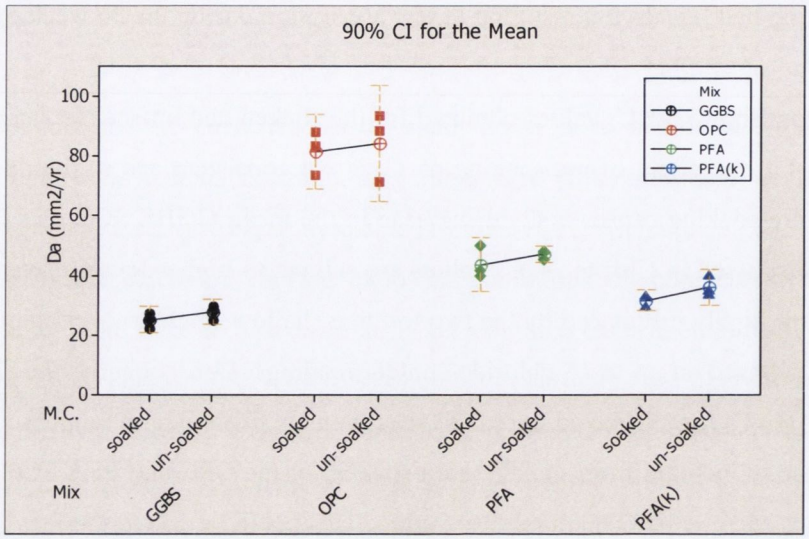


Figure 5.41 Soaked and un-soaked sample D_{app} values

Figure 5.42 shows the T_i value prediction for each of the mixes for both the pre-soaked and un-soaked samples.

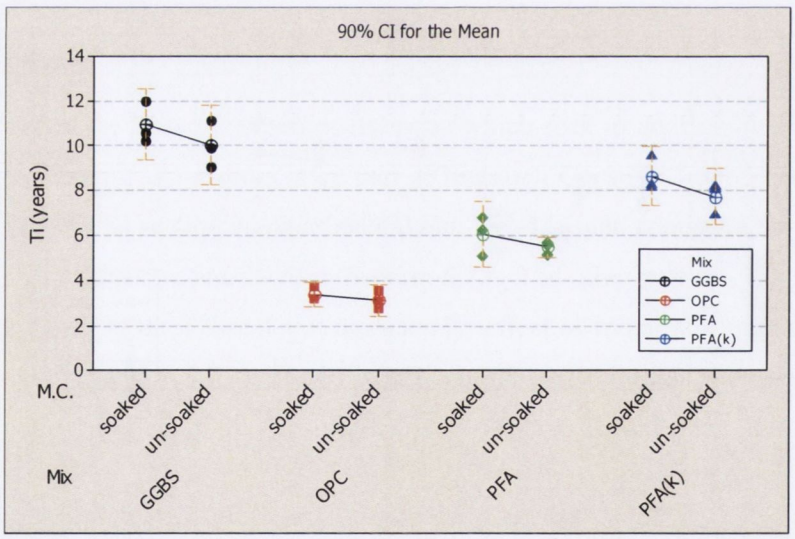


Figure 5.42 Soaked and un-soaked sample T_i values

There is a consistent reduction in T_i for the un-soaked samples. The lines connecting the means for each mix indicate that the GGBS and PFA(k) result in the greatest reduction in T_i values while the PFA would appear to show the third largest reduction in T_i for un-soaked samples. OPC illustrates the smallest reduction in T_i from soaked to un-soaked samples. It must be noted, however, that this pattern follows the increasing magnitude of the T_i values. Consequently, the relative merits of each mix, both soaked and un-soaked, are presented below in Table 5.7. This allows examination in the

percentage change in relative merit for each mix from un-soaked to soaked in terms of the OPC un-soaked specimen results.

Table 5.7 Change in relative merit due to pre-soaking

Durability Option	Mean Relative Merit $\left(\frac{T_i}{T_i OPC - US}\right)$	% Change from un-soaked to soaked
OPC-unsoaked	1.00	
OPC-soaked	1.08	+ 8%
PFA- unsoaked	1.76	
PFA- soaked	*1.95*	*+ 10%*
PFA(k)- unsoaked	*2.49*	
PFA(k)- soaked	*2.79*	*+ 12%*
GGBS- unsoaked	*3.23*	
GGBS- soaked	*3.52*	*+ 9%*

As can be seen from the table, the percentage change in the relative merit for the mixes is relatively consistent ranging from +8% for the OPC mix to + 12 % for the PFA(k) mix. It should be noted that these figures are plus or minus the effects of any experimental error and material variation. It is acknowledged that the initial soaking study detailed herein has limitations in terms of the number of samples studied and the large number of factors which may have affected results. Perhaps most important of these is the effect that an additional 13 days wet curing might have had on the durability characteristics of the soaked concrete samples. Unfortunately this cannot be separated from the initial absorption results. It is also noted that no statistically significant difference with 90% confidence could be detected between the un-soaked and soaked results. However, given the consistency of the results in Table 5.6 it can be said that the effect of the initial soaking, in terms of combined additional curing and initial absorption, would seem to be of the order of 10%.

The results detailed above would indicate that the assumption by Thomas and Matthews (2004) that initial absorption had a considerable influence on the relative performance of PFA concrete in their study was incorrect. It is, however, noted that it is not known how long the Thomas and Matthews specimens were left to dry before being exposed to the marine environment. Dhir et al. (1994), together with the other authors who pre-soaked specimens prior to testing (Basheer et al. 2008; Cheng et al. 2011; Dhir and Byars 1993; Dhir et al. 1994), did not consider unsoaked specimens. Consequently, it is not possible to compare the results obtained for the effect of initial absorption on natural chloride migration tests with similar studies in the existing literature.

The primary purpose of this experimental investigation was to explore the possibility that initial absorption had a large influence on the Phase 1 test results. The investigation suggests that initial absorption may have had a slight effect on the PFA and GGBS relative merits. However, based on the fact that the combined effect of an additional 13 days wet curing, and initial absorption, was found to be of the order of 10% it can be said that the effect of initial absorption on the Phase 1 relative merits was small. This section has, therefore, resolved this issue, and achieved Phase 2 experimental aim No. 1. It is noted that the achievement of this aim does not contribute directly to Research Question 1, but rather confirms that the findings in Chapter 4 relating to Research Question 1 were of sound basis.

5.7.3 OPC + Silane Results

Phase 2 experimental aim No. 2 sets out to identify the effect of silane application method on the variability of silane performance. As discussed in Section 5.2.3, the drying period adopted before the application of the silane surface treatment to the Phase 2 OPC-S specimens was not of sufficient length to comply with manufacturers guidelines. The short drying time should not, however, affect the primary purpose of studying the OPC-S durability performance variability arising from two different application procedures. This will be investigated in this section through comparison of the variation of performance within the Phase 1 and Phase 2 OPC-S sample sets. If the influence of spraying uniformity is small, one would expect the variation associated with the Phase 2 OPC-S results to be similar to that of the Phase 1 results. If however, the influence of spraying uniformity is large, one would expect a notable reduction in the variability of results in the Phase 2 sample set when compared to the two Phase 1 sample sets.

Figure 5.43, Figure 5.44 and Figure 5.45 show the scatter of the individual values of C_s, D_{app}, and T_i, across the three sample sets examined in both phases of testing. It is noted that the individual values are normalized by dividing them by the sample set mean value. This allows comparison of dimensionless scatter of the individual values with a sample set mean value of 1.0. In the plots, the *cu* represents the Phase 1 cubes while *sl* represents the Phase 1 slabs. Figure 5.46 presents the COV values for each parameter across the three OPC-S sample sets. Essentially this is a summary of the three previous plots; however Figure 5.46 does not provide information on individual value scatter.

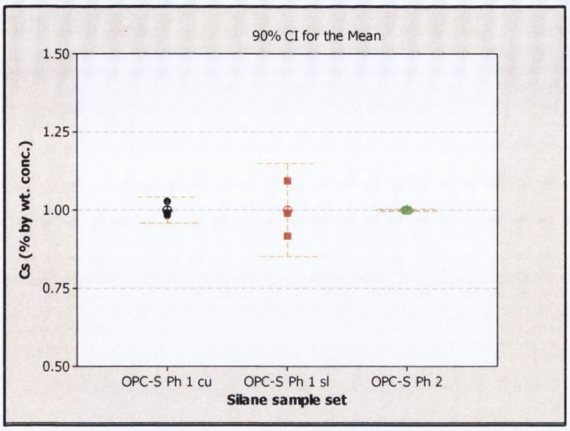


Figure 5.43 OPC-S C_s variation across three sample sets

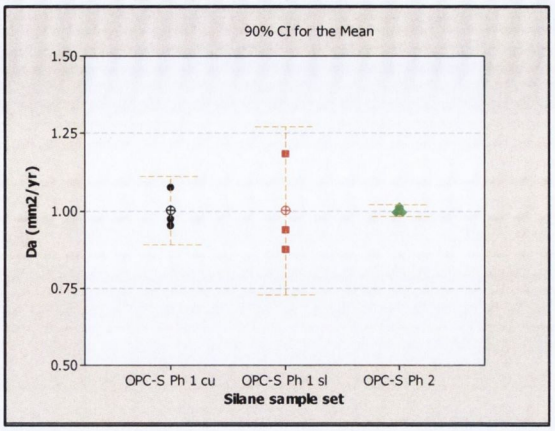


Figure 5.44 OPC-S D_{app} variation across three sample sets

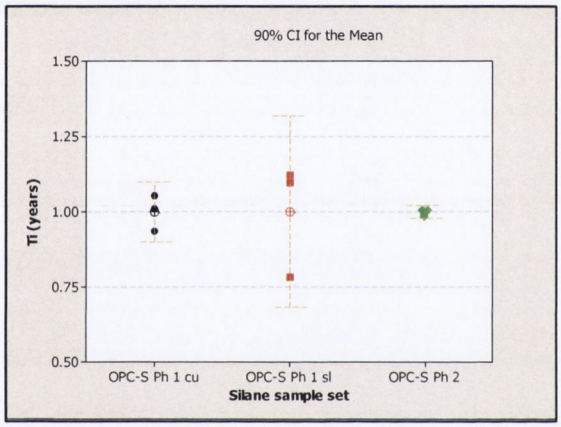


Figure 5.45 OPC-S T_i variation across three sample sets

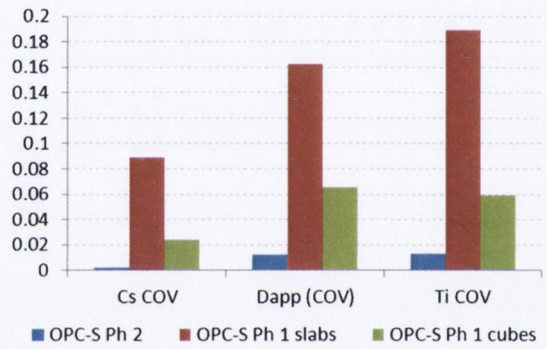


Figure 5.46 COV associated with OPC-S across three sample sets

As can be seen from the plots, the Phase 2 OPC-S samples displayed only a fraction of the variation associated with the Phase 1 OPC-S sample sets. Considering the T_i value COV it can be noted that the Phase 2 OPC-S COV is only 22% of the Phase 1 cube COV and only 7% of the Phase 1 slab COV. This represents a considerable improvement in consistency of results. Consequently, in terms of experimental aim No. 3, it can be said that the results presented above would indicate that the extent of uniformity of silane application has a notable influence on the variability of OPC SCC with silane surface treatment.

The silane performance literature comparison in Chapter 4 highlighted the variability in silane performance from study to study. It is possible that some of this variation is due to differences in silane application method employed from study to study. From a practical onsite perspective the results indicate that the nature of silane application on site will greatly affect durability, with spatial variability of protection across a structure likely to be an issue.

Having examined the effect of silane application method on performance variability it is noted that the failure to implement pre-application drying times in accordance with manufacturer's guidelines does not render the relative performance findings of OPC-S Phase 2 investigations useless. It is suggested that a 48 hour drying time is not entirely practical for an Irish marine bridge, much less for concrete elements in the splash zone and tidal zone. Thus the results for the Phase 2 OPC SCC + silane specimens are of some use in terms of service life prediction, particularly for tidal zone concrete elements. In addition the results will give an idea of the effect of the moisture condition of the sub-strata on the performance of silane through comparison of Phase 1 and Phase 2 results.

Figure 5.47 presents the OPC-S relative merit obtained from the Phase 2 salt fog chamber testing. The figure also presents the Phase 1 results for OPC-S. Firstly, considering the Phase 2 result, it is noted that the mean relative merit obtained for this option was 1.13. Thus the OPC-S performed only 13% better than the OPC SCC in the Phase 2 testing. Examining the performance of the OPC-S across the three sample sets, it is clear that the OPC-S option performed better in the Phase 1 testing than in the Phase 2 testing. The mean relative merit for the OPC-S dropped from between 1.75 and 1.78 in Phase 1 to 1.13 in Phase 2. The most likely cause of this performance reduction is the effect of the high moisture content in the Phase 2 specimens at the time of silane application. Basheer et al. (1998) investigated the effect of moisture condition upon silane application, on the chloride ingress resistance of concrete. The study involved applying silane to samples whose moisture content was dictated by three different moisture scenarios as follows; 1) three days soaking followed by three days drying at 20°C and 55% RH, 2) three days soaking followed by 4 weeks drying at 20°C and 55% RH and 3) three days soaking followed by 2 weeks drying at 40°C and 12% RH. Basheer et al. (1998) found that the moisture condition of the samples upon silane application affected the depth of silane penetration into the concrete, with greater penetration observed in the drier samples. The authors also found that four out of the five slab specimens which exhibited corrosion over the testing period were subjected to moisture scenario 1 prior to silane application.

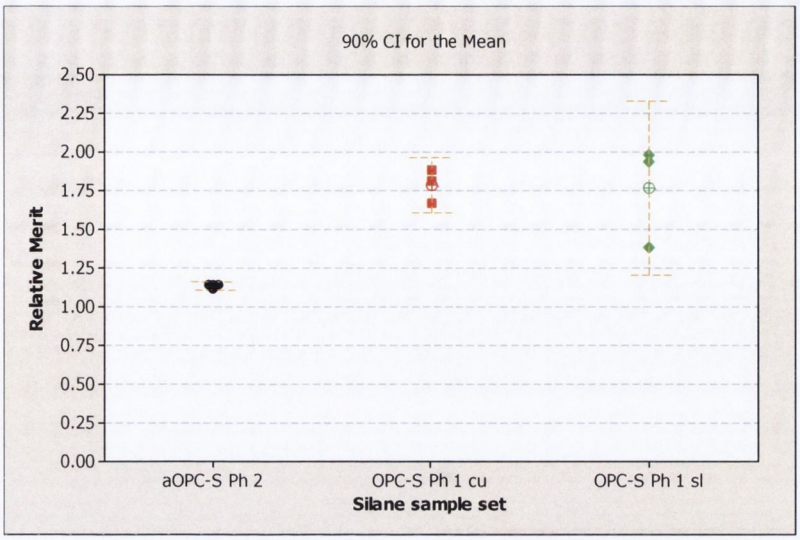


Figure 5.47 OPC-S relative merits for Phase 1 and Phase 2

Given the results of Basheer et al. study, and the known effect that concrete moisture content has on silane performance (de Vries and Polder 1995; Evonik Industries 2011), it seems clear that the reduction in OPC + silane performance from Phase 1 to Phase 2 testing was caused by the change in moisture condition of the sub-strata. However, in the context of the variation of silane performance identified in the literature in Chapter 4, it is interesting to observe the difference in performance across two moisture conditions which could easily be encountered in practice when applying silane to different areas of a marine structure. The discussion above relating to experimental aim No. 2 also indicated that the level of variation of silane performance is influenced by the uniformity of silane application. Thus, the Phase 2 study of OPC-S has highlighted two sources of performance variation which logic dictates would be an issue for on-site application of the surface treatment. This is an important consideration in the context of comparing the relative performance of different SCC options under core PhD Research Question 1.

5.7.4 Comparison of SCC Performance and VC Performance

The results presented in this sub-section seek to address a well-established need in the literature for data relating to the comparison of the marine durability of SCC and equivalent VC (Sonebi and Nanukuttan 2009; Zhu and Bartos 2003; Zong et al. 2011). The lack of existing knowledge in this area was established in the literature review, where Research Question 1, part b), was developed. In order to facilitate of clear presentation of results in this Chapter the task of addressing Research Question 1, part b) was designated as Phase 2 experimental aim No. 4 in Section 5.1. This aim, together with Phase 2 experimental aim No. 6, was identified as the most important Phase 2 experimental aim in the Introduction section of this chapter. As previously mentioned the

comparison of SCCs and VCs was carried out in the Phase 2 testing through examination of OPC, OPC + PFA and OPC + GGBS SCC and VC. The C_s and D_{app}, chloride profiles, T_i values and relative merits are presented for both SCCs and equivalent VCs in this sub-section. Subsequently, the performance of the SCCs relative to the equivalent VCs obtained herein will be compared to other studies in the literature.

Figure 5.48 below shows the C_s values obtained for both the SCCs and the VCs for the OPC concrete, the GGBS concrete and the PFA(k) concrete. Firstly, considering the VC and SCC separately, it can be noted from the plot that all three SCCs have almost identical C_s values ranging from 0.99% to 1.00%. For the VC the PFA(k) and GGBS mean C_s values are very similar at values of 1.20% and 1.15% respectively. The OPC VC mean C_s value is, however, slightly lower at a value of 1.04%.

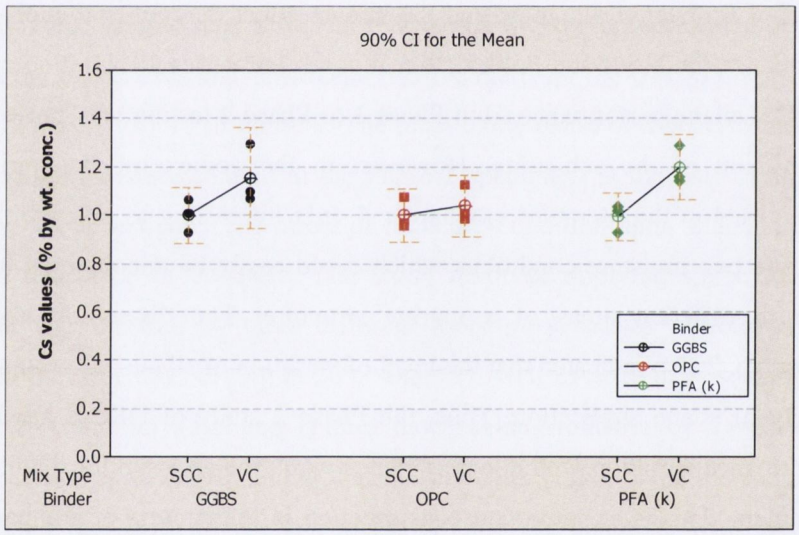


Figure 5.48: SCC and VC Cs values with 90% confidence intervals

As can be seen from the plot, the VCs result in a greater mean C_s value than the SCC for all the concrete mixes examined. This could be due to the fact that SCC is known to have a better surface finish than VC, with less surface defects (Sukumar et al. 2008). These results regarding the increased C_s values for VCs are interesting in the context of the lack of research in the literature examining SCC under natural chloride migration conditions, required to examine C_s values. In fact this work represents the first comparison of C_s values for SCCs and equivalent VCs which contain SCMs in the available literature. The percentage increases in the mean C_s values from SCC to VC for the OPC, PFA(k) and GGBS concretes are 4%, 20% and 15% respectively. Although the differences between the mean VC and SCC C_s values are considerable for the GGBS and PFA(k)

concretes, it is noted from the plot that due to C_s variation, none of the differences detected can be said, with 90% confidence, to represent a statistically significant difference in long run means.

Figure 5.49 below presents the D_{app} values obtained for the SCC and the VC. Interesting the VCs illustrate lower mean D_{app} values than the SCC for the OPC, OPC + PFA (k) and OPC + GGBS concretes. The other most notable point from Figure 5.49 is the large difference in D_{app} values for the OPC SCC and VC. For the OPC concretes the percentage reduction in the mean D_{app} value from the SCC to the VC is 43%. The corresponding figures for the PFA (k) concretes and the GGBS concretes are 30% and 19% respectively.

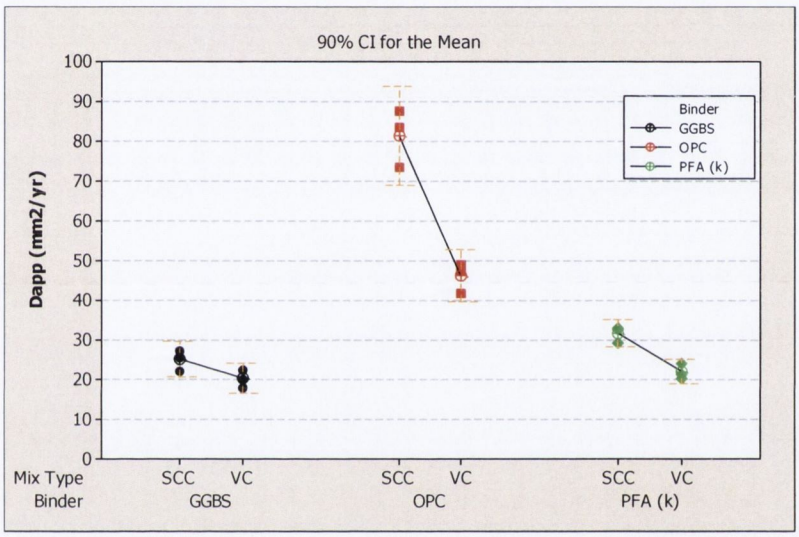


Figure 5.49 SCC and VC D_{app} values with 90% confidence intervals

It is probable that the increase in the D_{app} value from VC to SCC is due to the increased volume of cement paste in the SCCs. It is acknowledged that the movement of chlorides through concrete occurs primarily through the cement paste and through voids rather than through aggregates. Consequently, an increase in cement paste volume is likely to result in an increased migration of chlorides into concrete. It is notable that the percentage increase in the D_{app} value from VC to SCC is smaller for the PFA(k) and GGBS concretes. This is likely because the use of these SCMs in general results in a cement paste which is more resistant to chloride ingress than plain OPC paste, as has been established herein. Thus, the increase in paste volume in the SCCs which use these binders has less effect than the same increase in an OPC concrete. This theory is supported by the fact that the use of the GGBS binder, which has been shown herein to have the greatest durability performance, results in the smallest difference between mean SCC and VC D_{app} values.

From Figure 5.49 it can be seen that the differences detected between the SCC and VC D_{app} values for the OPC concretes and for the PFA(k) concretes indicate that the VC long run mean D_{app} value is statistically significantly lower with 90% confidence than the SCC long run mean D_{app} value. A statistically significant difference between the GGBS VC and the GGBS SCC D_{app} values was not detected with 90% confidence.

It should be noted that the VC specimens tested in the lab underwent perfect compaction having been poured in two layers, with each layer compacted using a vibrating table. It is unlikely that this degree of compaction would be uniformly achieved on site. Consequently, the performance of the SCC, relative to the VC, is likely to be somewhat improved if comparison was carried out using on site full scale members, due to the fact that the SCC is not subject to defects resulting from improper compaction in full scale members on site.

Figure 5.50, Figure 5.51 & Figure 5.52 compare the mean chloride profiles for the SCC and VC for the OPC concrete mixes, the PFA (k) concrete mixes and GGBS concrete mixes respectively. The SCCs are represented in the plots by the solid lines while the VCs are represented by the dashed lines.

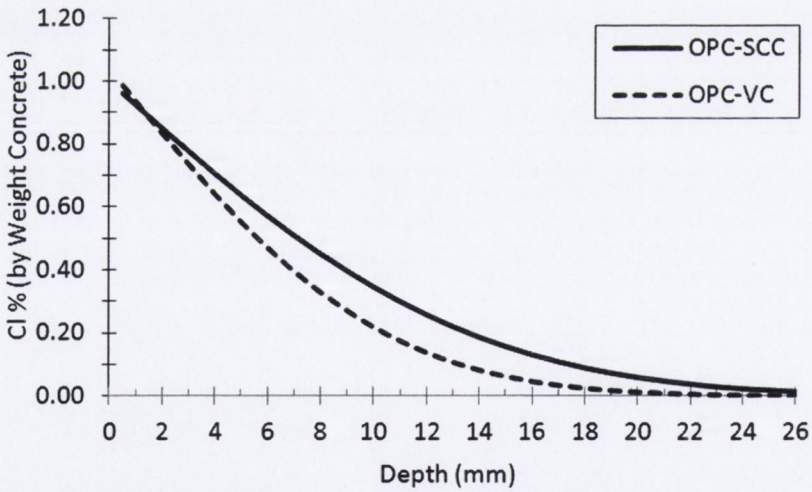


Figure 5.50 Mean chloride profiles for OPC SCC and VC

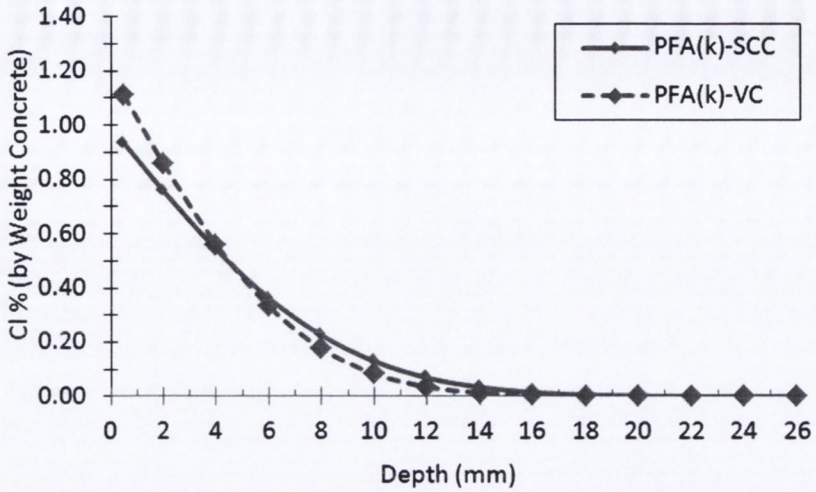


Figure 5.51 Mean chloride profiles for PFA SCC and VC

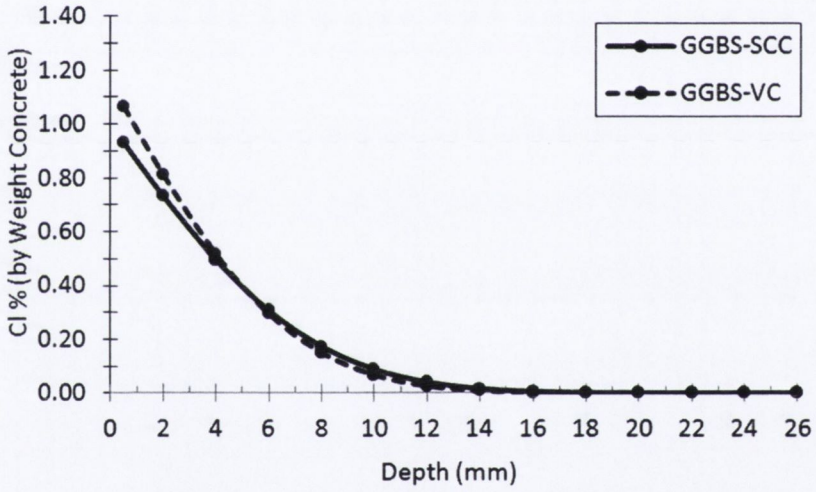


Figure 5.52 Mean chloride profiles for GGBS SCC and VC

Firstly considering the OPC concrete comparison in Figure 5.50 it can be noted that the SCC has a lower chloride content up to a depth of 2mm after which, the VC shows a lower chloride content. This is due to the fact that the increased C_s value leads to higher chloride content in the VC at the shallower depth. However, the lower VC D_{app} value leads to a greater reduction in chloride content with depth than that experienced in the SCC. Consequently, it is clear from Figure 5.50 that the OPC VC performs better than the OPC SCC over the exposure period. Examination of Figure 5.51 and Figure 5.52 reveal a similar pattern for PFA(k) concretes and the GGBS concretes. It is noted, however, that the difference in performance identified between SCC and VC is far less for these concrete mixes than experienced in the OPC concrete.

Figure 5.53 below shows the T_i values for the three SCCs and the three VCs. As can be seen from Figure 5.53 the SCCs result in shorter T_i values for each of the options considered. Examining the confidence intervals in the plot, it can be noted that the difference between SCC and VC mean T_i values obtained represent a statistically significant difference in long run mean T_i values with 90% confidence for the OPC and PFA(k) concretes, but not for the GGBS concretes. The reduction in mean T_i value from VC to SCC for the OPC concretes, the PFA(k) concretes and the GGBS concretes is 3.3 years, 3.7 years and 2.4 years, respectively. The percentage reduction in mean T_i value is greatest for the OPC concrete at 42%, compared to 24% and 14% for the PFA(k) and GGBS concretes respectively. This is primarily due to the influence of the large difference in the VC and SCC OPC D_{app} values discussed above in the context of Figure 5.49. This greater percentage reduction in T_i from OPC VC to SCC has consequences for the relative merits of the SCM materials when compared to OPC as seen in Figure 5.54.

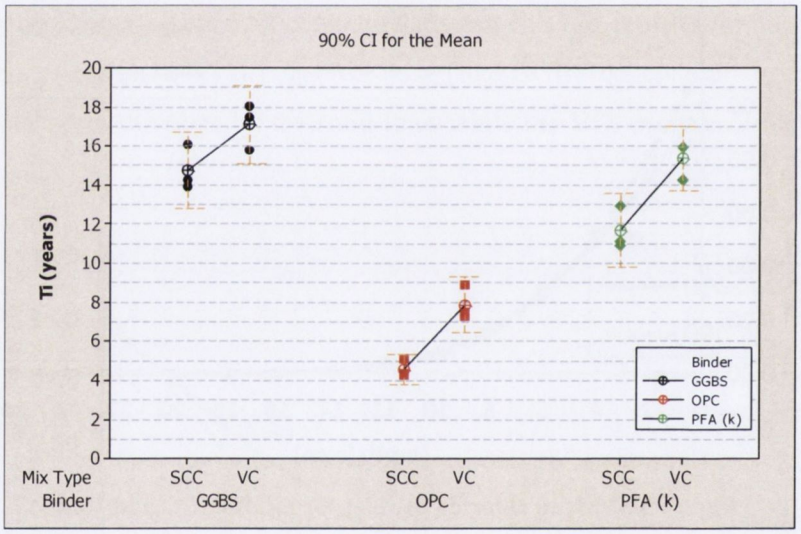


Figure 5.53 SCC and vibrated concrete T_i values with 90% confidence interval

Figure 5.54 illustrates that the relative performance of the PFA(k) and GGBS VC are notably lower than the equivalent VC. The mean VC and SCC GGBS relative merits are 2.2 and 3.2 respectively while the mean PFA(k) VC and SCC relative merits are 2.0 and 2.6 respectively. As previously discussed in the context of the D_{app} values in Figure 5.49, this increased relative performance of SCM in the SCCs is most likely due to the fact that the SCCs have a greater proportion of cement paste and, thus, the more durable SCM binders have a greater effect in the SCCs.

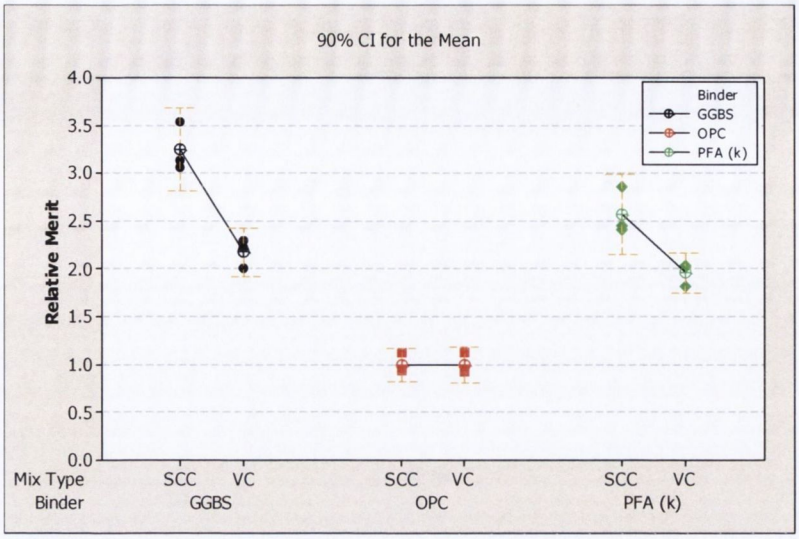


Figure 5.54 SCC and VC relative merits with confidence intervals

The final part of the SCC vs. VC analysis is presented below in Table 5.8. This table presents the mean T_i values for the SCC and VC which were plotted in Figure 5.53. Based on these values the performance of the VC relative to the SCC for each of the concrete mix types is calculated allowing performance comparison to be summarised in a single set of figures. The figures comparing SCCs to VCs have been termed relative performance values so that that they can be distinguished from relative merit values which have been used in this thesis to compare OPC concretes to alternative concrete options.

Table 5.8 VC/SCC relative performance Ti

Concrete Mix	T_i (years)	Mean Relative Performance $\left(\frac{T_i VC}{T_i SCC}\right)$
OPC-SCC	4.53	1.00
OPC-VC	*7.86*	1.73
PFA(k)-SCC	11.67	1.00
PFA(k)-VC	*15.38*	*1.32*
GGBS- SCC	*14.72*	*1.00*
GGBS-VC	*17.10*	*1.16*

Overall, from the relative performance values in Table 5.8 it can be noted that the difference in performance between the SCC and VC is greatest for the OPC concrete with a 73% increase in predicted T_i from SCC to VC. The corresponding figures for the PFA(k) and the GGBS concretes

are 32% and 16%. These findings are particularly interesting when it is considered that no study could be found in the literature which compared OPC, PFA and GGBS VCs and SCC with equal w/b ratios using natural chloride migration testing.

Having discussed all aspects of the SCC and VC results comparison, the findings will now be discussed in the context of results from existing literature. As previously discussed in Chapter 3, there is a recognised shortage of comparisons between SCC and VC in the available literature (Sonebi and Nanukuttan 2009; Zhu and Bartos 2003; Zong et al. 2011). The Phase 2 test results will, however, be compared to all forms of testing which examine SCC and VC, with the exception of the RCPT for reasons discussed in Chapter 3 relating to the unsuitability of this test for comparing concretes in a quantitative manner.

The literature comparison will be carried out using a relative performance dot plot which is labelled as Figure 5.55. First, however, the details of the studies utilised in the results comparison are presented in Table 5.9 to provide necessary background to the comparative study.

The relative performance plot is presented as Figure 5.55. The y-axis of the dot plot represents the relative performance identified in the publication. The x-axis of the dot plot presents the source study with the letters indicating the study grouping based on the type of testing adopted and whether the supplementary cementing material was used as a filler or cement replacement. The relative performance shown in the dot plot represents the performance of the VC relative to the SCC. The (a) group denotes the Phase 2 relative performances presented in Table 5.8 above. The (b) group denotes relative performances from natural diffusion testing which use supplementary cementing materials (SCMs) as OPC replacements. The (c) group represents results from testing employing electrical currents to accelerate chloride movement and which use SCMs as OPC replacements. Group (d) represents the results obtained from studies which use SCMs as filler materials and not as cement replacements and which utilise testing employing electrical currents to accelerate chloride movement. The line in the plot at 1.0 represents the point at which the performance of the VC and SCC are equal.

Table 5.9 Experimental details of studies used for literature comparison

Exposure	Exposure time (years)	Concrete mix			Relative performance VC perform / SCC perform	Notes	Reference
		Binder	w/b VC:SCC	w/b counting filler			
Phase 2 testing: Salt fog chambers	0.7	OPC	0.45:0.44	_	1.73		Current Study
		PFA (k)	0.45:0.44	_	1.32		
		GGBS	0.45:0.44	_	1.16		
Bulk Diffusion	0.14	OPC	0.35:0.40	_	1.38	w/b: SCC=0.40, VC=0.35	(Loser et al. 2010)
		PFA	0.35:0.40	_	1.21	PFA direct replacement	
		GGBS	0.35:0.40	_	0.67		
		OPC	0.45:0.40	_	0.76	w/b: SCC=0.40, VC=0.45	
		PFA	0.45:0.40	_	0.68	PFA direct replacement	
		GGBS	0.45:0.40	_	0.55		
Cyclic wetting test	up to 0.7	OPC	0.46:0.46	0.28:0.46	0.79	240 kg limestone filler used in SCC	(Audenaert et al. 2005)
		GGBS	0.46:0.46	0.28:0.46	0.59	240kg limestone filler used in SCC	
		OPC (PFA filler)	0.46:0.46	0.28:0.46	0.48	240kg PFA filler used in SCC	
RCMT	Hrs-Days*	OPC	0.42:0.41	_	1.72		(Zhu and Bartos 2003)
		PFA	0.36:0.35	_	0.95	Filler in terms of w/b but not in terms of total cement content (See Chapter 3)	
		OPC	0.57:0.58	_	1.25		
		PFA	**0.48:0.41**	_	0.65	Filler in terms of w/b but not in terms of total cement content (See Chapter 3)	
RCMT Non-steady state	up to 0.7	OPC	0.46:0.46	0.28:0.46	0.89	240 kg limestone filler used in SCC	(Audenaert et al. 2010)
		GGBS	0.46:0.46	0.28:0.46	0.43	240kg limestone filler used in SCC	
		OPC (PFA filler)	0.46:0.46	0.28:0.46	0.02	240kg PFA filler used in SCC	
RCMT Non-steady state	Hrs-Days*	OPC	0.40:0.35	_	3.39	w/b: SCC=0.40, VC=0.35	(Loser et al. 2010)
		PFA	0.40:0.35	_	1.67	PFA direct replacement	
		GGBS	0.40:0.35	_	1.50		
		OPC	0.40:0.45	_	0.79	w/b: SCC=0.40, VC=0.45	
		PFA	0.40:0.45	_	0.56	PFA direct replacement	
		GGBS	0.40:0.45	_	0.30		
Steady state Migration test	Hrs-Days*	OPC & OPC + LP	0.50:0.55	0.50:0.50	1.09		(Assié et al. 2007)
Steady state migration	Hrs-Days*	OPC + 25% PFA + 5% SF	0.42:0.42	_	1.31	No OPC Concrete tested	(Hwang and Khayat 2009)
		OPC + 25% PFA + 5% SF	0.42:0.42	_	0.96		

Hrs-days*: means test duration of the order of hours to days, LP = Limestone Powder,

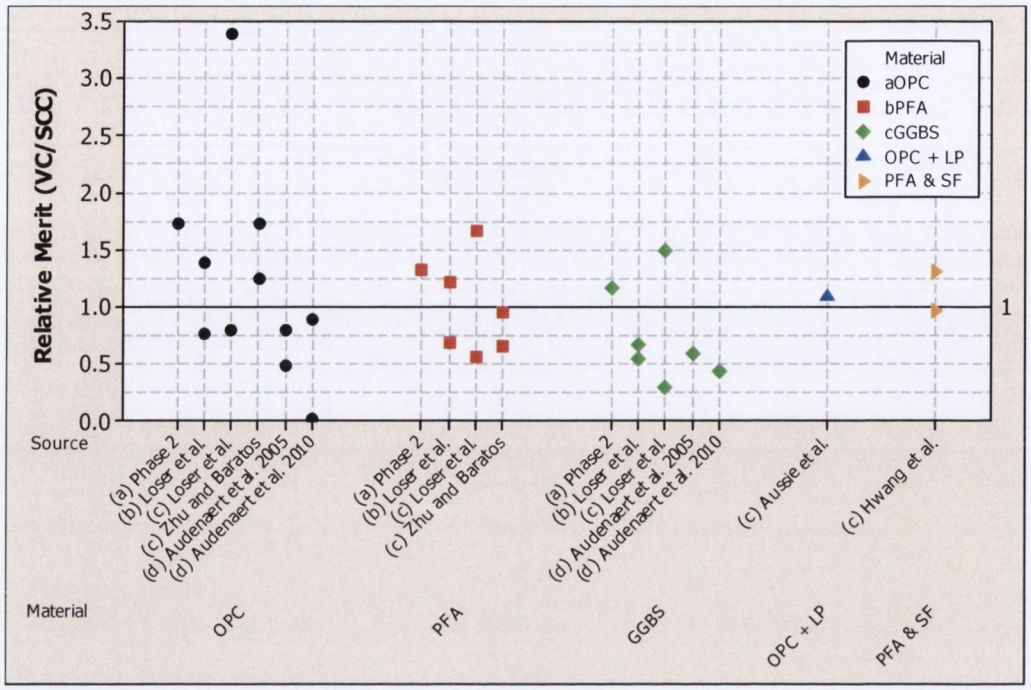


Figure 5.55 Literature comparison of VC/SCC relative performance

Firstly, considering the OPC VC/SCC relative performance it can be noted that of the 11 points plotted, 5 are above the 1.0 line and 6 are below the 1.0 line. The points below the line from group (b) and (c) represent a comparison between a VC with a w/b ratio of 0.45 and a SCC with a w/b ratio of 0.4. By definition the group (d) points represent studies which utilised SCM as a filler. The two lowest (d) points plotted represent a SCC which utilised PFA as a filler material. For the two other group (d) points plotted the filler material was limestone powder. The limestone powder and PFA is not considered as a SCM by the authors of these publications. If, however, one considers the binder contribution of the limestone power or PFA in these studies the SCCs have a far greater binder content than the VC with which they are compared.

It is noted that due to the testing methods adopted to examine SCCs to date, none of the studies plotted above have compared the C_s values of SCCs and VCs. Consequently, the relative performance points plotted above, with the exception of the Phase 2 test data, are based on chloride diffusion coefficient and chloride migration coefficients. As discussed in the context of Figure 5.48 above the C_s values of SCC were found to be lower than the equivalent VC C_s values for GGBS and PFA(k) concretes examined herein.

Considering the PFA VC/SCC comparison, it is noted that of the seven data points plotted, four indicate a superior SCC performance. The two points from the Loser et al. (2010) study, and the lower point from the Zhu and Bartos (2003), are derived from SCC and VC comparisons where the

SCCs have lower w/b ratios as can be seen from Table 5.8. Thus once again a somewhat false impression of the enhanced durability of SCCs is given.

Considering the GGBS VC/SCC comparison it can be seen that of the seven points plotted five of the points are less than 1.0. The Phase 2 GGBS value is also quite close to 1.0 at 1.16. The other GGBS point greater than 1.0 is the RCMT result by Loser et al. It is noted from the plot that for the GGBS concretes there is poor agreement between the RCMT results and the bulk diffusion test results obtained by Loser et al. from identical concretes. This possible issue with accelerating chloride migration with an electrical current has been discussed in Chapter 4 and will not be discussed again in the interest of brevity.

It appears from Figure 5.55 that SCC which utilises GGBS as a direct cement replacement are almost equal to, or superior to, equivalent VCs in terms of chloride transport properties. This change is likely due to the enhanced durability characteristics of OPC + GGBS cement paste when compared to OPC cement paste. However, in order to surpass an equivalent VC in terms of performance, the SCC would also require fewer air voids than the VC. This is because the portion of increased cement paste in the SCC will have higher chloride diffusivity characteristics than the equivalent volume of aggregate which it replaces in a VC, regardless of the binder used. Consequently, it is likely that greater homogeneity and reduced voids, combined with the reduced chloride diffusion in the cement paste, can result in an increased durability performance for GGBS SCCs when compared to GGBS VCs.

Finally, due to the shortage of literature in this area two studies are also included which utilise concrete types not studied herein. As can be seen from the plot Aussie et al. (2007) found the VC to be superior. This is likely due to the reduced cementing power of limestone powder when compared to OPC. The Hwang and Khayat (2009) study found that for the concretes studied, one showed SCC to be superior while the other showed VC to be superior.

The interpretation of the Phase 2 testing results in the context of the studies in the literature provides a clear picture of the comparative durability of SCC and VCs. This comparison suggests that for both OPC concretes and PFA(k) concretes, when SCC and VC w/b ratios are equal, and SCMs are not used as a filler in SCC, VC are more durable if sufficiently compacted. For GGBS the comparison suggests that the SCC performance may be equal to or superior to the VC for the same conditions. This is a highly relevant finding when it is considered that publications on SCC, which do not necessarily examine durability, often state that SCC results in better quality more durable concrete than VC (Kasemchaisiri and Tangtermsiriku 2008; Tande and Mohite 2007; Zhao

et al. 2012; Zhao et al. 2012). Thus, it is sometimes assumed that SCC is in all cases more durable than VC. As shown herein, however, this is not necessarily the case.

Overall the results presented in this sub-section can be said to have addressed Research Question 1, part b) through the achievement of the Phase 2 experimental aim No. 4. In addition, given a) the shortage of research in this area b) the scale of the experimental programme carried out herein, and c) the test methods utilised in the experimental programme, the work detailed above constitutes a significant contribution to the existing knowledge in the area of the durability of SCCs in chloride rich environments.

5.7.5 *k*-Value Concept for PFA (Efficiency Factor)

As discussed in Section 5.1, the work described in this section addresses Research Question 1, part a) by further investigating OPC+PFA SCCs through examination of the influence of the *k*-value efficiency factor on chloride transport properties of SCC. The analysis involves comparison of the C_s and D_{app} results for OPC, OPC + PFA and OPC + PFA(k) SCCs. Following this assessment the chloride profiles will be compared to assess performance before predicted T_i values and relative merit values are for the three SCCs are presented. The OPC + PFA performance relative to the OPC performance will then be compared with the results available in the existing literature.

Figure 5.56 below presents the SCC C_s values for OPC, PFA and PFA(k).

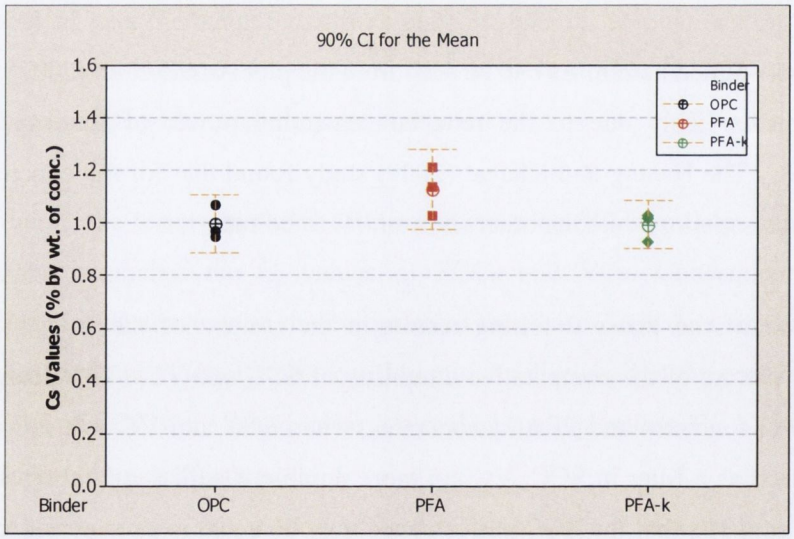


Figure 5.56: SCC C_s values and confidence intervals for OPC, PFA and PFA (k)

As can be seen from the plot the OPC and PFA (k) have similar C_s values with the mean C_s values almost identical at 1.00% and 0.99%, respectively. The PFA option has a slightly increased C_s mean value of 1.13%. It can be noted from the plot that a statistically significant difference cannot be detected between the three C_s values with 90% confidence.

Figure 5.57 presents the D_{app} values for the OPC, PFA and PFA(k) SCCs. As can be seen from the plot, the use of PFA as a direct OPC replacement in the Phase 2 SCC resulted in a notable reduction in D_{app}. The figure below shows that the addition of PFA utilising the *k*-value efficiency factor results in a further reduction in the D_{app} value. The OPC, PFA and PFA(k) D_{app} mean values are 81.6 mm^2/yr, 43.6 mm^2/yr and 31.7 mm^2/yr, respectively. It is thus noted that the addition of PFA as a direct replacement for OPC has reduced the mean D_{app} value by 46% while the replacement of OPC using the *k*-value concept has reduced the D_{app} value by 61%. From the interval bars in Figure 5.57 it can be said with 90% confidence that the PFA concrete has a statistically significant lower D_{app} long run mean value than the OPC concrete, while the PFA(k) concrete has a statistically significant lower long run mean than the PFA concrete.

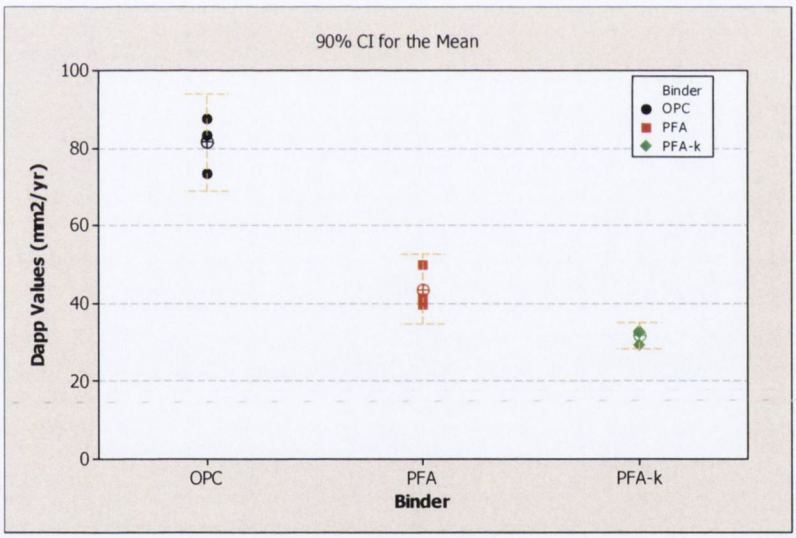


Figure 5.57: SCC D_{app} values and confidence intervals for OPC, PFA and PFA (k)

The average chloride profiles for the concrete samples OPC, OPC + PFA and OPC + PFA(k) are presented below in Figure 5.58. It is noted that the increased C_s value for the PFA SCC results in this durability option having the highest chloride content up to a depth of 4mm. As shown in Figure 5.56 the PFA SCC option has a lower mean D_{app} value than the OPC SCC option resulting in OPC having the greatest chloride content for the remainder of the depth profile. It is clear from the figure that the results of the Phase 2 testing indicates that when considering marine durability, the use of PFA as a direct OPC replacement results in a notable improvement over plain OPC concrete, while

the use of PFA with the *k*-value concept results in further improvement. It is noted, however, in the context of the discussion in Section 5.6.3, that this is based on exposure condition 3, i.e. the wettest of the exposure conditions examined in this thesis. Interestingly, when PFA has been utilised as a direct replacement for OPC in both the Phase 1 and Phase 2 testing, it has resulted in a higher C_s value than the OPC durability option. This is possibly due to the greater amount of absorption and evaporation in the PFA SCC discussed in Section 5.5.2. The use of PFA with the *k*-value implemented has resulted in an almost equal C_s value for OPC and PFA(k).

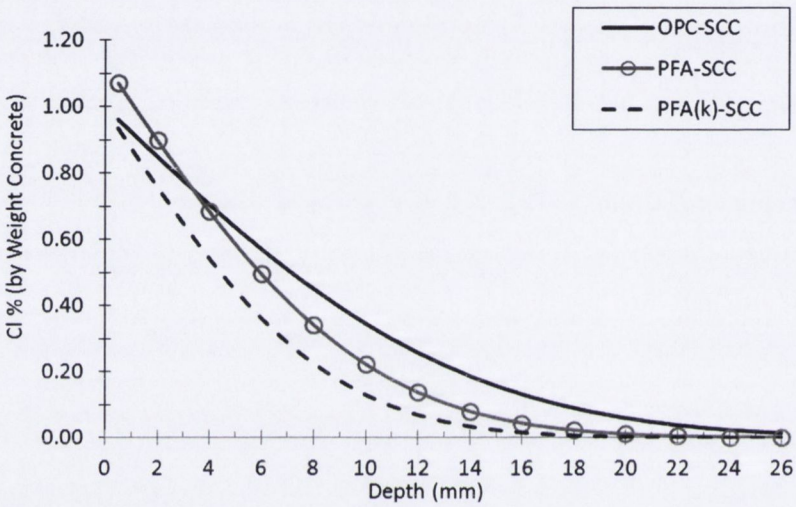


Figure 5.58 Chloride profiles for OPC, PFA and PFA(k) SCC options

The relative merit of the concretes aimed at examining the *k*-value will now be assessed in quantifiable form by comparing the predicted time to initiation of corrosion for each option using the T_i calculation procedure previously outlined. The T_i values and relative merits are shown in Table 5.10, with relative merits also presented in graphical form in Figure 5.59.

Table 5.10 Calculated mean T_i values and mean relative merits

Durability Option	T_i (years)	Mean Relative Merit $\left(\frac{T_i}{T_i OPC(VCorSCC)}\right)$
OPC-SCC	4.5	1.00
PFA-SCC	8.1	1.78
PFA(k)-SCC	11.7	2.57

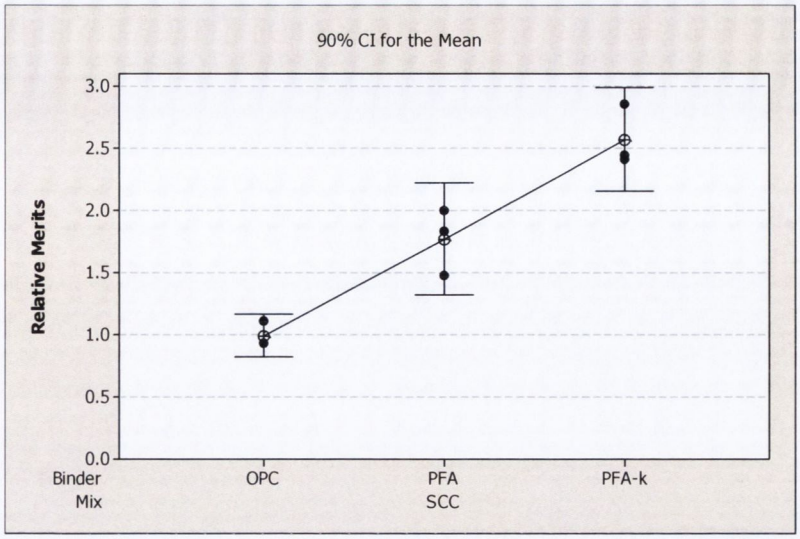


Figure 5.59 Relative Merits for OPC, OPC + PFA and OPC + PFA(k) SCCs

It can be seen from the plot that the addition of PFA as a direct replacement for OPC in Phase 2 has resulted in the predicted T_i value increasing by 78%. It is, therefore, clear that the chloride resistance efficiency factor for OPC + PFA concrete is greater than 1. If, however, the efficiency factor for PFA of 0.4 is employed in accordance with EN206-1:2000 it is noted that the T_i value for the OPC + PFA concrete is approximately 2.6 times greater than plain OPC SCC. The slope of the SCC relative merit mean connect line indicates that the improvement from OPC SCC to PFA SCC is approximately equal to the improvement from PFA SCC to PFA(k) SCC. As indicated by the confidence interval bars in Figure 5.59, there is a significant variation associated with the individual PFA and PFA(k) SCC relative merit results. However, the difference between the OPC performance and the performance of the two OPC + PFA SCCs represents a statistically significant difference in long run mean performance with 90% confidence. Again it is re-iterated that the discussion in Section 5.6.3 would indicate that the relative performance of OPC + PFA concretes will be reduced somewhat in exposure conditions which incorporate a greater amount of drying than those utilised in the Phase 2 experimental programme.

Having presented the relative merits obtained herein for OPC + PFA with and without the *k*-value it is now necessary to compare these results to the existing literature. This is somewhat difficult for SCC, given the shortage of studies which utilise appropriate test methods to examine the use of PFA as an OPC replacement (as discussed in Chapter 3). There are studies published in the literature which use the RCPT to assess the use of PFA as a direct replacement for OPC replacement (Gesoglu et al. 2009; Guneyisi et al. 2011; Pathak and Siddique 2012; Sahmaran et al. 2009). However, as discussed in Chapter 3, this is not an appropriate test method for assessing relative performance in a quantitative manner. Examples of examination of the *k*-value concept do

exist for, but these studies compare OPC + PFA(k) to OPC only. Table 5.11 below presents details of experimental programmes which utilise natural chloride migration in a laboratory setting, and in the marine environment, to examine OPC and OPC + PFA VCs with the *k*-value efficiency factor implemented. The table also gives details of some studies which examine PFA as a direct replacement for OPC. The author could not find a publication in the available literature which examined OPC + PFA with and without the *k*-value specified in EN206-1:2000 using natural chloride migration testing.

Table 5.11: Experimental Details for *k*-value comparison

Exposure	Exposure time (years)	Concrete mix			Conc. Type	Relative merit	Notes	Reference
		Binder	w/(c+*k*(PFA)	w/b				
Phase 2 testing: Salt fog chambers	0.7	OPC	0.44	0.44	SCC	1.00		Current Study
		PFA	0.54	0.44	SCC	1.78		
		PFA(k)	0.44		SCC	2.57		
		OPC	0.44		VC	1.00		Current Study
		PFA(k)	0.44		VC	1.96		
Salt Spray		OPC	0.45	0.45	VC	1.0	Water soluble chlorides examined	(Dhir et al. 1994)
		PFA(k)	0.45	0.35	VC	1.5		
Marine exposure site specimens (Splash Zone)	1,3	OPC	0.66	0.66	VC	1.0, 1.0	*k*-value of 0.31 used. Specimens exposed in the splash zone	(Thomas and Bamforth 1999)
	1,3	PFA	0.64	0.54	VC	1.7, 4.5		
Submerged Samples	0.5	OPC	0.57	0.57	VC	1.0	Samples submerged in 5M NaCl for 6 months. *k*-value of 0.4 used	(Dhir et al. 1993)
	0.5	35% PFA	0.57	0.45	VC	3.75		
STUDIES WITHOUT *K*-VALUE APPLIED								
Bulk Diffusion Tests	0.14	OPC	0.40	0.40	SCC	1.0	*k*-value not utilised, PFA used as direct replacement	(Loser et al. 2010)
	0.14	PFA	0.45	0.40	SCC	1.3		
Ponding	0.9	OPC	0.5	0.5	VC	1.0	*k*-value not utilised, PFA used as direct replacement	(McPolin et al. 2005)
	0.9	PFA	not stated but > 0.5	0.5	VC	2.1		
Bulk Diffusion Tests	0.14	OPC	0.45	0.45	VC	1.0	*k*-value not utilised, PFA used as direct replacement	(Loser et al. 2010)
	0.14	20% PFA	0.51	0.45	VC	1.2		
Bulk Diffusion Tests	0.5, 1.0, 4.0	OPC	0.5	0.5	VC	1.0, 1.0, 1.0	Submerged for varying exposure times 3 months after curing	(Stanish and Thomas 2003)
	0.5, 1.0, 4.0	PFA	0.59	0.5	VC	4.0, 6.2, 5.7		

The relative merits obtained from the Phase 2 testing and from the literature are presented as dot plots in Figure 5.60 below. It is noted that the relative merit obtained for the OPC + PFA(k) VC herein is included in the plot. The studies are divided into groups as follows: (a) Phase 2

experimental test results, (b) studies that utilise the *k*-value efficiency factor, (c) SCC studies which utilise PFA as a direct replacement, (d) VC studies which utilise PFA as a direct replacement.

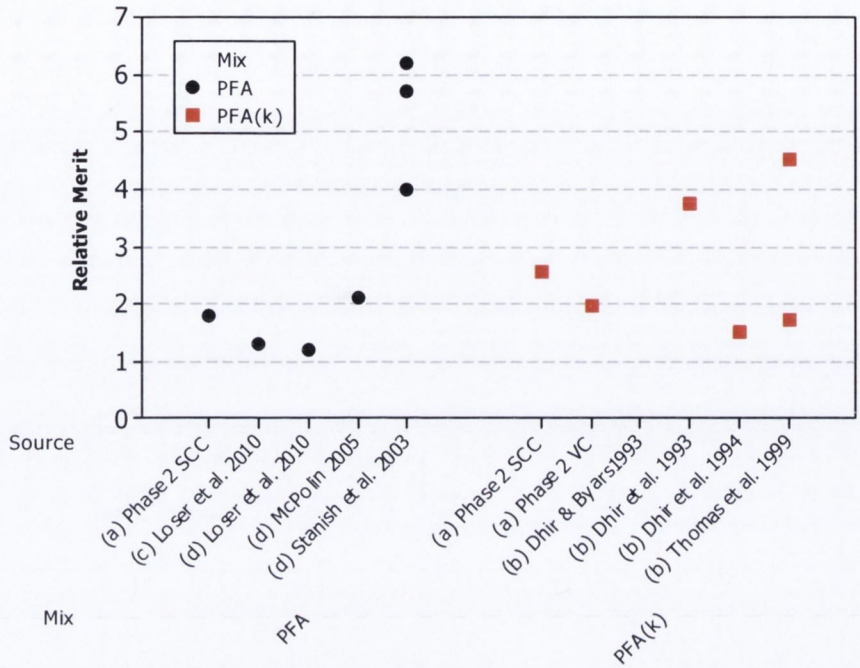


Figure 5.60 Phase 2 SCC and VC relative merits for PFA and PFA(k) compared to those obtained from the literature (0 – 7)

From Figure 5.60 it is noted that the Stanish and Thomas results yield the greatest relative merit. This may be due to the fact that the samples were cured for three months prior to exposure. This would have resulted in a greater performance in the OPC + PFA concretes due to the slower rate of hydration in concretes containing PFA. Overall, considering the data points from the literature, it can be said that the addition of PFA both as a direct replacement, and as a replacement using the *k*-value, has generally been found to result in an improved performance over that of plain OPC. Yet, it is not possible to utilise the data from the literature to quantify the difference in performance between PFA and PFA(k) concretes due to; a) the influences of time dependant diffusion and concrete age upon testing (highlighted by the data points from the Stanish and Thomas study and the Thomas and Bamforth study) and b) the influences of differences in test methods and exposure conditions used to assess the concretes across different studies (shown in Section 5.6.3 to influence relative merit). This emphasises the need to assess OPC, PFA and PFA(k) concretes in a single experimental programme as carried out herein.

Overall, the examination of the *k*-value efficiency factor presented above has achieved experimental aim No. 3, and thus further addressed Research Question 1, part a), by providing

additional information relating on how OPC + PFA SCCs options perform in the initiation phase of corrosion initiation.

5.8 SALT FOG CHAMBER RESULTS: RESEARCH QUESTION 3

5.8.1 Ferrycarrig Bridge Original Mix Results

As established in the introduction of this chapter, the examination of the mix designed to mimic the original Ferrycarrig Bridge (Ferr. Orig.) concrete in this chapter, does not directly address Research Question 3. Instead, obtaining chloride transport properties for this mix facilitates investigation into calibration of salt fog chamber testing in a probabilistic framework using real marine environment data in Chapter 7. The Ferr. Orig. concrete results are presented in this chapter merely to provide an indication of the durability of the original Ferrycarrig Bridge concrete. To this end the results of the Ferr. Orig. mix will be presented in this section together with the results for the OPC SCC and the OPC VC. This is done to facilitate a comparison of the Ferr. Orig. mix with the other OPC concrete mixes tested.

Figure 5.61 below shows the C_s values for OPC SCC, Ferr. Orig. and OPC VC. As can be seen from the plot, the OPC VC has the highest C_s levels with a mean value of 1.25%. The OPC SCC and Ferr. Orig. mean C_s values are almost identical at 1.00% and 1.02%, respectively. It is noted that one of the Ferr. Orig. C_s values is far greater than the mean and appears to be an outlier. Yet, upon analysis of the test data, no reason could be found to justify rejection of this apparent outlier and thus, it is included in the Ferr. Orig. C_s value mean herein.

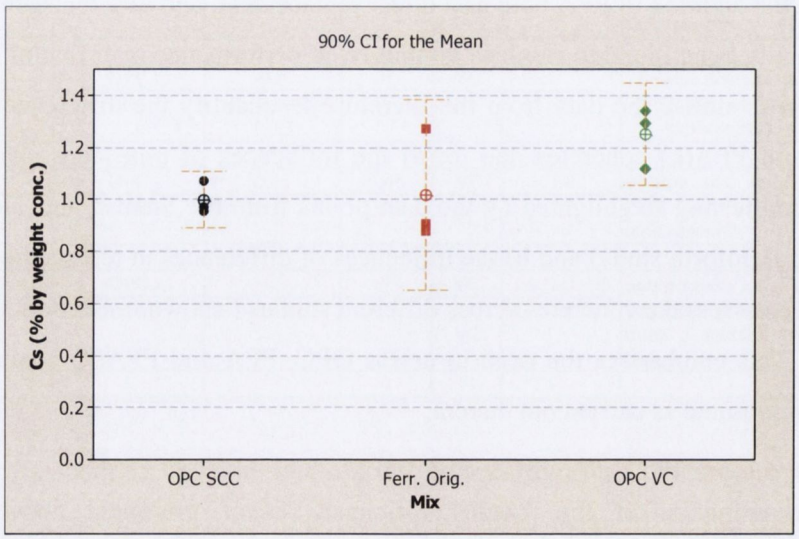


Figure 5.61 C_s value comparison of OPC SCC, Ferr. Orig. and OPC VC

Figure 5.62 below presents the D_{app} values for the three concrete mixes. As can be seen, the Ferr. Orig. mix has only a slightly lower D_{app} value than the OPC SCC. The two mixes have mean D_{app} values of 74.00mm^2/yr and 81.47mm^2/yr. Thus, the Ferr. Orig. D_{app} value is 9% smaller than the OPC SCC D_{app} value. The D_{app} value obtained for OPC VC is 53% smaller than the Ferr. Orig. D_{app} value. As can be seen from the plot, this difference represents a statistically significant difference between the two mixes long run mean D_{app} values with 90% confidence.

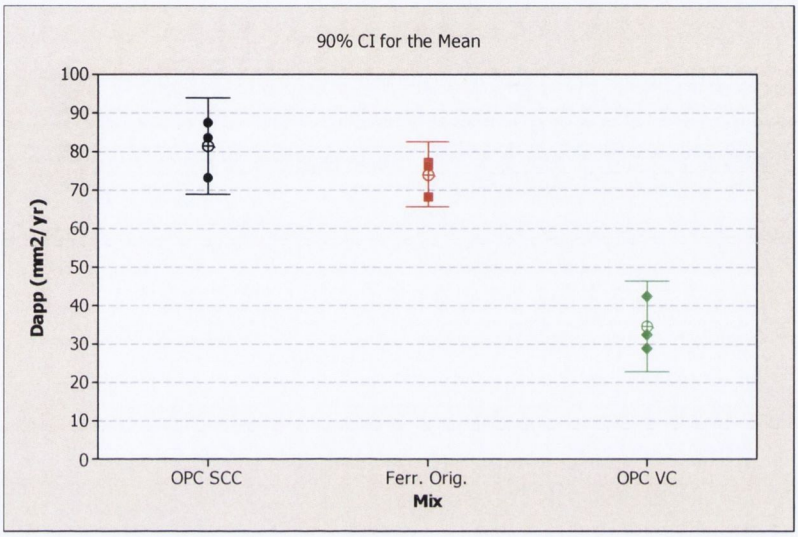


Figure 5.62 D_{app} value comparison of OPC SCC, Ferr. Orig. and OPC VC

The average chloride profiles for the three concretes are presented below in Figure 5.63. The Ferr. Orig. chloride profile is almost identical to the OPC SCC profile. The effect of the Ferr. Orig. slightly lower D_{app} value can, however, be seen in the plot. The plot also shows that the OPC VC performs far better than the Ferr. Orig. mix, due to the fact that the OPC VC D_{app} value is approximately half that of the Ferr. Orig. mix, as seen in Figure 5.62.

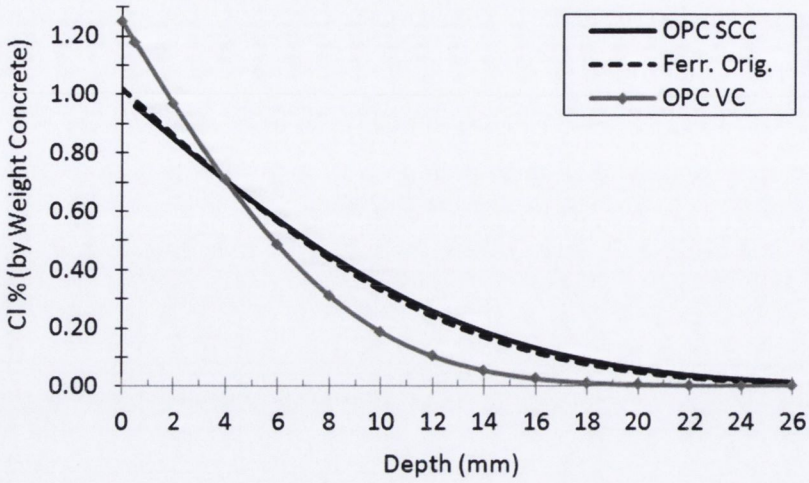


Figure 5.63 Chloride profiles for OPC SCC, Ferr Orig. and OPC VC

As a final comparison, the T_i values for the three concretes are presented below in Figure 5.64. The mean T_i value for the OPC VC is approximately twice that of the Ferr. Orig. with mean T_i values of 9.74 years and 4.96 years respectively. The superior performance of the OPC VC is to be expected given the higher w/b ratio and lower strength grade of the Ferr. Orig. mix. The Ferr. Orig. T_i was found to be very similar to that of the OPC SCC which had a predicted T_i value of 4.53 years. This represents a 9% reduction when compared Ferr. Orig. concrete.

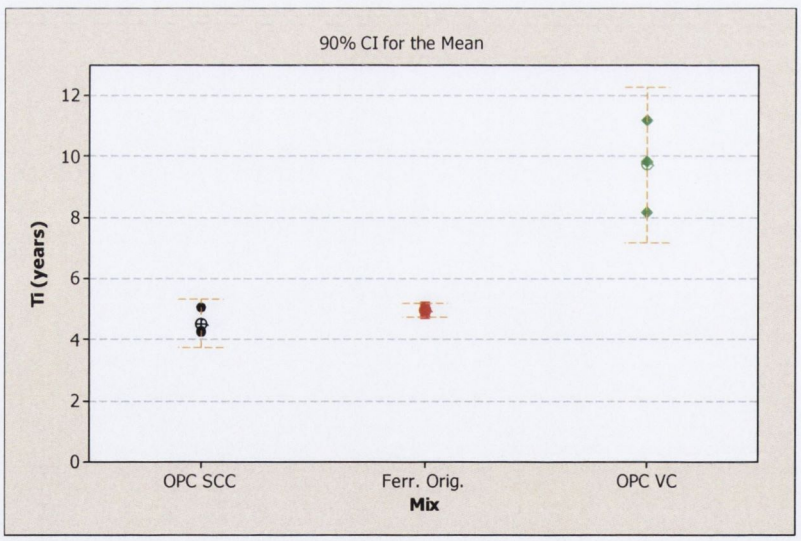


Figure 5.64 T_i comparison of OPC SCC, Ferr. Orig. and OPC VC

The discussion above simply presents the Ferrycarrig original mix results and compares them to the other OPC results in order to contextualise the values obtained. There is little point in comparing

the Ferr. Orig. mix to the other concrete mixes, or to results from the literature as the testing of this mix is solely to facilitate a calibration of the accelerated results, utilising field data in Chapter 7.

5.9 PHASE 2 CONCLUSIONS

Research Question 2:
Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

The literature review in Chapter 3 established that very limited research has been carried out to date investigating the effect of exposure conditions on the relative performance of different concrete types. This is likely due to the fact that the literature is dominated by concrete option comparisons tested in saturated conditions. The practical importance of studies in unsaturated conditions is clear when it is considered that corrosion of reinforcement does not in general occur in fully saturated reinforced concrete elements, due to the low level of oxygen available (Neville 2011; Richardson 2002). It is, therefore, important to assess the effect of varying exposure conditions (i.e. varying levels of drying) on concretes which have been established as being more durable based primarily on saturated experimental testing.

The presentation of the relative merits across the three different exposure conditions presented in this chapter, combined with analysis of the absorption, sorptivity and drying test results indicates that, when compared to OPC SCC, the relative performance of both OPC + PFA SCC and OPC + GGBS SCC are reduced with increased levels of drying. Analysis of the only other similar study in the literature (Ben Fraj et al. 2012), which examined OPC VC and GGBS VC, identified similar patterns to those found in this chapter for OPC SCC and OPC + GGBS SCC.

Overall in terms of performance the results presented herein indicated that OPC + GGBS concretes in particular, are likely to be more durable than OPC concretes regardless of exposure conditions, but, it is important to investigate the scale of reduction in relative performance of these durable concrete options under various exposure conditions. The comparison of OPC and OPC + PFA SCCs, however, indicated a more critical situation with the OPC + PFA performance found to be poorer than OPC in the Phase 1 SKWT 4300 testing (most drying) and 80% better than OPC in the Phase 2 SKWT4300 testing (least drying).

Research Question 1:

How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

g) Comparison different SCC options

h) Compare SCC to the more established traditional vibrated concrete type

Having addressed a substantial part of Research Question 1 a) in Chapter 4, the primary focus of this chapter, in the context of Research Question 1, was to address part b), through comparison of the marine durability of SCCs and VCs. As discussed in Chapter 3, it is recognised that SCC is, at present being used in industry without insight into the intrinsic durability of the material itself (Rilem Technical Committee et al. 2008). Consequently, it has been stated by a number of researchers that there is a need for studies comparing SCCs and equivalent VCs (Sonebi and Nanukuttan 2009; Zhu and Bartos 2003; Zong et al. 2011). The majority of the studies in the literature comparing various SCC durability options to equivalent VC options utilise test methods which incorporate the use of an impressed current to accelerate chloride movement. The results presented in this chapter compared SCC and VC under natural chloride diffusion conditions. The following are the key findings from this SCC vs. VC comparison:

For OPC concretes the VC was found to have a substantially better marine durability performance than the equivalent SCC, with the VC resulting in a 73% increase in T_i, when compared to the SCC. The performance of the SCC relative to the VC was found to improve for concretes containing PFA and GGBS. The increase in predicted T_i from SCC to VC was to 32% and 16% for the OPC + PFA and OPC + GGBS concretes, respectively. The likely cause identified for this reduction in the difference between SCCs and VCs from OPC to OPC + PFA(k) to OPC + GGBS was the increasing chloride resistance of the cement paste from OPC concrete to OPC + PFA(k) concrete to OPC + GGBS concrete.

It was also noted that publications on SCC, which do not necessarily examine durability, often state that SCC results in better quality, more durable concrete than VC (Kasemchaisiri and Tangtermsiriku 2008; Tande and Mohite 2007; Zhao et al. 2012; Zhao et al. 2012). Investigation into the existing limited SCC vs. VC literature revealed how many of the studies which found SCCs to be more durable than VCs used SCM as filler materials in the SCC mixes i.e. did not use SCM in calculation of w/b ratio. Importantly, the figures presented above suggest that, for both OPC concretes and PFA(k) concretes, when SCC and VC w/b ratios are equal, and SCMs are not used as a filler in SCC, VC are more durable if sufficiently compacted. For GGBS the comparison suggests that the SCC performance may be equal to or superior to the VC for the same conditions.

Given these findings, and the findings related to the effect of exposure condition under Research Question 2, it would be very interesting to carry-out the same comparison in different exposure conditions. Also, as noted above, the VCs studied herein underwent ideal compaction, a situation unlikely to occur throughout full scale members on site. These issues will be discussed further in Chapter 8 in the context of recommendations for future work.

Three of the Phase 2 experimental aims presented in this Chapter related to Research Question 1 a). All three of these experimental aims were achieved with the following findings;

The investigation into the *k*-value efficiency factor in the context of a marine durability indicated that the use of PFA as a direct replacement (k=1) in SCC, resulted in a relative merit of 1.8 when compared to OPC SCC, while the use of the *k*-value efficiency factor of 0.4 as specified in EN206-1:2000, resulted in a relative merit of 2.6. In other words, the Phase 2 testing indicated that when considering marine durability, the use of PFA has an actual efficiency factor greater than 1, while the use of the efficiency factor prescribed in EN 206-1:2000 results in a marine durability performance 2.6 times that of plain OPC SCC. This work can be said to contribute to addressing Research Question 1 through the achievement of Phase 2 experimental aim no. 3. It is noted, however, in the context of the findings in this chapter relating to Research Question 2, this analysis was carried out in the wettest exposure condition considered in this thesis. In the context of the findings relating to the effect of exposure conditions in Section 5.6.3, a marine durability based efficiency factor of greater than 1 would be inappropriate for OPC + PFA concrete.

Investigation into the Phase 2 experimental aim no. 2 indicated that the nature of application of silane surface treatment has a significant effect on the variability of silane performance. Comparison of the Phase 1 and Phase 2 OPC-S results also identified the initial moisture condition of the concrete sub-strata upon silane application as a cause of silane performance variation. Logic dictates that these issues would play a significant role in silane performance on site. Examination of the existing silane literature revealed a great deal of performance variation. It is likely that a portion of this performance variation is due to the effects highlighted herein. This is an important consideration in the context of comparing the relative performance of different SCC options under core PhD Research Question 1.

Finally investigation into Phase 2 experimental aim No. 1 revealed that the effect of initial absorption is unlikely to have had a significant effect on the Phase 1 test results, confirming that the findings in Chapter 4 relating to Research Question 1 a) were of sound basis.

Research Question 3:

How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

The work presented in this chapter does not directly address Research Question 3. However, obtaining the chloride transport properties for a mix designed to mimic the original Ferrycarrig Bridge concrete in this chapter, facilitates investigation into calibration of salt fog chamber testing in a probabilistic framework, using real marine environment data. This assessment is carried out in Chapter 7.

6. PROBABILISTIC MODEL

6.1 INTRODUCTION

Chapters 4 and 5 presented two detailed experimental studies which investigated the chloride transport properties of a number of different concrete types and addressed the core PhD Research Questions 1 and 2. As illustrated in previous chapters, and in the literature review in Chapter 3, these results constitute a valuable contribution to the existing literature in their own right. However, in order to gain maximum benefit from the experimental testing the chloride transport properties obtained must be utilised in a predictive model. The use of predictive modelling allows additional factors, which perhaps could not be considered experimentally, to be incorporated into the analysis. For instance it was not practical to examine time dependant diffusion over the life of the current PhD, yet modelling of the experimental output can facilitate the incorporation of this effect into the analysis. These time dependent effects are important when it is considered that the data collected herein, from concrete approximately 1 year old, is used as an indicator of relative performance of concrete options for structures which often have a design service life of 100 years or greater.

Having considered the importance of predictive modelling, it must also be noted that, as established in Chapter 3, there is significant uncertainty associated with modelling chloride ion ingress. This is due to the complex nature of the chloride migration process (Climent et al. 2002), and the high levels of uncertainty associated with some of the parameters which dictate chloride induced corrosion, such as the time dependent diffusion coefficient (Nokken et al. 2006) and the critical chloride content (Angst et al. 2009). As discussed in Chapter 3 these relatively high levels of engineering uncertainty have lead researchers in recent years to represent chloride-ion ingress using probabilistic methods (Bastidas-Arteaga et al. 2009; Lu et al. 2011; So et al. 2009; Stewart and Suo 2009; Suo and Stewart 2009; Vu and Stewart 2000). These probabilistic methods allow uncertainty to be quantified and interpreted in the context of a modelling output, providing a more robust and informed study. However, the lack of probabilistic studies examining the marine durability of different concrete options was also highlighted in Chapter 3. This shortcoming in the existing literature is particularly important for SCC options, due to the lack of any long term data for this relatively new concrete technology. As discussed in the literature review, this situation led to the development of Research Question 3 as follows:

Research Question 3: How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

This chapter of the thesis presents the probabilistic model which was utilised, together with the experimental results presented in Chapter 4 and 5, to address Research Question 3 in this PhD. The results of the probabilistic modelling itself are not presented in this Chapter, but are discussed in detail in Chapter 7. The first section of this chapter presents the theoretical basis for the probabilistic model. This involves discussion of a number of theoretical models utilised in the literature and the selection of an appropriate model for use herein. The next section of the chapter presents the details of the selected model and highlights an issue with the proposed model. The model is revised using a practical and theoretically grounded adaption to resolve the issue. The use of appropriate model parameters which must be adopted from the literature is then discussed in detail. Subsequently, the final section of the chapter presents a probabilistic sensitivity analysis of the proposed model which quantifies the extent to which each parameter influences the probabilistic model output.

6.2 CHLORIDE TRANSPORT MODEL

Since Collepardi et al. (1970) first attempted to represent the progression of chlorides into concrete a vast number of models have been proposed by different researchers to represent the ingress of chlorides into concrete (Nilsson 2009). Often these models are validated by chloride ingress data showing good results, however, this calibration may sometimes be biased, as in the case of Wang et al. (2011), with the same data used to calibrate and validate a model developed. In reality many of the models developed are not utilised by other researchers, due to their complex nature and need for model input parameters which are not readily available.

A sample of models from the available literature will be discussed in Section 6.2.1, however, given the number of chloride ingress models in the literature, it is neither possible, nor desirable to present an exhaustive discussion. The aim of the work herein is not to develop a new predictive model or to incorporate a complex, little used model into a probabilistic framework. Instead the aim is to examine different concrete types in a probabilistic framework using a relatively simple, well-established and recognised predictive model, which facilitates fair comparison between the different concrete types.

6.2.1 Selection of Appropriate Chloride Transport Model from the Literature

Before probabilistic modelling can be carried out a chloride transport model must first be selected from the literature. The selected model will form the basis for the probabilistic analysis of the time

to initiation of corrosion using the Phase 1 and Phase 2 experimental results as input parameters. It is however, recognised that the transport of chlorides in concrete is a very complicated process (Shi et al. 2012), with a wide variety of contributing parameters varying considerably from structure to structure. Therefore, it is also recognised that all available models are in some way flawed, resulting in substantial limitations on the accuracy of long term predictions (Nilsson 2009). Nevertheless, given the scale of the chloride induced corrosion problem, an attempt to predict service life in chloride rich environments must be made.

Perhaps the simplest and undoubtedly the most popular model for predicting chloride ingress into concrete, and the resultant time to initiation of corrosion, is Fick's second law with Crank's error solution. The model was presented in Chapter 2 and in Chapter 4 in the discussion regarding curve fitting to chloride profiles. It is presented again below in its simplest form as Equation 6.1:

$$C(x,t) = C_i + (C_s - C_i) \times \left(1 - erf\left[\frac{x}{\sqrt{4Dt}}\right]\right) \qquad \textbf{Equation 6.1}$$

where C(x,t) is chloride content at time t and at depth, x, into the concrete. D is the diffusion coefficient, erf is the error function, C_s is the surface chloride content and C_i is the initial chloride content in the concrete. This model has a number of inherent assumptions that are not representative of real marine exposure such as; a) the concrete is fully saturated during the exposure period, b) the concentration of chlorides at the concrete surface is constant, c) the apparent diffusion coefficient is constant over time and d) the concrete is assumed homogenous (Zhang and Jin 2011). As stated by Bentolini et al. (2004) these assumptions are not valid in real life due to issues such as, the wetting and drying of concrete leading to transport by absorption, the time dependant nature of *D*, and chloride binding due to adsorption or chemical reactions. It is however, also stated by Bentolini et al. that experience has shown that, even with these errors, chloride profiles from structures subject to wetting and drying etc. can be modelled with good approximation using Equation 6.1 above. This, combined with the relatively simple nature of Fick's second law with Crank's error solution, has resulted in the majority of probabilistic models published in the literature utilising Equation 6.1, to model time to initiation of corrosion (Bastidas-Arteaga et al. 2009; Mullard and Stewart 2012; So et al. 2009; Val and Stewart 2003; Vu and Stewart 2000).

Numerous researchers have, however, developed models which attempt to represent the ingress of chlorides without the need for some of the assumptions associated with Fick's second law. As mentioned in Chapter 3 for instance, Zhang and Jin developed a mathematical model of chloride transport in concrete, under wetting and drying conditions, which accounted for absorption and diffusion (Zhang and Jin 2011). The model is based on representing the moisture movement in unsaturated concrete and coupling this with the transport of chloride ions in a porous system. In order to utilise this model, however, a number of parameters must be known or estimated; the connected porosity of the concrete being considered, the surface tension and contact angle of the liquid being absorbed and finally the chloride binding isotherm, obtained in the Zhang and Jin publication by curve fitting to test results. The authors illustrated reasonable agreement between the predicted model and the chloride profiles used to validate the model, although inspection of the profile plots indicates that the model did seem to overestimate chloride contents at deeper depths.

Bastidas-Arteaga also developed a model to represent chloride migration in unsaturated concrete which accounted for chloride transport by capillary sorption and convection as well as diffusion (Bastidas-Arteaga et al. 2011). In addition to accounting for multiple transport mechanisms, the model also allowed for other impacting factors such as chloride binding and temperature effects. Bastidas-Arteaga utilised this model for the probabilistic analysis of time to initiation of corrosion. The results of the probabilistic modelling example showed that the difference between including convection in the model and omitting convection from the model were small. It should be noted however, that the complexity of the Bastidas-Arteaga model means that a large number of parameters are required to predict the ingress of chlorides into concrete.

Val and Trapper (2008) developed a 2-dimensional model to represent the ingress of chlorides into RC structures. The model was developed to account for the penetration of chlorides from two directions via both diffusion and convection. The model also considered chloride binding and the influence of temperature and humidity on chloride ingress. It was however, noted by Mullard (2009) that this complex model has many variables, some of which are empirical. Thus a large amount of input data must be calculated, tested or estimated for different field conditions and concrete types.

Other researchers have developed adaptions to the traditional Fick's second law to account for the influence of various observed effects on chloride ingress such as structural cracking (Lu et al. 2011; Ye et al. 2012) and time dependant diffusion (Luping and Gulikers 2007; Visser et al. 2002). Perhaps most notable of these adaptions, in the context of the work presented herein, is the issue of time dependant diffusion. As touched upon in previous chapters, it has been found by a number of

researchers that the apparent diffusion coefficient reduces with time (Cheng et al. 2011; Nokken et al. 2006; Stanish and Thomas 2003). It is also recognised that the rate and extent of the diffusion coefficient reduction over time has been found to be greater for concrete containing SCMs, such as PFA and GGBS, than that experienced in OPC concrete (Pack et al. 2010; Thomas and Bamforth 1999; Thomas et al. 2008). The work of Mangat and Molloy (1994) and more recently, Luping and Gulikers (2007), accounts for the time dependant diffusion in Fick's second law through the incorporation of an age factor utilising a power law. The value for this age factor increases for concretes containing SCMs, allowing OPC concretes and concretes containing SCMs to be compared in a fair manner over a time period appropriate to a structure's service life. This will be discussed in more detail later in Section 6.2.2. It is noted at this point however, that there is a lack of information in the literature relating to the age factor values (Nokken et al. 2006). This is due to the high cost and time associated with gathering diffusion coefficient information, using natural chloride migration testing at two different concrete ages (Andrade et al. 2011).

Overall, after consideration of the available models and the various adaptions to Fick's law it was decided to utilise Fick's second law with Crank's error solution with the time dependant diffusion adaption incorporated. There were a number of reasons for choosing this model over some of the more complex models developed in the literature. Firstly, it is recognised that while models that account for many variables have the potential to produce a more accurate result, any inherent accuracy of these more complicated models is likely to be made redundant by the uncertainty associated with the approximation of unknown parameters (Mullard 2009).

The second reason for adopting the more simplistic model was that Fick's second law is the most widely used model throughout the literature, and in engineering practice in the assessment of existing structures (Li et al. 2003). It's wide use has resulted in Fick's second law having a proven track record when utilised on marine structures subject to wetting and drying conditions (Bertolini et al. 2004). In addition, the data available for the Ferrycarrig Bridge site from the work of O'Connor and Kenshel (2013) was in the form of Fick's second law. As previously discussed, it is intended to use this site data in the probabilistic model to investigate the effect of calibration of the accelerated experimental data with data from the real marine environment. The use of Fickian based model will makes this calibration more straight forward. Finally the regression analysis carried out using Fick's second law with Crank's error solution and the experimental results herein resulted in excellent fit with r^2 values ranging from 0.942 to 0.998.

Given the requirement to utilise a simple model, it was not desirable to incorporate a number of adaptions to Fick's second law with Cranks error solution. It is noted however, that the

probabilistic modelling in this thesis utilises the experimental results detailed in Chapter 4 and 5 to compare various concrete types. The relatively short duration of the laboratory based experimental programs carried out herein means that long term differences between OPC concretes and concretes containing SCM were not fully accounted for. It has been established in the literature that concretes containing PFA and GGBS perform better than OPC concretes over time, due to the time dependant diffusion coefficient (Cheng et al. 2011; Pack et al. 2010; Stanish and Thomas 2003; Thomas et al. 2008). Consequently, it is clear that in order to carry-out a fair comparison of service life predictions for the various concrete types examined herein, the model selected must account for significant time dependant differences in OPC concrete and concretes containing SCMs. Thus it was deemed necessary to include time dependant diffusion in the chloride ingress model. This model is presented in the next section.

6.2.2 Presentation of Model Details

The time dependant diffusion coefficient is expressed in the literature by a power law as shown in Equation 6.2 (Audenaert et al. 2010; Thomas and Bamforth 1999):

$$D(t) = D_r\left(\frac{t_r}{t}\right)^m \qquad \textbf{Equation 6.2}$$

where D(t) is the diffusion coefficient at time t, D_r is the diffusion coefficient at reference time t_r and m is an age factor. There is however, considerable confusion in the published literature as to how this time dependant diffusion coefficient factor should be incorporated into Fick's second law (ChlorTest 2005; Luping and Gulikers 2007). A number of researchers have substituted Equation 6.2 above directly into the Fick's second law (DuraCrete 1998; Ferreira 2008). This is incorrect for two reasons. Firstly, as pointed out by Luping and Gulikers (2007), D(t) is a function of time that cannot be directly substituted into the error function solution in Fick's second law without time integration. Secondly, substituting this formula directly into Fick's second law neglects the historic change in the diffusion coefficient over time. This can be visualised in the context of Figure 6.1 below which shows the D(t) values based on Equation 6.2. Considering the 10 year value from the plot, it is clear that if Equation 6.2 is substituted directly into Equation 6.1 the chloride content at 10 years will be based on a constant diffusion coefficient value of 12 mm^2/year from 0 to 10 years. This obviously underestimates the quantity of chloride ingressing into the concrete over this time.

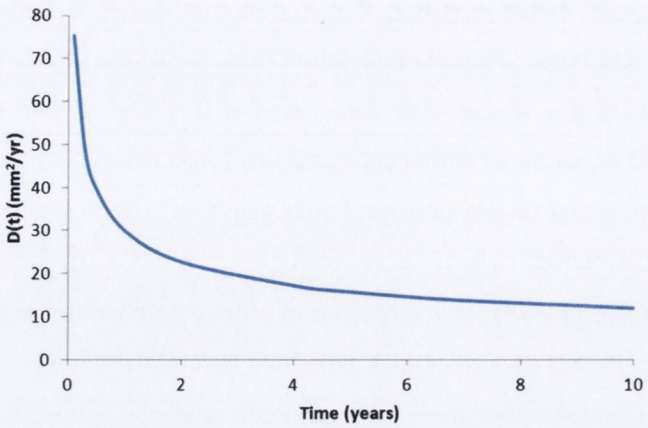


Figure 6.1: Change in diffusion coefficient over time

The confusion in the literature regarding this area is increased due to the fact that two forms of diffusion coefficient exist. The instantaneous diffusion coefficient, (D or D_r at a given time t_r) which reflects the rate of chloride progression through concrete at a given point in time, and the apparent diffusion coefficient, D_{app}, which is obtained by curve fitting to chloride profiles, as established in the previous chapters. This apparent diffusion coefficient reflects the average rate of chloride diffusion over the exposure period up to when the D_{app} value is obtained. As pointed out by Visser et al. (2002), when this D_{app} value is being used to obtain an instantaneous D_r value to be used in the time dependant diffusion calculation, it is important to account for the historic change in D_r over the exposure period. This can be done by integrating the curve shown above in Figure 6.1 and dividing by the exposure period to give a mean diffusion coefficient over the exposure period. This relation is expressed below in Equation 6.3.

$$D_m(t) = \frac{1}{t}\int_0^t D(t)dt = \frac{D_R}{1-m}\left(\frac{t_R}{t}\right)^m \qquad \textbf{Equation 6.3}$$

where $D_m(t)$ is the mean diffusion coefficient for the exposure duration, t and D_r is the instantaneous diffusion coefficient at a reference time t_r. This mean diffusion coefficient can be taken as constant over the exposure period being considered and substituted into Equation 6.1 to give an adaption of Fick's law which incorporates the reduction of the diffusion coefficient over time. This formula was also presented by Pack et al. (2010), however, in this instance the authors incorrectly used the formula by utilising an apparent diffusion coefficient for D_r, when in fact D_r integrated in this manner should reflect an instantaneous diffusion coefficient, as the apparent diffusion coefficient already reflects historic change by definition. Although it was not alluded to by Pack et al. or Visser, the $D_m(t)$ term above actually reflects the D_{app} term at time t. Thus the

apparent diffusion coefficient data at two different exposure periods can be used to obtain a D_r value and an m value as detailed by Stanish and Thomas (2003).

There is however, an assumption in Equation 6.3 which could lead to large errors in certain circumstances. As pointed out by the Chlortest project (2005) and Luping and Gulikers (2007), the integration of D(t) above assumes that exposure occurs from t = 0, i.e. the concrete is exposed to chloride from day 1. This may be an acceptable assumption for site data, collected from structures which are a number of years old, as relatively speaking, the age of the concrete is very close to the exposure duration. If however, the D_{app} data is obtained from relatively short duration testing with specimens cured for a number of weeks before exposure, considerable errors can be encountered by using Equation 6.3, as illustrated by Tang and Luping (2007). Consequently, when obtaining D_r values from the D_{app} values obtained in this thesis the adjusted formula developed by Tang and Luping will be utilised. This formula is presented below:

$$D_m(t) = D_{app}(t) = \frac{1}{t}\int_{t_{ex}}^{t} D(t)dt = \frac{D_r}{1-m}\left(\frac{t_r}{t}\right)^m \cdot \left[\left(1+\frac{t_{ex}}{t}\right)^{1-m} - \left(\frac{t_{ex}}{t}\right)^{1-m}\right] \qquad \textbf{Equation 6.4}$$

where t is the exposure duration and t_{ex} is the age of the concrete at the time when samples were initially exposed to chlorides. This formula allowed the D_r values to be obtained for the service life modelling herein, without the errors associated with neglecting the curing duration implemented prior to testing. As touched upon above, the 3 - 4 year timeframe of a PhD research project are not sufficient to carry out a robust study into the time dependant diffusion coefficient. Consequently, the estimation of the m from experimental testing was outside the scope of this research project. It is also hoped that the information from Ferrycarrig Bridge collected over a number of years will provide information on the time dependant diffusion in years to come. For the calculations herein appropriate m values were adopted from the available literature as will be discussed in detail in the next section.

Once an appropriate D_r value was obtained for each of the concrete mixes using Equation 6.4 above, this instantaneous diffusion coefficient value was used, together with m-values and t_r value, for service life prediction. For these calculations, it is deemed acceptable to utilise the D_r value obtained with the formula proposed by Visser et al. (2002) as there will be little difference between the age of concrete and the duration of exposure for the times applicable to service life. Thus service life predictions were based on the substitution of Equation 6.3 (using the D_r value obtained from Equation 6.4) into Equation 6.1. By rearranging this equation and inserting the critical

chloride content, C_{cr}, and cover depth, C_d, the time to initiation of corrosion, T_i can be calculated as follows:

$$T_i = \left[\frac{C_d^{\,2}}{4D_R t_R^m} \times (1-m) \left(erf^{-1} \left(\frac{C_{cr} - C_s}{C_i - C_s} \right) \right)^{-2} \right]^{\frac{1}{1-m}} \qquad \textbf{Equation 6.5}$$

Pack et al. (2010) used a similar formula to that used in Equation 6.5, although as previously stated the authors failed to implement the theory correctly by using an apparent diffusion coefficient as D_r. McNally and Sheils (2012) also utilised the equation presented above when using probabilistic methods and instantaneous chloride ingress test (RCMT) results, to predict chloride induced time to initiation of corrosion. It should be noted however, that McNally and Sheils did not model the *m* value probabilistically. In addition the *m* values used by McNally and Sheils were adopted from the work of Pack et al., who found the OPC *m* value to be 0.06 and the GGBS *m* value to be 0.23. These values, particularly in the case of GGBS, are significantly lower than those found by other researchers, with GGBS *m* values around 0.6 most common in the literature, as will be discussed later in the context of Table 6.2. The low *m* values found by Pack et al. are likely due to errors caused by utilising diffusion coefficients from different bridges to calculate an *m* value. The uncertainty surrounding this practice is reflected in the r^2 values associated with the regression fit used to obtain values for *m*, which ranged between 0.4 and 0.5.

If however, the model in Equation 6.5 is used in a probabilistic analysis with more appropriate, larger *m* values, problems arise. To illustrate this fact, the output from a probabilistic model is presented below in Figure 6.2 and Figure 6.3. Figure 6.2 below shows the result of the probabilistic model using the Pack et al. (2010) GGBS *m* value of 0.23. Figure 6.3 shows the probabilistic model result when a more appropriate *m* value of 0.60 is used. The statistical parameters for the probabilistic modelling example are presented in Table 6.1. The GGBS D_r value is adopted from the Phase 2 experimental results. These are the same parameters as will be used in the sensitivity analysis and will be discussed in detail in Section 6.3. The values are merely presented here to show the origin of Figure 6.2 and Figure 6.3.

Table 6.1 Statistical parameters for probabilistic analysis

Property	Units	Mean	COV	Dist.	Source
ER_{mod}	_	1.0	0.2	LN	(Rafiq et al. 2004)
C_{cr}	*%	0.07	0.25	LN	(Kenshel 2009)
Cover, x	mm	Spec. + 1.6	σ = 11.1	N	(Mirza and MacGregor 1979; Stewart and Suo 2009)
Pack GGBS *m* value Appropriate *m* value	_	0.26 0.6	0.20	N (truncated at 0.8)	Mean: (Thomas et al. 2008) COV & Dist: (Bentz 2003; Ferreira 2008; Van den Heede et al. 2012)
C_s	[1]%	1.00	0.06	N	Mean and COV: Phase 2 Exp. Results Distribution: (Stewart and Suo 2009; Vu and Stewart 2005)
D_R	mm^2/yr	25.07	0.11	N	Mean and COV Phase 2 Exp. Results Distribution: (Choe et al. 2009; Stewart and Suo 2009)

*Percentage by weight of concrete, Dist. = Distribution LN is log normal distribution, N is normal distribution, Spec. = Specified, Exp. = Experimental

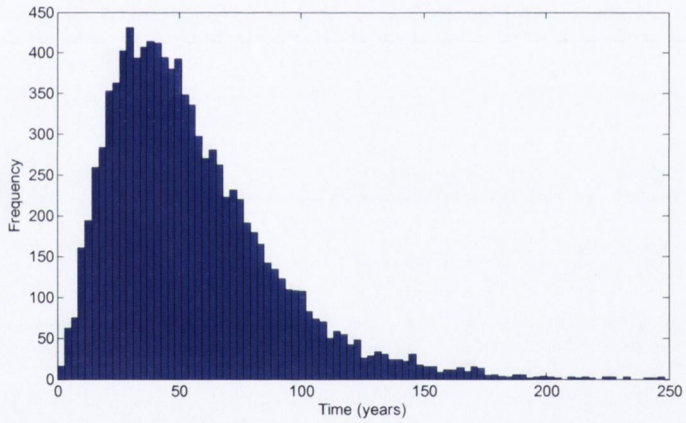


Figure 6.2: T_i values obtained using *m* =0.26 for GGBS

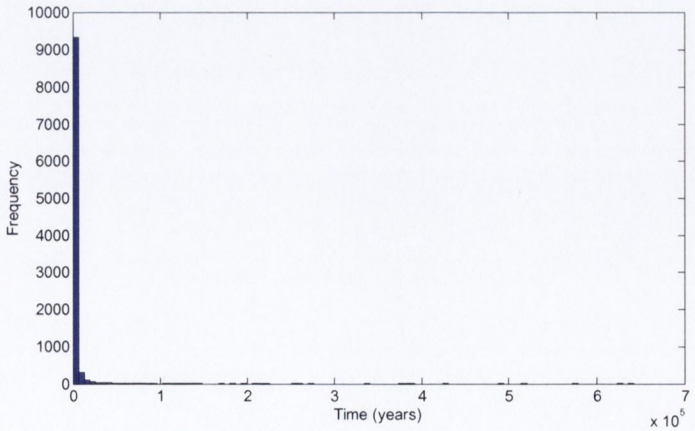


Figure 6.3: T_i values obtained using *m* =0.60 for GGBS

As can be seen from the figures, the use of the lower mean *m* value results in T_i values of up to 250 years. The use of the more appropriate GGBS *m* value with Equation 6.5, however, leads to T_i values as high as 700,000 years with 4274 of the 10,000 T_i values greater than 500 years. When it is considered that the D_{app} and C_s values utilised in the analysis were obtained from experimental testing, such high values of T_i are clearly erroneous. These unreasonably high T_i values are caused due to the fact that in the model proposed by Visser (2002), and utilised by Pack et al. (2010) and McNally and Sheils (2012) for service life prediction, the diffusion coefficient continues to decrease throughout the structure's service life. This leads to very small diffusion coefficient values at high t values. In a probabilistic framework this issue is exasperated due to the utilisation of parameter values from the entire parameter statistical distribution, and the nature of the mathematics in Equation 6.5. It is noted that the manipulation of this semi-empirical relationship to obtain T_i, results in the term inside the square brackets being raised is to the power of (1/1-m). This situation means that for values of *m* approaching 0.8 (not unusual for probabilistically modelled concretes containing GGBS and PFA) the term inside the brackets is raised to the power of 5. Thus when time dependant diffusion models are utilised for probabilistic Monte Carlo simulation service life prediction, with appropriate *m* values for OPC + GGBS and OPC + PFA concretes, a significant quantity of unreasonably high T_i values will result due to the (1/1-m) term.

In order to address this issue relating to probabilistic modelling using SCM concrete *m* values, it is proposed that after a given period of time, the instantaneous diffusion coefficient should cease to decrease after a given time t_c. This approach has been adopted in a deterministic approach by the commercial service life prediction software model Life 365 (Life 365 Consortium II et al. 2012) and in a probabilistic service life approach by Song et al. (2009). It is noted however that Song et al. did not model the time dependant diffusion coefficient probabilistically.

The application of the t_c parameter addresses the mathematical issue in Equation 6.5 and has a solid logical basis in the context of the behaviour of concrete as follows. It is recognised in the literature that the reduction of the diffusion coefficient with time is caused by a combination of Friedal's salt forming due to chloride binding and the refinement of pore structure due to continued hydration (Luping and Gulikers 2007; Nokken et al. 2006). It is also recognised however, that the aluminates, which form Friedal's salts are not as effective at binding chlorides at later ages of concrete. In addition, over the course of a structure's service life, Friedal's salts may be dissolved due to increases in the concrete pH with time (Neville 2011). Pore refinement as a result of hydration is also likely to cease after a number of decades. Consequently, it would seem unlikely that the instantaneous diffusion coefficient, which reflects the diffusion properties of the concrete at a given point in time, would decrease indefinitely throughout a structure's service life. In order to reflect

this the Life 365 commercial concrete deterioration model and Song et al. (2009) hold the diffusion coefficient constant at the 25-30 years until the end of the structure's service life, stating that hydration is complete at this point. Thomas et al. (2008), in their model, also prevent the diffusion from reducing indefinitely by using a slightly different approach. The authors add an ultimate diffusion coefficient to the time dependant diffusion coefficient term to prevent the diffusion coefficient falling below a pre-defined level.

In order to rectify issues with probabilistic modelling using Equation 6.5 with statistically sampled high *m* values, while utilising a model which incorporates both the historic change in the diffusion coefficient, and a time dependant diffusion coefficient which remains constant after a number of decades, an adaption to Equation 6.5 is proposed below based on a new $D_m(t)$ term. This model holds the instantaneous diffusion coefficient constant after a given time t_c as shown below in Figure 6.4

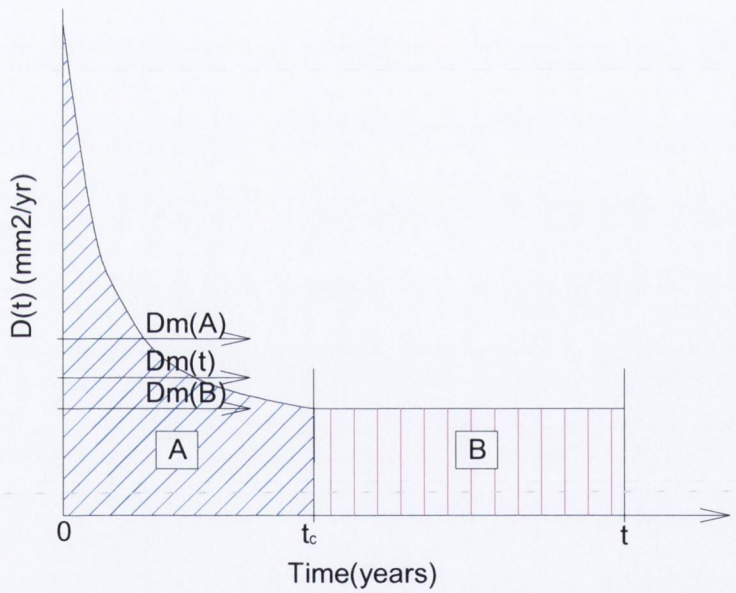


Figure 6.4 Calculating $D_m(t)$ based on time dependant instantaneous diffusion coefficient being constant after time t_c

Figure 6.4 above shows the instantaneous diffusion coefficient reducing up to time t_c, shown by area A shaded in blue. After t_c, the diffusion coefficient remains constant as illustrated by area B in Figure 6.4. The new $D_m(t)$ term to be substituted into Equation 6.1 must account for the historic change in D(t) in the period up to t_c and the subsequent constant D(t) phase from t_c to t. When calculating the time to initiation of corrosion the exposure period, t will equal T_i. The $D_m(t)$ term was obtained by averaging the D(t) values over the two phases using the following equation.

$$D_m(t) = \frac{\left[\frac{D_r}{1-m}\left(\frac{t_r}{t_c}\right)^m .t_c\right] + \left[D_r\left(\frac{t_r}{t_c}\right)^m .(t-t_c)\right]}{t} \qquad \textbf{Equation 6.6}$$

where t_c is the time at which the diffusion coefficient becomes constant and t is the time being considered. The term in the first square brackets represents area A, the terms in the second square brackets represents area B. An underlying assumption is that the age of the concrete is close to the exposure duration. As previously stated, this is an acceptable assumption for service life predictions. It should be noted that the $D_m(t)$ term above will be equal to the D_{app} value obtained at the exposure duration *t*, assuming the t_c value utilised is correct. Thus it is clear from Figure 6.4 that the D_{app} value will continue to decrease with time as the relative contribution of the area labelled A in Figure 6.4 decreases with the increase of time beyond t_c. This is a logical approach as, although there is theory to suggest why the instantaneous diffusion coefficient should remain constant after a given time, there is no such justification for setting the D_{app} value constant at a given time. This is because the D_{app} reflects the historic change in chloride diffusion over time and not diffusion at any one point.

Substituting Equation 6.6 into Equation 6.1 and again inserting C_{cr} and C_d the T_i value can be calculated as follows:

$$T_i = \left(\frac{C_d^{\,2}}{4D_{app-R}\left(\frac{t_r}{t_c}\right)^m} \times \left(erf^{-1}\left(\frac{C_{cr}-C_s}{C_i-C_s}\right)\right)^{-2}\right) - t_c.\left(\frac{m}{1-m}\right) \qquad \textbf{Equation 6.7}$$

It should be noted that the term in the brackets in Equation 6.7 is not raised to the power of (1/1-m) ruling out the mathematical issue which caused problems in the probabilistic analysis carried out with Equation 6.5. For the probabilistic modelling herein the following model will be utilised:

$$T_i = \left[\frac{C_d^{\,2}}{4D_{app-R}t_R^m} \times (1-m) \left(erf^{-1}\left(\frac{C_{cr}-C_s}{C_i-C_s} \right) \right)^{-2} \right]^{\frac{1}{1-m}} \quad \text{for } T_i \le t_c$$

$$T_i = \left(\frac{C_d^{\,2}}{4D_{app-R}\left(\frac{t_r}{t_c} \right)^m} \times \left(erf^{-1}\left(\frac{C_{cr}-C_s}{C_i-C_s} \right) \right)^{-2} \right) - t_c \cdot \left(\frac{m}{1-m} \right) \quad \text{for } T_i \ge t_c$$

Equation 6.8

In order to illustrate the effect of Equation 6.8, the figures below present the T_i values calculated across a range of *m* values. The other parameters in the equation (D_r, C_d & C_{cr}, C_s) are from one of the simulations which resulted in unrealistically high T_i values (T_i > 100,000 years) in Figure 6.3. Figure 6.5, Figure 6.6 and Figure 6.7 show the *m* value ranges from 0 to 0.60, 0 to 0.70 and 0 to 0.80 respectively.

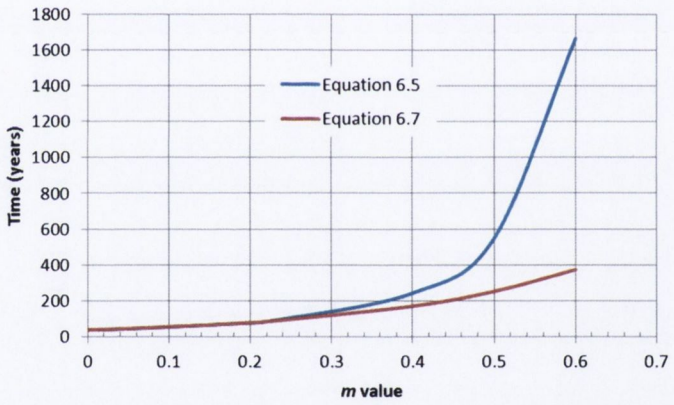


Figure 6.5: Equation 6.5 and 6.7 plotted for *m* values from 0 to 0.60

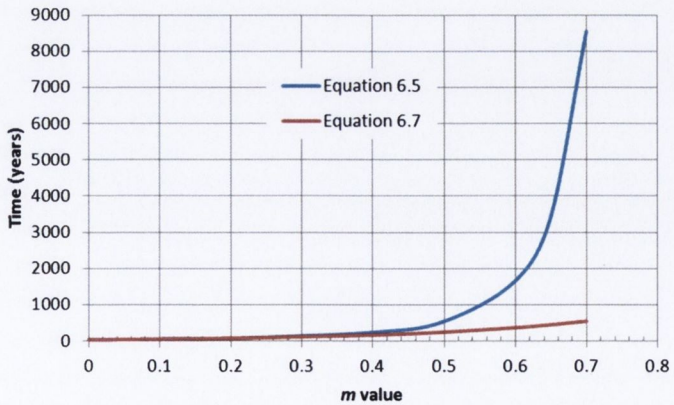


Figure 6.6: Equation 6.5 and 6.7 plotted for *m* values from 0 to 0.70

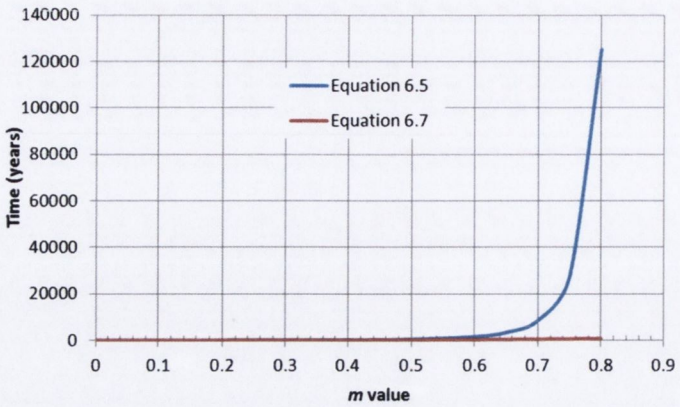


Figure 6.7: Equation 6.5 and 6.7 plotted for *m* values from 0 to 0.80

As can be seen from Figure 6.5 to Figure 6.7, the model adaption presented above has resolved the problem with the original model presented in Equation 6.5. To illustrate the improvement further, the probabilistic output obtained using Equation 6.8 and identical statistical parameters as those used for Figure 6.3 is presented below. As can be seen from the figure, the predicted T_i values are now in a far more realistic range.

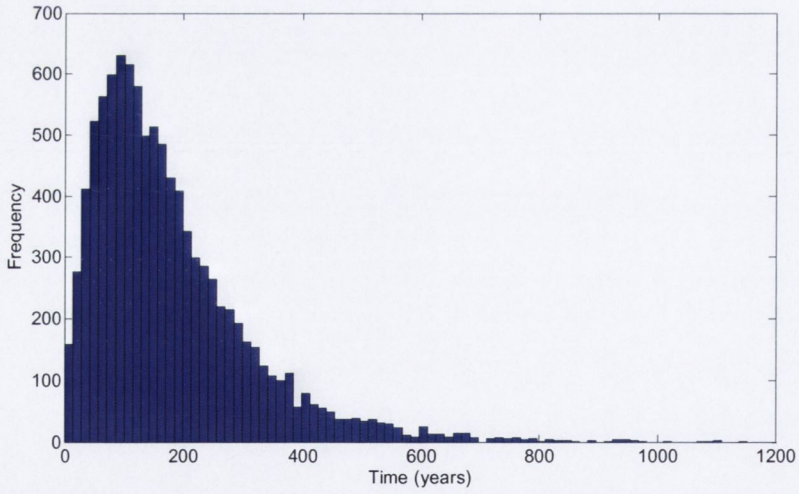


Figure 6.8 Distribution of predicted T_i values based application of Equation 6.8

6.2.3 Selection of Parameters from the Literature

Examination of Equation 6.8 reveals that four of the parameters in the formulae above were not determined experimentally in Chapters 4 and 5, and thus must be adopted from the literature. These four parameters are the time dependant diffusion coefficient, *m*, the critical chloride content, C_{cr}, the cover depth, C_d, and the time after which the diffusion coefficient remains constant, t_c. The

selection of these parameters will be discussed in this section. It should also be noted that the sensitivity of model output to all parameters in the model will be assessed in Section 6.3.

The selection of C_d is relatively straight forward. The specified C_d will be 50mm as per the C_d value specified for the Ferrycarrig Bridge repair in 2007. This is a common cover depth for XS 3 elements which are located in the splash zone and tidal zone of marine structures. In accordance with a work of Stewart and Suo (2009) a C_d standard deviation of 11.1mm and a C_d mean bias of + 1.6mm will be used herein. These figures were used by Stewart and Suo based on data collected from RC beams by Mirza and MacGregor (1979). In accordance with the findings of Mirza and Macgregor, the mean bias and standard deviation remains constant regardless of the magnitude of the specified cover.

Luping (2007) states that the time dependent diffusion coefficient was only modelled for the first time in 1988 by Takewaka and Mastumoto (1988). It is therefore not surprising that there is little information available in the literature as to when the diffusion coefficient ceases to decrease with time. Markeset and Skjolsvold (2010) state that the t_c figure is in the region of 10 to 15 years. This figure would however seem a little low in the context of the findings of Thomas et al. (Thomas et al. 2008) who reported time dependant diffusion up to 25 years on the basis of a 25 year study of OPC concrete and GGBS concrete. The Life 365 commercial service life prediction software model, of which Thomas is one of the committee members, utilises a t_c value of 25 years. A similar value of 30 years was also used by Song et al. (2009). However none of the authors mentioned above have modelled the t_c parameter probabilistically. Thus the statistical distribution parameters must be carefully considered. Having explored a number of distribution and COV values a lognormal distribution with a COV of 0.5 was deemed most appropriate. A histogram of the chosen t_c distribution is shown below as Figure 6.9. The large COV value was chosen to represent the uncertainty surrounding the t_c value and lack of knowledge concerning the effect of different concrete types and different exposure conditions on t_c. The lognormal distribution results in approximately 96% of values being greater than 10 years and approximately 96% of values being smaller than 50 years.

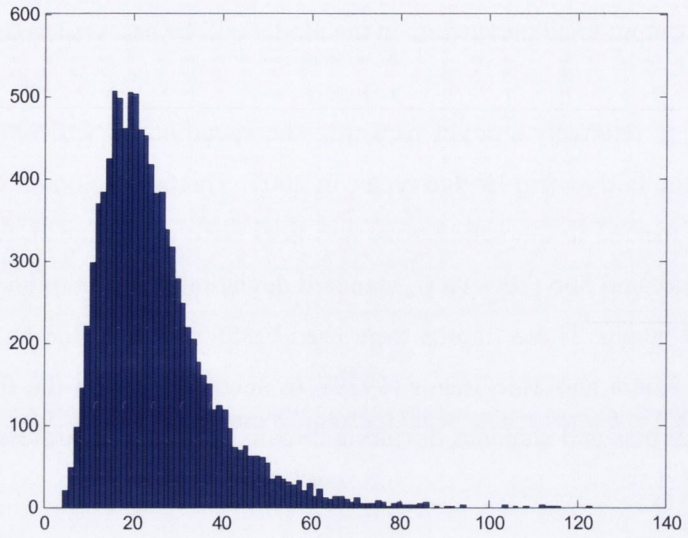


Figure 6.9: Histogram of t_c for chosen statistical parameters

There is also significant uncertainty surrounding the two remaining parameters, *m* and C_{cr}. Firstly, considering the *m* value, it is recognised in the existing literature that there is limited information on appropriate values of *m* for different concretes (Nokken et al. 2006). Table 6.2 below presents values of the time dependant diffusion coefficient available in the literature for OPC and concretes containing PFA and GGBS.

As can be seen from the table, the *m* values reported in the literature vary from study to study. One likely cause for this variation is that the *m* values in Table 6.2 are obtained from a number of different types of testing ranging from the measure of apparent diffusion coefficient values obtained from real marine structures up to 48 years old (Pack et al. 2010), to instantaneous diffusion coefficient obtained from RCMT carried out over a number of days (Audenaert et al. 2010). The effect of the type of testing is highlighted by the Andrade et al. (2011) study where two different sets of *m* values were obtained for the same concrete types using two different forms of test. Given the variability in reported *m* values and the small number of studies examining OPC, OPC + PFA and OPC + GGBS concretes, probabilistic methods are well suited to represent time dependant diffusion. The probabilistic methods will allow uncertainty to be quantified and incorporated in the analysis. In order to do this however, mean *m* values must be selected for OPC, OPC + PFA and OPC + GGBS based on the values in Table 6.2.

Table 6.2 OPC, PFA and GGBS *m* values from the available literature

Authors	*m* Value OPC	*m* Value PFA	*m* Value GGBS	Basis
Stanish et al. (2003)	0.32	0.66	–	Bulk diffusion tests carried out for up to 4 years
Pack et al. (2010)	0.06	–	0.23	Data from marine bridges up to 48 years old
Audenaert et al (2010)	0.28 (SCC)	–	0.43 (SCC)	RCMT data carried out over 5 years
Andrade et al. (2011)	-0.03	1.0	–	Samples cured for 1 month or 12 months and exposed for 3, 12 and 24 months (Ponding Test)
Andrade et al. (2011)	-0.24	0.07	–	Samples cured for 1-12 months and tested using an accelerated chloride migration test
Thomas et al. (2008) & Life 365 model ((Life 365 Consortium II et al. 2012)	0.26	0.5	0.60	Based on formula produced from model validated using 25 years marine exposure site data. m=0.26+0.4(PFA/50+GGBS/70) max m = 0.6 (Life 365, D_{app} constant after 25 years)
Thomas and Bamforth (1999)	0.1	0.7	1.2	Based on model validated from 8 years marine exposure site data
Costa and Appleton (1999)	0.36-0.60	–	–	Based on 3-5 years of marine site exposure
Goltermann (2003)	0.75 for all concrete types			Based on 109 chloride profiles from spray or atmospheric exposure with w/c < 0.55
Goltermann (2003)	0.50 for all concrete types			Based on 476 chloride profiles from the submerged or splash zone with w/c < 0.55
Cheng et al. (2011)	0.13-0.2	0.39-0.63	–	Based on 30 to 180 days ponding experiment of OPC samples and OPC + 20% PFA samples
(Markeset and Skjølsvold 2010)	0.19	0.40	0.52	Based on 9 year Norwegian field study

SCC = Self Compacting Concrete, RCMT= Rapid Chloride Migration Test

The average m values across the studies presented in the table above for OPC, OPC + PFA and OPC + GGBS are 0.24, 0.57 and 0.61 respectively. The formula proposed by Thomas et al. (2008), and presented below as Equation 6.9, is in good agreement with these average m values for the most popular PFA and GGBS addition percentages. For instance, using the Thomas et al. formula to predict m values for the OPC concrete, the 30% PFA concrete and the 60% GGBS concrete, as used herein results in m values of 0.26, 0.50 and 0.60, respectively. Given the proximity of these values to the average of the m values presented in Table 6.2 and the fact that Equation 6.9 has been validated for 25 years of marine exposure data, this formula was used in this thesis to calculate m. It is also noted that the Thomas et al. formula has been used by a number of researchers to predict

values for m and is used in the commercial service life prediction package, 365 Life (Bentz 2003; Life 365 Consortium II et al. 2012).

$$m = 0.26 + 0.4\left(\frac{\%PFA}{50} + \frac{\%GGBS}{70}\right) \quad m \leq 0.60 \qquad \textbf{Equation 6.9}$$

As discussed in Chapter 3, probabilistic studies in the literature primarily focus on OPC concretes. Consequently, there are a limited number of m value statistical parameters published in the literature. Ferreira (2008), van de Heede (2012), and Bentz (2003) did, however, utilise m values in probabilistic studies with COVs of 0.07, 0.11-0.20 and 0.25 respectively. Given the level of variability associated with the values presented in Table 6.2, one of the larger values of COV seems appropriate. Thus an m value COV of 0.20 will be used for the probabilistic model. The influence of this m value COV will be assessed in the sensitivity model in Section 6.3. It is noted that the COV value selected is not intended to account for the variation of m from study to study due to method of analysis. In accordance with the work of Ferreira and Bentz, the m value will be modelled as a normally distributed parameter.

When considering an appropriate mean C_{cr} value for the probabilistic analysis one must consider the issue of chloride binding. It is generally accepted that chloride ions bound in the cement paste via chlorides adsorbed in the C-S-H bond, or bound to C_3A to form Friedel's salts, do not cause corrosion. Thus only the chlorides in the pore solution of concrete are thought to cause corrosion (Bertolini et al. 2004). Concretes containing PFA and GGBS have greater C_3A contents and therefore, have greater potential for chloride binding. Bertolini et al. also states however, that bound chlorides may play a role in reinforcement corrosion when citing the work of Glass and Buenfeld (1997), whom found that a large part of the bound quantity of chlorides are released as soon as the pH of concrete drops below 12, which may occur locally in voids at the steel/concrete interface. A similar finding was reported by Reddy et al. (2002), whom found that most bound chlorides will be released by a relatively small reduction in pH (pH approximately = 11). Reddy states that the release of chlorides at this pH value compared to that required to sustain passive film breakdown, results in a similar corrosion risk posed by bound chlorides and free chlorides. Neville (2011) highlights the fact that sulphate attack and carbonation of concrete will also release bound chlorides. Finally, in terms of the time effect on bound chlorides, it was pointed out by Richardson (2002), and by Neville (1995), that aluminates are very effective at binding chlorides at early ages, however, they are not as effective at binding chlorides at later ages. Thus by the time chlorides have ingressed within the concrete cover at later ages, many of the aluminates will have reacted

with other ions. Considering these points it can be said that the role of bound chlorides in reinforcement corrosion in aging structures is uncertain.

When considering the effect of the addition of GGBS on chloride binding, it was stated by Shi et al. (2012) that, based on the literature, GGBS addition improves both the chemical and physical binding capacity of the concrete, yet it also decreases the pH of the pore solution. Thus the effect of the addition of GGBS in concrete on the chloride threshold is uncertain. Conflicting findings have been published in the literature with Dhir et al. (1996) reporting reduced C_{cr} values for GGBS concretes and Schiessl and Breit (1996) and Oh et al. (2003) reporting higher or similar C_{cr} values in concretes containing GGBS.

The effect of PFA on the chloride threshold is also somewhat uncertain. In a review of the available chloride binding literature Yuan (2009) states that the addition of PFA increases the chloride binding capacity of concrete. This was confirmed experimentally by Dhir et al. (1997) and Arya and Xu (1995). Nagataki et al. (1993), however, found that the replacement of OPC with 30% PFA reduced the binding capacity of cementitious material exposed to external chlorides. A four year study by Thomas (1996) examining the performance of concretes containing various levels of PFA from 0% to 50% in the real marine environment, found that the chloride threshold decreased with increasing PFA content in concrete.

Given the uncertainty surrounding the effect of time on chloride binding and the uncertainty of the effect of SCMs on the C_{cr} value, it was deemed appropriate to utilise the same chloride threshold content for all concretes considered herein. This approach has been taken by other researchers using probabilistic methods to examine concretes containing GGBS and PFA (McNally and Sheils 2012; Van den Heede et al. 2012). The C_{cr} value will be expressed in terms of total chloride content in accordance with the recommendations of the DuraCrete project, due to uncertainties surrounding chloride binding, as mentioned in Chapter 4 (DuraCrete 1998).

Selecting an appropriate mean C_{cr} value in terms of total chloride content is also difficult. Angst et al. carried out an extensive literature review in 2009 to ascertain an appropriate C_{cr} value range. The authors concluded that a reliable range of chloride threshold values could not be found based on the literature due to variations in experimental set-ups and the use of unsuitable testing procedures. Kenshel (2009) however, utilised a mean C_{cr} value of 0.07% by weight of concrete which was lognormaly distributed with a COV of 0.25. These figures were based on fitting the lognormal distribution to data collected from over 20 publications, both deterministic and probabilistic. These C_{cr} statistical parameters were used in the probabilistic model herein.

6.3 PROBABILISTIC SENSITIVITY ANALYSIS

Having presented the theoretical model and the parameters to be utilised in the model, a sensitivity analysis of the model will now be conducted. This analysis will assess the effect of each parameter on the output of the model presented in Equation 6.8. This will be done by increasing the mean and COV value for each parameter in turn by 10% and presenting the resultant change in the predicted time to initiation of corrosion; T_i in line with standard procedures (O'Connor and Enevoldsen 2009). The predicted time to initiation of corrosion for the analysis is defined as the 50% probability of corrosion initiation. The Phase 2 cube specimen results will be used to generate mean values for D_{app} and C_s for illustrative purposes. The COV values obtained from the experimental testing will also be used in the sensitivity analysis. It is recognised that the results from a real marine structure will be subject to greater variability than the laboratory results, for a number of reasons such as; spatial variability leading to differences in exposure levels, defects in onsite concrete due to workmanship, effects of wind and other variable environmental effects onsite etc. The COV values obtained from the experimental testing are a reflection of the variability of the concrete material and the experimental testing under controlled conditions albeit for a small sample size. This lower variability will be used in the probabilistic modelling herein to facilitate a more sensitive comparison of the concrete durability options.

The statistical parameters for the probabilistic sensitivity analysis, before the 10% increases, are listed below in Table 6.3. As can be seen from the table, an ER_{mod} factor has been included in the probabilistic analysis. This factor incorporates model error into the probabilistic analysis. It was adopted from the work of Rafiq et al. (2004) whom multiplied the factor by the T_i prediction of Fick's law with Crank's error solution. It has thus been used in an identical manner herein to reflect the variability, due to the error associated with the predictive model.

Table 6.3 Statistical parameters for probabilistic analysis

Property	Units	Mean	COV	Dist.	Source
ER_{mod}	_	1.0	0.20	LN	(Rafiq et al. 2004)
C_{cr}	[*]%	0.07	0.25	LN	(Kenshel 2009)
Cover, x	mm	51.6	σ = 11.1	N	(Mirza and MacGregor 1979; Stewart and Suo 2009)
m value OPC	_	0.26	0.20	N (truncated at 0.8)	Mean: (Thomas et al. 2008) COV & Dist:(Bentz 2003; Ferreira 2008; Van den Heede et al. 2012)
OPC C_s	[1]%	1.00	0.065	N	Mean and COV: Phase 2 Exp. Results Distribution: (Stewart and Suo 2009; Vu and Stewart 2005)
OPC D_r	mm^2/yr	74.73	0.09	N	Mean and COV Phase 2 Exp. Results Distribution: (Choe et al. 2009; Stewart and Suo 2009)
t_c	years	25	0.50	LN	(Life 365 Consortium II et al. 2012; Thomas et al. 2008)

[*]Percentage by weight of concrete, Dist. = Distribution LN is log normal distribution, N is normal distribution, Spec. = Specified, Exp. = Experimental

The D_r value of 74.73 mm^2/yr was obtained by letting the Phase 2 OPC cube specimen D_{app} value equal $D_m(t)$ in Equation 6.4 in accordance with the procedure outlined in Section 6.2.2. An *m* value of 0.26 was used in the calculation for this OPC diffusion coefficient in accordance with Thomas et al. (2008). The reference time t_r was set equal to the exposure duration of 0.692 years. The value C_s in Table 6.3 is the mean C_s value obtained for the Phase 2 OPC specimens. The mean *m* value used in the probabilistic service life prediction is 0.26, again in line with Thomas et al.

From Table 6.3 it can also be noted that the *m* value distribution is truncated at a lower bound of 0 and an upper bound of 0.8. The lower bound truncation is logical as it is not expected that the D_{app} value for any of the concretes examined herein will increase with time. When considering the upper bound, it is noted that the adopted formula from Thomas et al. (2008) imposes an upper bound of 0.6. This value is, however, for deterministic assessment. However, Figure 6.10 illustrates that an upper bound on the *m* value probability distribution function (PDF) is necessary due to the nature of the $D_m(t)$ calculation presented in Equation 6.3.

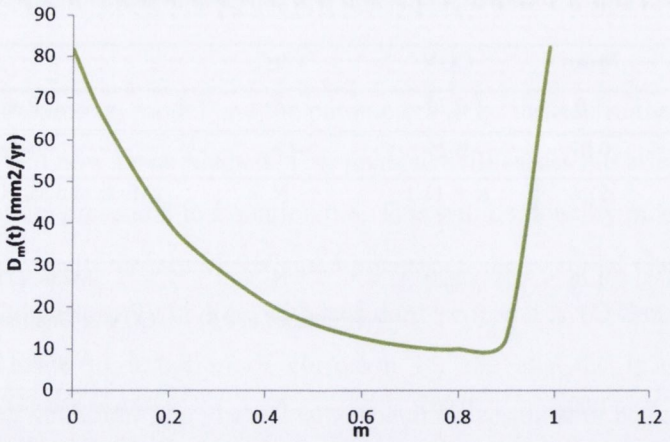
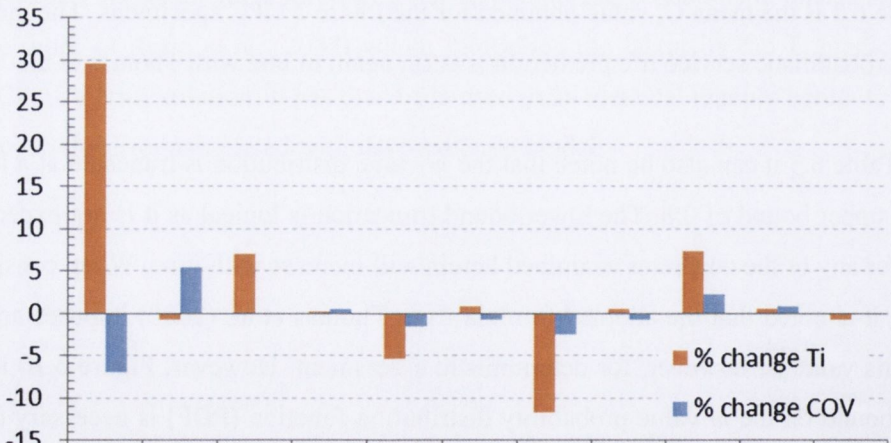


Figure 6.10: Change in $D_m(t)$ across a range of m values for a given D_{app} value at t = 40 years

As can be seen from the figure above, once the value of *m* goes beyond 0.8 the $D_m(t)$ value becomes distorted. This is due to the effect of the (1/1-m) term in the $D_m(t)$ formula resulting from the D(t) integration. For values from approximately 0.8 to 1 this term causes a sharp increase in $D_m(t)$. This behaviour has no logical basis and is purely down to the influence of the mathematics in Equation 6.3. Consequently, the *m* value PDF will be truncated at 0.8 for all probabilistic calculations in this thesis.

The results of the sensitivity analysis are presented below in Figure 6.11. It is noted that the base analysis, with all parameters as per Table 6.3, resulted in a 50% probability of corrosion initiation at 7.18 years. The COV value associated with the 100,000 calculated T_i values was 0.66.



	Cd μ	Cd COV	Ccr μ	Ccr COV	Cs μ	Cs COV	Dapp-R μ	Dapp-R COV	m μ	m COV	Model Error cov
% change Ti	29.6	-0.7	7.0	0.3	-5.3	0.8	-11.7	0.5	7.3	0.5	-0.8
% change COV	-7.0	5.4	-0.1	0.5	-1.5	-0.7	-2.4	-0.8	2.3	0.8	-0.1

Figure 6.11: Results of sensitivity analysis 1 (+ 10%) utilising Phase 2 OPC parameters

Firstly, considering the percentage change in the 50% probability of T_i (referred to as predicted T_i), it is clear that cover depth has the greatest influence on this value. A 10% increase in the mean C_d results in a 29.6% increase in T_i. The next most influential parameter is D_r with a 10% mean increase resulting in an 11.7% reduction in the predicted T_i value. The magnitude of the effect of the mean C_s value increase and the *m* value increase are similar, with changes of approximately 7% observed for both. As expected an increase in C_s decreases T_i while the increase in *m* increases T_i.

It can also be seen from the plot that C_d is the only parameter whose 10% increase in mean value has a notable effect on overall T_i COV with the increase in mean C_d resulting in a 5.4% decrease in T_i COV. This is because the C_d standard deviation remains constant regardless of the magnitude of C_d, in accordance with the findings of Mirza and MacGregor (1979).

On examining the effect of the 10% increase on parameter COV, it is noted that the C_d COV increase has a notable impact, increasing the overall COV by 5.4%. The COV increase for the remainder of the parameters has little influence on either mean T_i or overall T_i COV.

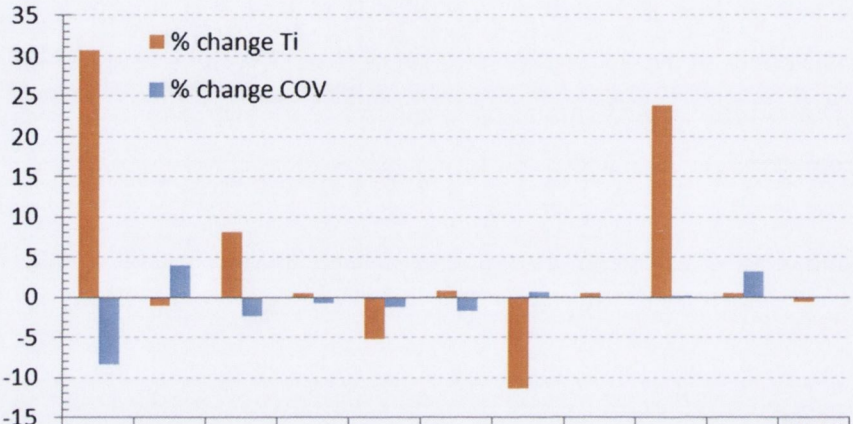
Having explored the sensitivity analysis in terms of OPC, it is noted that the PFA and GGBS concretes result in far larger T_i values and have a substantially larger *m* value than the standard OPC SCC option. Consequently, it was considered possible that the model sensitivity, particularly to *m*, could vary notably from OPC to OPC + PFA and OPC + GGBS. To investigate this, a second sensitivity analysis was carried out utilising the Phase 2 PFA SCC parameters with the appropriate *m* value of 0.50 in accordance with Equation 6.9 for OPC + 30%PFA. The parameters varying from the analysis above are presented in Table 6.4. The mean T_i for the PFA parameters was 33.41 years with a COV of 0.89.

Table 6.4 Statistical parameters for probabilistic analysis

Property	Units	Mean	COV	Dist.	Source
m value PFA	_	0.50	0.20	N (truncated at 0.8)	Mean: (Thomas et al. 2008) COV & Dist: (Kwon et al. 2009)
PFA C_s	^{1}%	1.13	0.08	N	Mean and COV: Phase 2 Exp. Results Distribution: (Stewart and Suo 2009; Vu and Stewart 2005)
PFA D_r	mm^2/yr	36.55	0.12	N	Mean and COV Phase 2 Exp. Results Distribution: (Choe et al. 2009; Stewart and Suo 2009)

Dist. = Distribution, N is normal distribution, Exp. = Experimental

The results of the PFA SCC sensitivity analysis are presented below in Figure 6.12. As can be seen from the plot the effect of the 10% increase in *m* has been greatly increased. A 10% increase in *m*, now 0.050 (10% of *m*=0.50) as opposed to 0.026 (10% of 0.26) in the previous analysis, results in a 23.8% increase in predicted T_i. The percentage change due to the 10% increase in the other parameter mean values has remained unchanged.



	Cd μ	Cd COV	Ccr μ	Ccr COV	Cs μ	Cs COV	Dr μ	Dr COV	m μ	m COV	Model Error cov
% change Ti	30.61	-1.12	8.20	0.55	-5.29	0.76	-11.48	0.55	23.80	0.55	-0.67
% change COV	-8.32	3.92	-2.29	-0.72	-1.20	-1.72	0.68	-0.06	0.13	3.23	-0.10

Figure 6.12: Results of sensitivity analysis 1 (+ 10%) utilising Phase 2 PFA parameters

Overall, considering the two sets of sensitivity analysis, it can be concluded that the predictive model is most sensitive to the C_d parameter. The D_r parameter also has a significant influence on mean T_i across the two sets of analysis. It was also found that the larger the *m* value the greater the sensitivity of the model to changes in the *m* value. Thus for the concretes containing PFA and GGBS, the *m* influence will be close to that of C_d. Finally, it should be noted that each sensitivity analysis was carried out a number of times with the percentage effects of the parameters changing from analysis to analysis by less than 1%. Thus the sensitivity analysis presented above is stable and representative of the sensitivity of the model to each parameter for a given Monte Carlo simulation.

6.4 CONCLUSIONS

The work described in this chapter has contributed to addressing Research Question 3 by developing the basis for the probabilistic model which is used in Chapter 7 to consider the service life consequences of adopting different concrete options. Having presented a number of predictive models from the existing literature Fick's second law with Crank's error solution was deemed most appropriate. Based on the need to compare different concrete types in a fair manner it was established that a Fickian model incorporating time dependant diffusion should be utilised herein. Use of the proposed model in a probabilistic analysis highlighted an issue with utilising this popular service life model as a basis for probabilistic analysis. A practical and theoretically grounded solution to the problem was presented. This solution allows the time dependent diffusion to be incorporated into Fick's law in a probabilistic framework, with appropriate *m* values, without the calculation of unreasonably high, erroneous T_i values, which skew the probabilistic results. The selection of appropriate model parameters from the literature was also presented in this chapter in some detail. Finally, a sensitivity study analysis was carried out to investigate the influence of the statistical model parameters on the overall probabilistic output. It was found that C_d was the most influential parameter, followed by D_r for OPC concrete. It was also found that the influence of the *m* value parameter is comparable to the C_d parameter for concretes containing SCMs.

7. PROBABILISTIC MODELLING RESULTS

7.1 INTRODUCTION

The Introduction section of the previous chapter established the advantages associated with combining predictive modelling and experimental testing to explore the marine durability of different concretes over service life durations. It was also noted in the previous chapter, however, that there is considerable uncertainty associated with modelling chloride ion ingress and predicting the resultant reinforcement corrosion due to a) the complex nature of the chloride migration process (Climent et al. 2002), and b) and the high levels of uncertainty associated with some of the chloride ingress model parameters and time dependent effects (Angst et al. 2009; Nokken et al. 2006). These high levels of uncertainty make probabilistic methods the optimal analytical tool for such studies, due to their ability to incorporate and quantify uncertainty associated with the prediction of service life performance. As established in Chapter 3, there is a distinct lack of literature utilising probabilistic methods to examine different concrete options using probabilistic methods. This shortcoming is particularly relevant for SCCs, as no long term marine durability data exists to date for this new concrete technology. This situation led to the development of Research Question 3 in Chapter 3 as follows;

Research Question 3: How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

Having established the probabilistic model necessary to address this research question in Chapter 6, this chapter presents the probabilistic model results which directly address the question. The first section of this chapter examines the predicted long term performance of a number of SCC options through incorporating the Phase 2 testing results in the probabilistic framework. The nature of the probabilistic model output is discussed in terms of key influencing factors and a form of sensitivity analysis is utilised to identify the model parameters which contribute most to overall model output uncertainty. Section 7.2 also examines the effect of calibration of accelerated experimental test results on predicted long term performance of different SCCs, and on the relative performance of the different SCCs. This is an important consideration in the context of the statement by Li (2000), who noted that the link between results from experimental testing and conditions typical of service is a significant issue that requires the continuous efforts of researchers.

Having compared a number of SCCs, Section 7.3 further addresses Research Question 3 by investigating the SCC vs. VC comparison over service life timeframes. This allows the SCCs to be

benchmarked against VCs over times applicable to service life; investigating if differences identified in the experimental study in Chapter 5 still exist for over longer durations.

Section 7.4 of this chapter carries out a similar type of investigation, but this time in terms of Research Question 2, by examining if the effect of exposure conditions identified in Chapter 5 is diminished by the incorporation of long term time dependent effects.

Finally, Section 7.5 of this chapter presents Ferrycarrig Bridge as a case study whereby the time to initiation of corrosion is predicted for the Ferrycarrig Bridge crosshead beams. This analysis facilitates examination of the OPC + increased cover option. The case study also provides the predicted service life of the crosshead beams with the location specific exposure level for each crosshead beam taken into account.

7.2 PROBABILISTIC MODELLING AND CALIBRATED ANNALYSIS OF SCC

The first half of this section examines the Phase 2 salt fog chamber SCC results using the probabilistic model developed in Chapter 6. The model output is discussed and the nature of the probabilistic model and time dependant effects are examined. Subsequently, the effect of calibration of the accelerated test results using marine data is investigated. This area is explored in the context of the Phase 2 SCC results using the Ferrycarrig Bridge imitation mix tested in Phase 2, together with data collected from the pre-repaired Ferrycarrig Bridge in 2007 by O'Connor and Kenshel (2013). It is noted that the OPC-S SCC option is not examined in this section. This is primarily due to the lack of information in the existing literature relating to the effective life of surface treatments (Stewart et al. 2012). As shown in Chapter 5, there are also high levels of silane performance uncertainty associated with silane application uniformity and moisture content of concrete sub-strata upon silane application. The lack of any statistical information in the literature relating to such effects makes probabilistic modelling of OPC-S difficult i.e. one cannot represent uncertainty which is not understood. However, the OPS-S is examined in a probabilistic framework in the Ferrycarrig Bridge case study in Section 7.5, where it is noted that the predicted OPC-S performance is purely indicative and must be treated with caution.

The statistical parameters which remain constant throughout the probabilistic analysis in this chapter are presented in Table 7.1. The selection of these parameters has been discussed in detail in the Chapter 6, will not be discussed again in this chapter.

Table 7.1 OPC, PFA and GGBS *m* values from the available literature

Property	Units	Mean	COV	Dist.	Source
ER_{mod}	_	1.0	0.2	LN	(Rafiq et al. 2004)
C_{cr}	*%	0.07	0.25	LN	(Kenshel 2009)
Cover, x	mm	Specified + 1.6 (Specified=50 mm)	σ = 11.1	N	(Mirza and MacGregor 1979; Stewart and Suo 2009)
t_c	years	25	0.25	LN	Mean: (Life 365 Consortium II et al. 2012)
OPC *m* value	_	0.26	0.20	N (trunc. @ 0.8)	Mean: (Thomas et al. 2008) COV & Dist: (Bentz 2003; Ferreira 2008; Van den Heede et al. 2012)
PFA *m* value	_	0.50	0.20	N (trunc. @ 0.8)	Mean: (Thomas et al. 2008) COV & Dist: (Bentz 2003; Ferreira 2008; Van den Heede et al. 2012)
PFA(k) *m* value	_	0.46	0.20	N (trunc. @ 0.8)	Mean: (Thomas et al. 2008) COV & Dist: (Bentz 2003; Ferreira 2008; Van den Heede et al. 2012)
GGBS *m* value	_	0.60	0.20	N (trunc. @ 0.8)	Mean: (Thomas et al. 2008) COV & Dist: (Bentz 2003; Ferreira 2008; Van den Heede et al. 2012)

Table 7.2 below presents the SCC C_s and D_{app} values obtained from the Phase 2 experimental program. The D_r value for each SCC is also presented in the table. As discussed in the previous chapter these D_r values were calculated from the D_{app} values obtained from the experimental testing using Equation 6.4, with *m* values adopted from the work of Thomas et al. (2008). The D_r values represent the rate of diffusion within the concrete at a reference time t_r. The use of this instantaneous D_r value approach facilitates incorporation of a) the time dependant diffusion effects, b) the effect of age of the test specimens upon chloride exposure and c) the cessation of the reduction in the instantaneous diffusion coefficient with time at a given point. As per the Phase 2 timelines, the age of the concrete at chloride exposure, $t_{ex.}$ was 10.5 weeks and the length of chloride exposure, t, was 36 weeks. As discussed in the previous chapter, the COV values for the diffusion coefficient and the surface chloride content are based on the COV values obtained in the experimental program. These parameters are modelled as normally distributed in accordance with the existing literature (Choe et al. 2009; Stewart and Suo 2009; Vu and Stewart 2005).

Table 7.2 Statistical Parameters for Phase 2 SCCs

Concrete Type	C_s		D_{app}		D_r	
	Normal Dist.				Normal Dist.	
	μ	COV	μ	COV	μ	COV
OPC SCC	1.00	0.065	81.47	0.090	74.73	0.090
PFA SCC	1.13	0.082	43.60	0.123	36.55	0.123
PFA(k) SCC	0.99	0.059	31.66	0.064	26.96	0.064
GGBS SCC	1.00	0.068	25.07	0.106	20.18	0.106

In 2009 it was pointed out by Gulikers that there is no generally accepted probability of corrosion initiation which is utilised for the assessment of the vulnerability of marine structures. Two probabilities of corrosion initiation will be considered in this chapter to assess the various concrete types. The first is the 50% probability of corrosion initiation. This probability was utilised by Bentz (2003) to indicate the likely expected time to chloride induced corrosion. The time corresponding to the 50% probability of corrosion initiation will be referred to herein as the predicted T_i value. The 10% probability of corrosion will also be considered in the probabilistic analysis in this chapter. This value was specified as the depassivation limit state in the FIB Model Code (CEB-FIB 2006). This 10% probability of corrosion initiation has been used by a number of researchers in the existing literature as an indication of service life (Ferreira 2004; Van den Heede et al. 2012). Given the high variability of T_i predictions (discussed below), the 10% probability of corrosion initiation would seem an appropriate durability design specification tool to assess the required cover and quantity of SCMs necessary to achieve a given service life. However, this limit state has also been adopted by researchers for service life predictions of existing structures (Song et al. 2009). This would seem overly conservative, as will be discussed in the context of the uncertainty associated with predictions below.

The results of the probabilistic output for the four SCCs are presented below in Figure 7.1 in the form of a Cumulative Distribution Function (CDF) plot. The Monte Carlo simulation was performed with 100,000 realisations of T_i for each of the SCCs. The plot shows the probability of corrosion initiation for each of the SCCs from 0 to 200 years. The 10% probability line and 50% probability line are also plotted in Figure 7.1. Table 7.3 presents the exact figures relating to the 10% and 50% probability of corrosion initiation, in addition to the relative merits obtained from the Phase 2 experimental results analysis. The COV value associated with each T_i Probability Distribution Function (PDF) is also presented in the table, facilitating quantification of the variation associated with the prediction of time to initiation of corrosion.

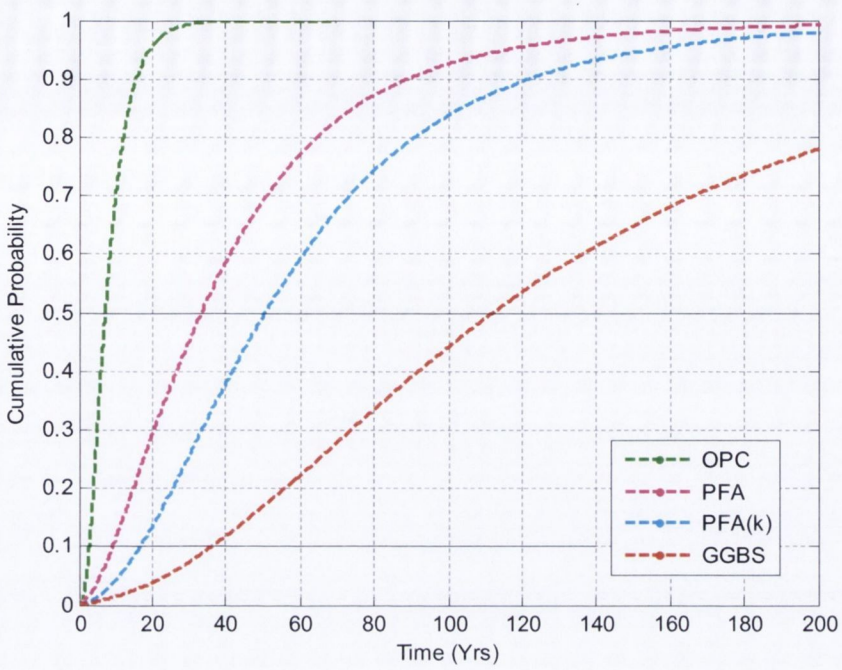


Figure 7.1 CDF of time to initiation of corrosion for four Phase 2 SCCs

Table 7.3: Probabilistic output from the Phase 2 SCC results

Durability Option	50 % Probability (Predicted T_i)	R.M P(0.50)	10% Probability	R.M P(0.10)	COV	Experimental RM
OPC SCC	7.2	1.00	2.8	1.00	0.67	1.00
PFA SCC	33.2	4.65	9.1	3.25	0.88	1.78
PFA(k) SCC	50.5	7.06	16.7	5.99	0.78	2.57
GGBS SCC	112.1	15.68	36.2	12.99	0.84	3.25

Firstly, considering the magnitudes of the predicted times to initiation of corrosion (50% probability of corrosion initiation) it can be noted, that for these accelerated test conditions the OPC SCC results in a small predicted T_i value of approximately 7.2 years. Despite the accelerated salt fog chamber conditions, the predicted T_i value for the GGBS SCC is still large at a value of 112.1 years. This may seem very high in the context of accelerated testing, however, it should be remembered that the specified cover adopted in the examples herein is 50mm. If an identical probabilistic analysis is carried out for a 20mm cover, the predicted T_i values for OPC, PFA, PFA(k) and GGBS are 0.7 years, 1.1 years, 2.5 years and 4.1 respectively. These figures are in general agreement with results of Asrar et al. (1999), who carried out salt fog chamber testing for up to 2 years and found that corrosion initiated after 0.75 years in the OPC samples. Asrar et al. found that a more durable concrete containing 10% silica fume resulted in corrosion initiation in only a small number of samples after 2 years.

Examining the 10% probability of corrosion initiation, it can be seen from Figure 7.1 and Table 7.3 that according to the serviceability limit state specified in the FIB model Code, none of the four SCC mix designs with 50mm cover achieved a 50 year design life for the salt fog chamber exposure conditions. The OPC SCC FIB Model Code service is seen to be particularly short at just 2.8 years.

Although the magnitude of the T_i values obtained are of interest, what is of primary concern in this analysis under salt fog chamber exposure conditions, is the relative performance of the concretes containing SCMs. To this end the predicted long term performance of each SCC alternative relative to the OPC SCC are specified in Table 7.3. Taking the 50% probability Relative Merits (Relative Merit will be referred to as RM in this chapter) as an example, the GGBS SCC RM was obtained by dividing the GGBS T_i value corresponding to the 50% probability of corrosion initiation by the OPC T_i value corresponding to 50% probability of corrosion initiation.

As can be seen from Table 7.3 the probabilistic modelling has increased the relative performance of the SCCs containing SCMs over the relative performance identified in the experimental results analysis in Chapter 5. This is due to the incorporation of the time dependant diffusion effects in the probabilistic model in the form of *m* values. As established in Chapter 6, these *m* values are greater for concretes containing SCMs. The GGBS experienced the biggest increase in performance with the RM values at the 10% and 50% probability of corrosion increasing by a factor of 4.0 and 4.8, respectively, over the experimental RM values. This greater increase in the GGBS relative merit is due to a combination of the fact that a) GGBS has the greatest *m* value in accordance with Equation 6.9, and b) GGBS has the lowest D_r value before the application of the *m* coefficient. This lower D_r value results in more of the GGBS T_i simulations reaching the t_c value (t_c = time at constant $D_r(t)$, equal to 25 years), which means greater advantage is obtained from the time dependant diffusion. In contrast, the higher D_r value for the OPC SCC means that a small portion of the OPC SCC T_i simulations obtained the full advantage from the time dependant diffusion, with approximately 96% of OPC T_i simulations less than 20 years, as seen from Figure 7.1. In practical terms this means that, for the majority of cases, corrosion initiates in the OPC SCC before the concrete reaches full maturity (in a chloride diffusion sense). Thus, for these exposure conditions the OPC SCC does not obtain full advantage from the fact that the instantaneous diffusion coefficient of concrete decreases with time up to approximately 25 years. This effect also causes lower SCM RMs in the 10% probability of corrosion initiation than in the 50% probability of corrosion initiation due to the fact that the 10% probability of corrosion initiation occurs at a shorter time. This will be discussed further in the context of the calibrated test results. The two SCCs containing PFA also experience a notable increase in performance from experimental test results to

probabilistic modelling. Again this is due to the time dependant effects incorporated in the probabilistic modelling. The PFA SCC can be seen to increase by a factor of 1.8 and 2.6 for the 10% and 50% probabilities of corrosion initiation, respectively. The corresponding figures for the PFA(k) SCC are 2.3 and 2.7.

It is important to consider the results discussed above in the perspective of the COV values presented in Table 7.3. As can be seen from the table the COV values associated with the T_i PDFs are high, ranging from 0.67 to 0.89. These high values are a reflection of the stochastic nature of chloride ingress and the high levels of uncertainty associated with modelling both the movement of chlorides into concrete and the resultant initiation of reinforcement corrosion. When it is considered that the standard deviation associated with the T_i values is as large as 85% of the mean T_i value, the 10% probability of corrosion initiation set as the limit state in the FIB Model Code seems appropriate for durability design. It still, however, seems somewhat conservative for service life predictions, depending of course on the inspection interval, consequence of failure and a number of other parameters. The detailed discussion of this area is, however, outside the scope of the work in this thesis.

In the context of reliable service life predictions it is desirable to reduce the COV values associated with the prediction of T_i. In order to do this the uncertainty surrounding the model parameters must be reduced. To highlight which areas should be the focus of future research efforts aimed at reducing this uncertainty the primary parameters contributing to the large overall COV value must be identified. This will be done by preforming a sensitivity analysis in accordance with the work of Vu and Stewart, in which each parameter is considered to be a stochastic variable in turn, while all other parameters are fixed at their mean values (Vu and Stewart 2000). As stated by Vu and Stewart, this will allow the absolute influence of each random variable on the T_i variability to be calculated in percentage terms. The GGBS SCC was utilised for the sensitivity study which had a COV of 0.84. The results of this analysis are presented below in the form of a pie chart in Figure 7.2.

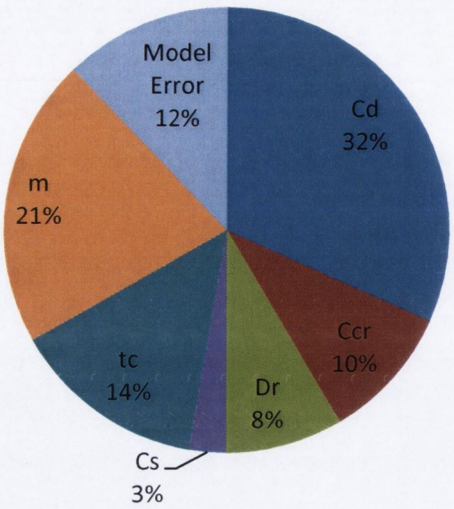


Figure 7.2: Relative importance of variability of parameters on variability of T_i

As can be seen from the Figure 7.2 the C_d and *m* parameter variability have the greatest influence on the overall T_i variability at 32% and 21%, respectively. Model error, t_c, critical chloride content and the diffusion coefficient also have a notable influence on overall T_i uncertainty. Considering C_d first, the variability surrounding this parameter was obtained from the work of Mirza and MacGregor (1979), and is related primarily to workmanship and errors on site. The C_d parameter is easily measured, and consequently, it may be possible to reduce the uncertainty for the analysis of a given structure through a covermeter survey. This will, however, depend on the quality of workmanship during the construction of the structure. If workmanship was poor the uncertainty surrounding the depth of cover concrete at a given point will increase. The *m* value, however, is not an easily measured or understood parameter. The uncertainty associated with this parameter and the t_c parameter (combined influence = 35%) are due to a lack of knowledge surrounding the time dependant diffusion coefficient. Therefore, from a research perspective the best way to reduce uncertainty surrounding service life predictions in marine environments is to focus research efforts on the determination of the time dependant diffusion process. This reiterates the need for research in this area, pointed out in Chapter 6.

Having examined the Phase 2 SCC probabilistic model results and the nature of the probabilistic model output variability, the effect of the calibration of the statistical input parameters, using marine bridge data, will now be presented. As stated in Chapter 5, the bridge data used for the calibration was obtained by O'Connor and Kenshel (2013) in 2007 from Ferrycarrig Bridge, before the crosshead beams were extensively repaired. The extensive site investigation, carried out by O'Connor and Kenshel, involved extraction of a total of 42 concrete cores from five of the seven

pre-repaired crosshead beams at Ferrycarrig Bridge. O'Connor and Kenshel obtained total chloride content profiles from the cores and fitted Fick's second law with Crank's error solution to these profiles to obtain 42 values of C_s and D_{app}. These C_s and D_{app} values reflect both the exposure conditions at Ferrycarrig Bridge and the material properties of the crosshead beam concrete poured in the original construction of the bridge in 1980. The Ferr. Orig. mix, discussed in Chapter 5, was used in the calibration to attempt to account for the effect of material properties allowing the isolation and quantification of the Ferrycarrig Bridge exposure effects. This was done through the development of calibration factors using the C_s and D_{app} values obtained from the Phase 2 experimental testing together with the 2007 site investigation data. It is hoped that these calibration factors for C_s and D_r reflect the difference in exposure conditions between the salt fog chamber and the Ferrycarrig Bridge site. It is recognised, however, that this is an approximate calibration with a number of limitations such as:

- A lack of an exact concrete mix design for the original Ferrycarrig Bridge mix
- A lack of knowledge as to how exactly the accelerated test conditions will have affected the performance of the Ferr. Orig. mix
- The effects of micro cracks on the ingress of chlorides have not been accounted for in the probabilistic model or Phase 2 testing, however, these cracks are likely to have played some unknown role in the ingress of chlorides at Ferrycarrig Bridge

Despite the limitations listed above, a calibration of the experimental results is warranted given the quality of data available for the Ferrycarrig Bridge site, and the discussion in Chapter 5 which established the need to investigate the calibration of accelerated test results (Li 2000). The probabilistic framework allowed some of the uncertainty associated with the calibration to be incorporated into the analysis. This was done through an additional model error factor which was included in the calibrated probabilistic analysis. The issue of calibration of accelerated test results has not been addressed in a probabilistic framework in the available literature to date. Consequently, statistical parameters for this model error factor cannot be obtained from the existing literature. As a starting point, a calibration model error mean value of 1.0 was adopted (no mean bias) in this analysis with a calibration model error COV value of 0.20.

The average C_s values and D_{app} values obtained from the Ferrycarrig Bridge site investigation are presented in Table 7.4. It is noted that two sets of values are presented. The first set is the average D_{app} and C_s values across the 42 core locations. The second set of figures presents the average C_s and D_{app} values obtained from the 10 most critical cores obtained from Ferrycarrig Bridge. This second set of values represents the top 25% of exposure severity across the bridge. This set of

values will be utilised in this analysis, which considers the time to first corrosion in the crosshead beams. The Ferr. Orig. C_s and D_{app} results obtained from the Phase 2 experimental testing are also presented in Table 7.4.

Table 7.4 Calibration details

	Cs	D_{app}	m	D_r	t_r
Ferrycarrig Mean	0.22	10.45	0.26	20.05	0.692
Ferrycarrig Top 10	0.34	14.98	0.26	28.73	0.692
Ferr. Orig. Mix	1.02	73.97	0.26	67.85	0.692
Calibration Factors	0.33			0.42	

The C_s calibration factor was calculated by dividing the Ferrycarrig Bridge C_s value by the Ferr. Orig. C_s value. This figure therefore estimates the factor by which the C_s value obtained from salt fog chamber test conditions differs from the C_s resulting from 27 years exposure at the Ferrycarrig Bridge site. The diffusion coefficient calibration was carried out through the adjustment of the instantaneous diffusion coefficient, D_r. This rules out errors associated with comparing D_{app} values obtained at approximately 1 year and 27 years (1980-2007). The D_r values calculated for the Ferrycarrig site data and the Ferr. Orig. results presented in Table 7.4 were obtained using Equation 6.4 with the $t_r = 0.692$. This allowed instantaneous diffusion coefficient values to be obtained for both the site data and the experimental data at the same reference time as that utilised for the other Phase 2 concretes. The D_r calibration factor was then calculated in the same way as the C_s calibration factor, by dividing the Ferrycarrig Bridge D_r value by the Ferr. Orig. D_r value. The calibration of GGBS SCC C_s value is shown below in Equation 7.1 as an illustrative example of the calibration procedure.

$$C_{s\,\mathrm{GGBS\text{-}SCC\text{-}}CAL} = C_{s\,\mathrm{GGBS\text{-}SCC}} \times \mathrm{Cal}_{Cs}$$

Equation 7.1

$$\text{where } Cal_{Cs} = \frac{C_{s-Ferrycarrig.Bridge}}{C_{s-Ferr.Orig.Phase2}}$$

The chloride diffusion parameters for the calibrated probabilistic analysis are presented in Table 7.5.

Table 7.5: Calibrated C_s and D_r values

Concrete Type	C_s		D_r	
	Normal Dist.		Normal Dist.	
	μ	COV	μ	COV
OPC SCC	0.33	0.065	31.65	0.090
PFA SCC	0.37	0.082	15.48	0.123
PFA(k) SCC	0.33	0.059	11.42	0.064
GGBS SCC	0.33	0.068	8.54	0.106

The results of the calibrated probabilistic output for the four SCCs are presented in a CDF plot in Figure 7.3, which shows the probability of corrosion initiation from 1 to 200 years. Figure 7.4 shows both the calibrated and uncalibrated SCC probabilistic output. The calibrated CDF curves are shown as solid lines in this plot while the uncalibrated CDF curves are shown as dashed lines. Table 7.6 presents a summary of the calibrated SCC probabilistic analysis results. As with the previous analysis, and all analyses carried out herein, 100,000 realisations of T_i were carried out for each concrete durability option examined.

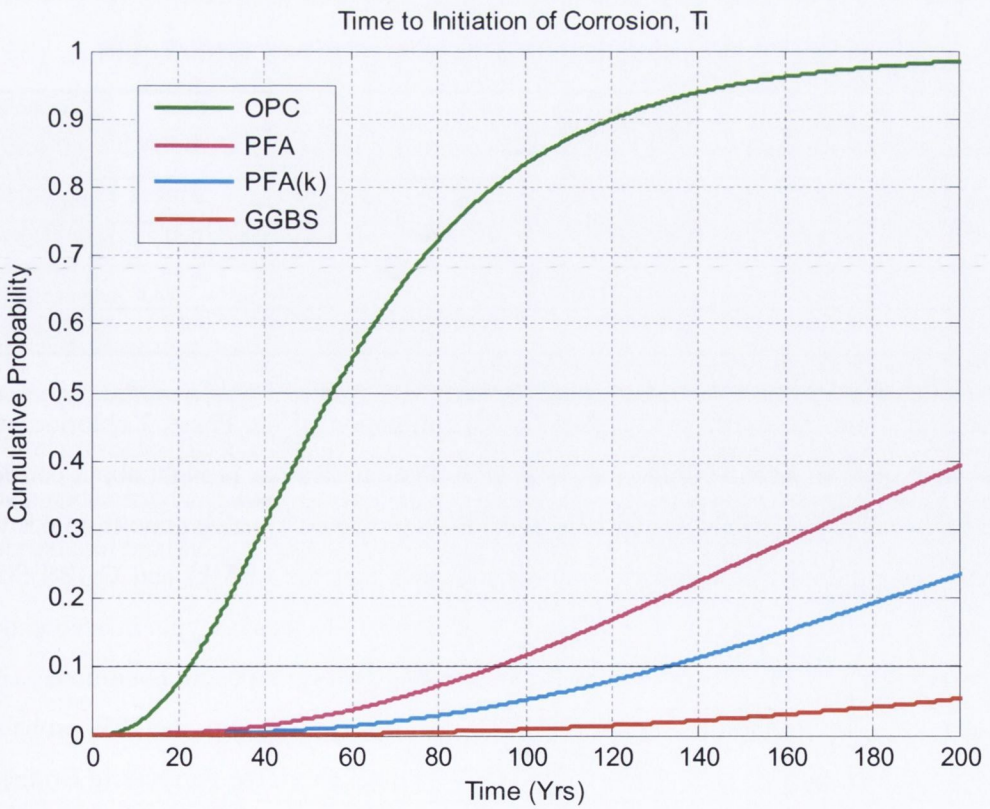


Figure 7.3 Calibrated CDF plot of time to initiation of corrosion for four Phase 2 SCCs

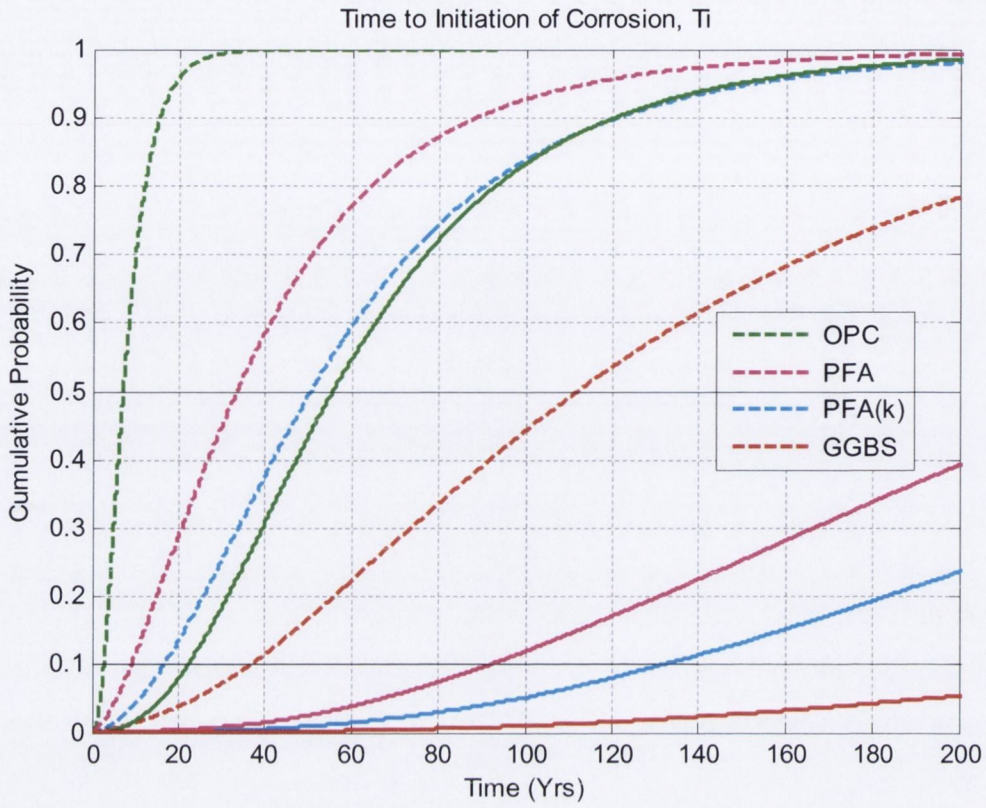


Figure 7.4 Comparison of CDF curves for calibrated and uncalibrated probabilistic results

Table 7.6: Probabilistic output from the calibrated Phase 2 SCC results

Durability Option	50 % Probability (Predicted T_i)	R.M P(0.50)	10% Probability	R.M P(0.10)	COV	Experimental RM
OPC SCC	55.7	1.00	22.4	1.00	0.68	1.00
PFA SCC	244.3	4.39	93.3	4.16	0.81	1.78
PFA(k) SCC	331.2	5.95	132.2	5.89	0.77	2.57
GGBS SCC	696.7	12.51	260.4	11.60	0.83	3.25

As can be seen from the figures and Table 7.6 the calibration of the Phase 2 chloride transport properties with the Ferrycarrig Bridge 2007 site investigation data has brought about considerable increases in the calculated T_i values, with the OPC SCC predicted T_i value gone from 6.8 years to 55.7 years. The calibrated probabilistic analysis indicates that the PFA(k) and GGBS SCC mix designs, with 50mm specified cover, satisfy 100 year service life requirements in accordance with the FIB Model Code (CEB-FIB 2006), for the Ferrycarrig Bridge exposure conditions. The PFA SCC is also very close to the FIB Model Code 100 year service life, with the 10% probability of corrosion initiation occurring at 93.3 years. By way of a model check the Ferrycarrig Bridge C_s and D_r values for the top 10 critical zones were also probabilistically modelled. The resultant predicted

T_i value was 61.7 years while the 10% probability of corrosion initiation occurred at 24.3 years. This is in general agreement with the findings at the bridge at the time of repair in 2007, when after 27 years of exposure, no significant signs of reinforcement corrosion were detected in the crosshead beams (National Roads Authority 2005).

Examining the RMs relating to the calibrated predicted T_i values it can be noted that the calibration has led to a decrease in RM value of 6%, 16% and 20% for the PFA SCC, PFA(k) SCC and the GGBS SCC, respectively. This reduction in RM values is due to the fact that the calibration of the input parameters has resulted in approximately 88% of OPC T_i values being greater than the mean t_c value of 25 years. The corresponding figure for the uncalibrated analysis is approximately 3%. Therefore, the performance of the OPC SCC has increased meaning a drop in relative performance of the SCCs containing SCMs. The difference in the RM decrease from the PFA SCC (6%) to the PFA(k) SCC and the GGBS SCC (16% & 20%) relates to the fact that in the uncalibrated analysis the PFA SCC RM was also influenced somewhat by the fact that its predicted T_i value was 33.2 years, meaning a considerable portion of the calculated PFA SCC T_i were less than t_c. Table 7.6, however, also illustrates that the RMs obtained from this calibrated analysis ate still far larger than those obtained from the experimental testing due to the influence of time dependent effects. Finally from Table 7.6, it noted that there is little difference between the 10% and 50% probability of corrosion initiation relative performance. Again this is due to the interaction of the *m* value and t_c value for OPC SCC explained above.

For both the calibrated and uncalibrated analysis presented above, the probabilistic modelling of the experimental results over times applicable to service life has led to an increase in the relative performance of the concretes containing SCMs. This effect is to be expected, but, it is necessary to investigate if the RMs obtained from probabilistic modelling with experimental results are of the same order of magnitude as the long term relative performance identified in the literature. To this end details of studies which compare the long term performance of concretes containing PFA and GGBS are presented below in Table 7.7. Particular attention was paid to those studies which examine performance at more than one exposure duration. The RM values from the literature, from the experimental testing and from the probabilistic modelling are presented in the dot-plot in Figure 7.5.

Table 7.7 Studies examining long term performance of GGBS and PFA concretes

Exposure	Exposure time (years)	Concrete mix			Relative merit	Notes	Reference
		Binder	w/b	Concrete Type			
Phase 2 - Experimental	0.7	30% PFA	0.44	SCC	1.78		Phase 2 Experimental
		PFA(k)	0.37	SCC	2.56		
		60% GGBS	0.44	SCC	3.25		
Phase 2 + Probabilistic Modelling		30% PFA	0.44	SCC	3.25, 4.16		Phase 2 + Probabilistic Modelling
		PFA(k)	0.37	SCC	5.99, 5.89		
		60% GGBS	0.44	SCC	12.99, 11.60		
Marine exposure site specimens (Splash Zone)	1,3,8	OPC	0.66	CVC	1.0, 1.0, 1.0	Specimens exposed in the splash zone	(Thomas and Bamforth 1999)
	1,3,8	30% PFA	0.54	CVC	1.7, 4.5, 10.8		
	1,3,8	70% GGBS	0.48	CVC	1.0, 5.0, 10.9		
Marine exposure site specimens (Tidal Zone)	2,4	OPC	0.49	CVC	1.0, 1.0		(Thomas and Matthews 2004)
	2,4	30% PFA	0.39	CVC	3.1, 10.9		
Marine exposure site specimens (Tidal Zone)	25	OPC	0.40	CVC	1.0	No duplicate samples	(Thomas et al. 2008)
		65% GGBS	0.40	CVC	14.9		
Marine exposure site specimens (Tidal zone)	15	OPC	0.45	CVC	1.0	Tidal cycles, 7.5 hrs wet, 4.5 hrs dry	(Mohammed et al. 2002)
	15	10-20% PFA	0.45	CVC	2.5		
	15	60-70% GGBS	0.45	CVC	7.1		

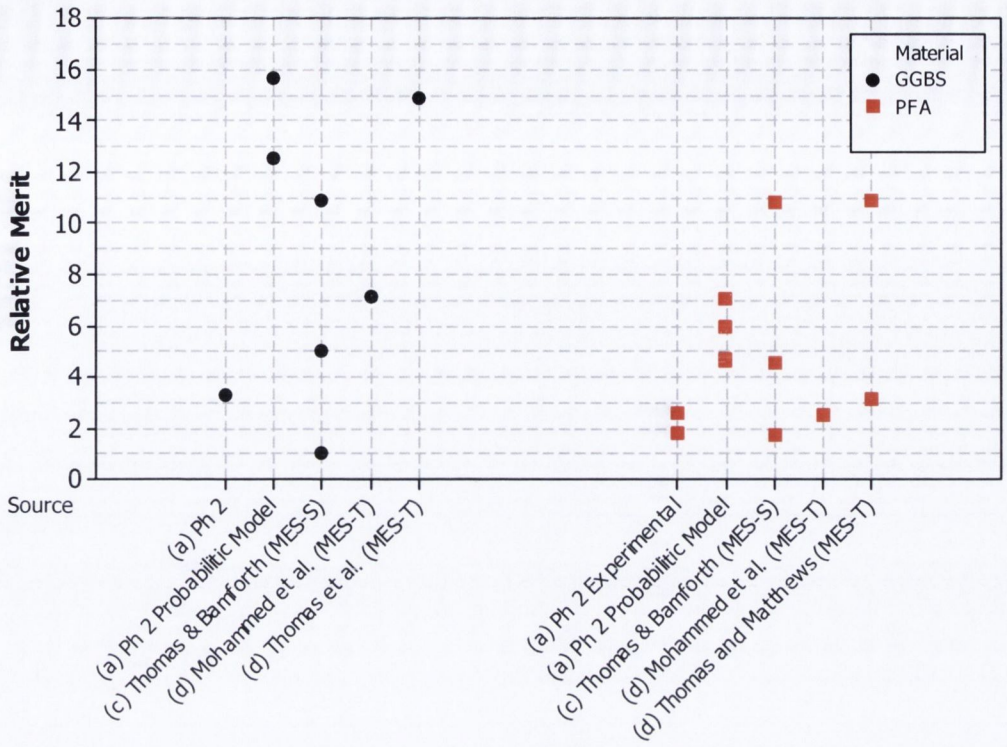


Figure 7.5 Comparison of probabilistic model results with long term performance from the literature

From Figure 7.5 it can be seen that the relative merit values obtained from the probabilistic modelling are in the same range as those of presented in the existing literature. The uncalibrated GGBS RM marginally represents the highest RM of the values plotted. The calibrated analysis however, results in GGBS RM value which is closer to the other values in the literature at 12.5. It is noted that the PFA relative merits calculated from the probabilistic analysis lie around the mid-point of the values plotted.

Overall, the probabilistic modelling incorporating the Phase 2 experimental test results has addressed Research Question 3 by examining the predicted performance of a number of different SCC options over timeframes appreciable to service life. For both the calibrated and uncalibrated analysis it was found that there was a notable increase in relative merits of the SCMs, due primarily to the effect of time dependant diffusion coefficient. In terms of predicted T_i values the, the calibration of the Phase 2 chloride transport properties with the Ferrycarrig Bridge data increased predicted T_i substantially. It was noted that the COV values associated with the model predictions values were high. A sensitivity analysis was carried out to highlight how level of uncertainty could be reduced through further research. Examination of the time dependent diffusion process was identified as the most powerful mean of reducing prediction uncertainty. Finally, the comparison of the probabilistic results and long term results from the literature existing literature showed that the

time dependant effects identified in long term studies and the probabilistic model output illustrated reasonable agreement.

7.3 COMPARISON OF VC AND SCC

As discussed in detail in Chapter 3, there is a need to compare the marine durability of SCC to traditional VC in a form of benchmarking (Sonebi and Nanukuttan 2009; Zhu and Bartos 2003; Zong et al. 2011). Chapter 5 addressed this need through short term experimental testing. In light of the lack of any long term durability data for SCC's in marine environments, this need will be further addressed in this section in the context of Research Question 3 through the utilisation of probabilistic modelling. Probabilistic assessment was carried out on two levels using the Phase 2 experimental results. The first level assesses the service life of SCCs and VCs for the salt fog chamber exposure conditions. The second level assesses the comparison of the SCCs and VCs for exposure conditions applicable to Ferrycarrig Bridge through the calibration of the Phase 2 experimental results according to the procedure outlined in Section 7.2. It is noted that the salt fog chamber represents an extremely harsh environment, while the crosshead beam location on the Ferrycarrig Bridge represents a moderately harsh marine environment, with no signs of serious reinforcement corrosion detected during the repair of the original OPC crosshead beams after 24 years of exposure, as discussed in Chapter 2 (National Roads Authority 2005).

100,000 values of T_i were calculated for each SCC and each VC for both the uncalibrated and calibrated service life predictions. The CDF plots for the uncalibrated T_i distributions for the OPC SCC and OPC VC are presented in Figure 7.6. The dashed line represents the OPC SCC while the solid line represents the OPC VC. The calibrated OPC CDF curves for the VC and SCC are presented in Figure 7.7. Table 7.8 presents the 50% and 10% probability of corrosion initiation, representing the predicted T_i value, and the FIB model code serviceability limit state respectively. The table also presents the performance of the OPC VC, relative to the OPC SCC, for both the 10% probability and the 50% probability. As with Chapter 5, this metric will be referred to as a Relative Performance or RP so that it can be distinguished from Relative Merit (RM) values which have been used in this thesis to compare OPC to alternative concrete durability options. Thus, relative performance values discussed in this section relate the performance of the VC to the SCC in a single figure. Taking the 50% probability of corrosion initiation as an example, it is simply obtained by dividing the median OPC VC T_i value by the median OPC SCC T_i value. The corresponding RP values obtained from the analysis of the experimental results in Chapter 5 are also presented in Table 7.8 to facilitate comparison.

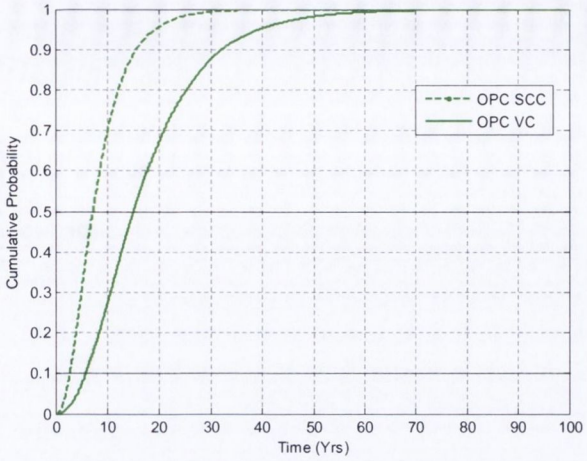


Figure 7.6 CDF plot of uncalibrated T_i distributions for OPC VC and OPC SCC

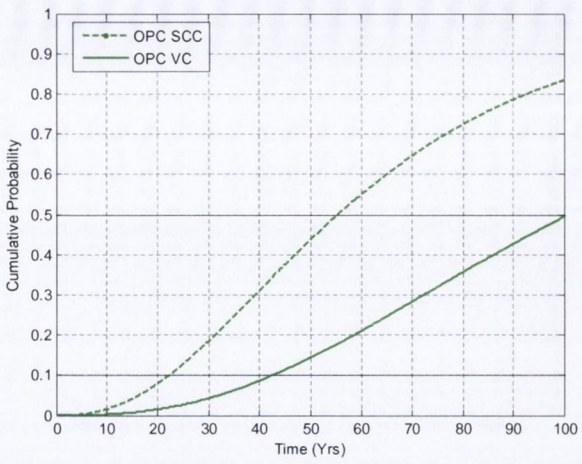


Figure 7.7 CDF plot of calibrated T_i distributions for OPC VC and OPC SCC

Table 7.8: Probabilistic output for OPC VC vs. OPC SCC

Durability Option	50 % Probability (Predicted T_i)	10% Probability	COV	RP P(0.50)	RP P(0.1)	RP Experimental	% Diff.
OPC SCC	7.2	2.8	0.67	2.09	2.10	1.73	+21%
OPC VC	15.0	5.9	0.65				
OPC SCC CAL	55.4	22.2	0.67	1.82	1.91	1.73	+5%
OPC VC CAL	100.8	42.5	0.65				

As can be seen from Figure 7.6 and Figure 7.7 the OPC VC resulted in significantly reduced probability of corrosion at each time step, when compared to the OPC SCC. The OPC SCC predicted T_i values and 10% probabilities of T_i were discussed in Section 7.2, the probabilistic analysis of the VC has not, however, been discussed up to this point. Table 7.8 shows the OPC VC predicted T_i value to be 15.0 years for the salt fog chamber exposure conditions and 100.8 years for the calibrated probabilistic analysis. The service life according to the FIB Model Code specification is 42.5 years, a considerable improvement over the OPC SCC value of 22.2 years (CEB-FIB 2006). However, as discussed in Chapter 5 the VC test specimens underwent full compaction in the Phase 2 experimental program. This is unlikely to be achieved for VC poured onsite, especially for structural bridge members with restricted access or congested reinforcement. The SCC by definition, however, is not susceptible to reductions in durability due to lack of compaction on site.

Examining the relative merit values of the VC, it is noted that the values obtained from the probabilistic analysis are greater than those obtained from the analysis of the experimental results. A 21% increase in RP was found for the uncalibrated analysis (1.73 to 2.09), and a 5% increase in

VC relative merit was identified for the calibrated analysis (1.73 to 1.82) based on the predicted T_i values. This is due to the influence of the time dependent diffusion and the t_c value. The VC is more durable than the SCC and thus obtains greater benefit from the time dependant diffusion coefficient even though both concretes have the same *m* value. Taking the uncalibrated analysis as an example, the SCC reaches a predicted T_i after 7.2 years. Once corrosion initiates, cracks will develop in the concrete within a small number of years, after which point the ingress of chlorides will be dominated by movement through the cracks (Li 2000). Consequently, only 7 years (plus time to crack >0.2mm) of the 25 year diffusion coefficient reduction period will make a significant contribution to the chloride resistance capacity of the OPC SCC. The predicted T_i value for the OPC VC on the other hand is 15 years. Thus, this concrete will gain advantage from approximately 15 years of the 25 year diffusion coefficient reduction period. Consequently, despite both the VC and SCC having the same *m* value the relative performance of the VC is increased by the incorporation of the *m* value in the probabilistic analysis. It is noted, however, that this effect will be reduced as the predicted T_i values for the concretes become greater than t_c, as can be seen from the comparison of the calibrated RP values and the uncalibrated RP values.

Overall it can be said that as found in the experimental testing, the VC results in a significantly more durable concrete over periods appreciable to service life predictions. The effect of improper vibration would have to reduce the durability characteristics of the VC by a factor of approximately two before the performance of the SCC and VC would be the same on site. Finally, it is noted that the COV values for the analysis range from 0.65 to 0.67. Again these values reflect the uncertain nature of this form of analysis and highlight the limitations associated with making long term service life predictions. These high COV values also reiterate the need for a probabilistic approach, as opposed to deterministic analysis, which fails to identify these high levels of uncertainty associated with predictions of time to initiation of corrosion.

The CDF plots for the PFA(k) SCCs and VCs are presented below in Figure 7.7 and Figure 7.8. The summary statistics for the uncalibrated and calibrated analysis are presented in Table 7.9. It is noted that the x-axis scales for the PFA(k) concretes and GGBS concretes are double the scale of the OPC x-axis scale above.

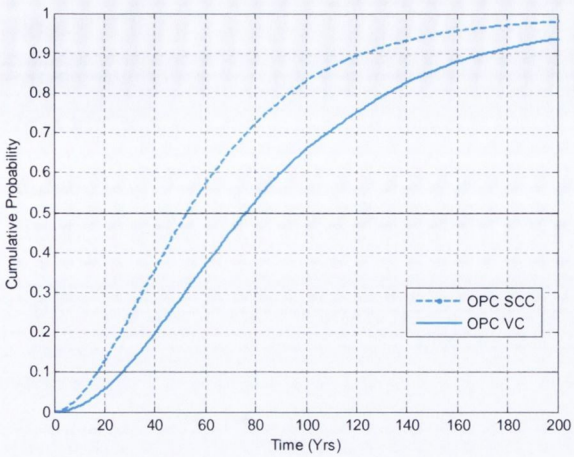


Figure 7.8 CDF plot of uncalibrated T_i distributions for PFA VC and OPC SCC

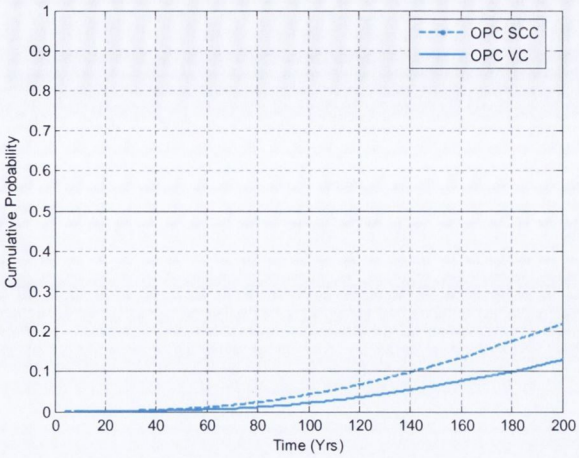


Figure 7.9 CDF plot of uncalibrated T_i distributions for PFA VC and OPC SCC

Table 7.9: Probabilistic output for PFA(k) VC vs. PFA(k) SCC

Durability Option	50 % Probability (Predicted T_i)	10% Probability	COV	RP P(0.50)	RP P(0.1)	RP Experimental	% Diff
PFA(k) SCC	51.1	17.1	0.81	1.45	1.55	1.32	+10%
PFA(k) VC	74.1	26.4	0.78				
PFA(k) SCC CAL	326.7	129.1	0.76	1.27	1.26	1.32	-4%
PFA(k) VC CAL	413.3	163.0	0.74				

As can be seen from the CDF plots the difference between the SCC and VC predicted long term performance is far smaller for the PFA(k) concretes than the OPC concretes. This is due to the greater chloride ingress resistance of the PFA(k) cement paste as discussed in Chapter 5. Examining the PFA(k) VC predicted T_i values it can be seen that a value of 74 years is obtained from the probabilistic analysis for the salt fog chamber exposure conditions. The calibrated analysis results in a predicted T_i value of over 300 years while the service life according to the FIB Model Code for the PFA(k) VC is 129 years for the Ferrycarrig Bridge exposure conditions. This indicates a very durable concrete.

The VC RP values obtained from the uncalibrated probabilistic analysis are 10% greater than those obtained from the experimental results analysis. Again, this is due to the more durable VC benefiting more from the time dependant diffusion due to its higher T_i values. This is not the case for the calibrated analysis, however. In this case, both the 10% and 50% probability of corrosion occur at times far greater than 25 years. Consequently, both the SCC and the VC have obtained equal advantage from the time dependant diffusion period. It is in fact noted that the VC RP values for the calibrated analysis are lower than the experimental RP values. This slight reduction is due to

the influence of the *m* value, which has reduced the effect of the difference in the initial diffusion coefficients obtained from the experimental testing. The COV values for the PFA(k) concretes are greater than those obtained in the OPC probabilistic analysis. This is due to the fact that the PFA(k) has a mean *m* value of 0.46 while the OPC concretes have a mean *m* value of 0.26. The greater *m* value pushes outlier points further to the right of the distribution increasing the spread of the T_i value distribution.

Overall, the probabilistic analysis has indicated that the long term performance of the VC is between 1.5 and 1.25 times better than the performance of the SCC, depending on the severity of the exposure condition being considered. Thus, it is foreseeable that depending on difficulties with access, congestion of reinforcement, and quality of workmanship, the SCC could actually perform better than the VC on site. Logic dictates that the degree to which compaction effects marine durability of concrete is extremely difficult to measure, given the amount of other contributing variables involved. Consequently, it is simply stated that the margin of error in compaction and workmanship, facilitated by the difference in VC and SCC service life performance for the PFA(k) concrete, is not large at 25% to 50%. Again it should be noted that there is considerable variation associated with the analysis carried out above. Finally the CDF plots for the GGBS SCC and VC are presented below in Figure 7.10 and Figure 7.11, while the summary statistics for the OPC + GGBS concretes are presented in Table 7.10.

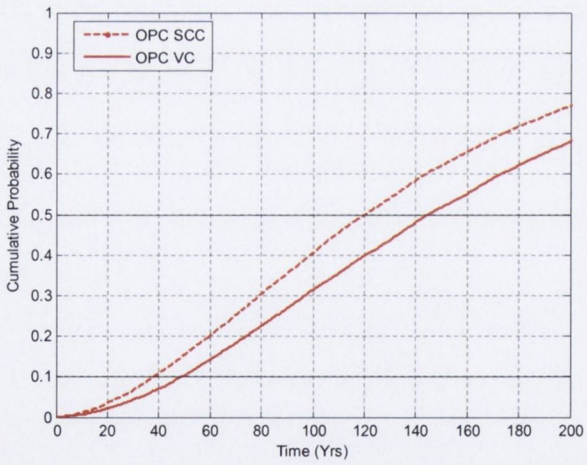


Figure 7.10 CDF plot of uncalibrated Ti distributions for PFA VC and OPC SCC

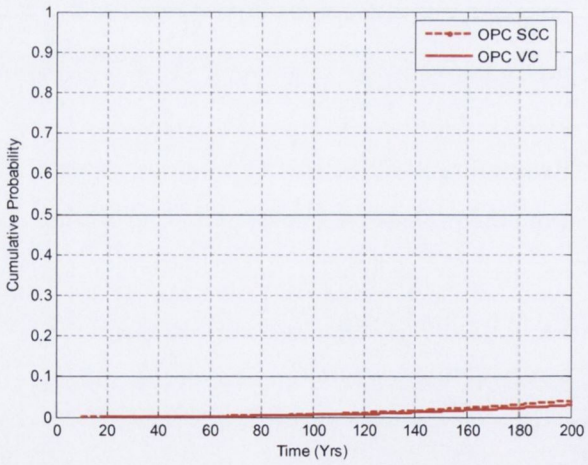


Figure 7.11 CDF plot of uncalibrated T_i distributions for PFA VC and OPC SCC

Table 7.10: Probabilistic output for GGBS VC vs. GGBS SCC

Durability Option	50 % Probability (Predicted T_i)	10% Probability	COV	RP P(0.50)	RP P(0.1)	RP Experimental	% Diff
GGBS SCC	113.5	36.4	0.84	1.21	1.26	1.16	+4%
GGBS VC	137.3	46.8	0.81				
GGBS SCC CAL	681.9	259.6	0.81	1.12	1.12	1.16	-3%
GGBS VC CAL	766.5	289.6	0.80				

From the two CDF plots, it is clear that the performance of the SCC and VC are closest for the GGBS concretes. This is reflected in the table with the predicted T_i values for the SCC and VC quite similar. It is notable that the predicted T_i value for the calibrated GGBS VC is 681 years while the FIB Model code service life prediction for this concrete is 258 year for the Ferrycarrig Bridge exposure condition. This indicates that corrosion will not occur over the service life of a structure with this VC and 50mm cover utilised. Again it is noted that the effects of microcracking are not accounted for in the analysis.

The probabilistic assessment RP values for the GGBS VC are 4 % higher than the experimental RP for the uncalibrated analysis and 3% lower than the experimental RP for the calibrated analysis. The reasons behind this have been discussed in the context of the PFA(k) results. It is, however, noted from the table that the RP values for the GGBS VC are close to 1. Given the high COV values this would indicate that there is little detectable difference between the durability performance of the GGBS SCC and the GGBS VC. Bearing in mind that the VC tested herein underwent perfect compaction, it is likely that the SCC will result in a greater durability performance than the VC on site. It is, however, acknowledged that the analysis conducted above does not account for the effects of shrinkage which will be far greater for the SCC given the high proportion of fines in the mix.

Considering the OPC, PFA(k) and GGBS probabilistic assessment of the long term relative performance of VC and SCCs it can be concluded that the analysis indicates that the OPC VC is substantially more durable than the equivalent SCC. However, the incorporation of SCMs results in a reduction in the difference between VC and SCC meaning that given the susceptibility of the VC to workmanship errors, the SCC could result in a more durable concrete. It is noted that the analysis above was only carried out for the Phase 2 exposure condition. Given the findings relating to Research Question 2 in Chapter 5 it would be interesting to carry out the comparison above in the context of different exposure conditions. It would seem likely that increased levels of drying

would reduce the relative resistance of the concrete paste containing SCMs, increasing the performance of VC relative to SCCs. It was not possible to investigate this herein as the SCC and VC were only tested together one exposure condition (Phase 2 testing).

7.4 EXPOSURE CONDITIONS

Section 5.6 of Chapter 5 examined the effect of varying exposure condition on the performance of the PFA SCC and the GGBS SCC, relative to OPC SCC in order to address Research Question 2. Three exposure conditions were examined using three sets of experimental test results. This section seeks to further address Research Question 2 by examining if the patterns identified in Chapter 5 from short term testing still holds firm when time dependent effects are incorporated over timeframes applicable to service life i.e. do time dependent effects diminish the identified effect of exposure conditions. Again this is done by utilising the model developed in Chapter 6. As with Chapter 5 the effect of exposure conditions will be examined in this section through comparison of the PFA SCC RMs and the GGBS SCC RMs across the three exposure conditions in a manner which eliminates the size/mould effect. Again, this will involve comparing the Phase 1 slab specimens to the Phase 2 slab specimens facilitating comparison of exposure condition 1 (driest test condition) to exposure condition 3 (wettest test condition). The Phase 1 cube specimens RM values will be compared to Phase 2 cube specimens RM values allowing comparison between exposure condition 2 and exposure condition 3.

The Phase 1 and Phase 2 diffusion coefficient and surface chloride content values used in the probabilistic analysis are presented below in Table 7.11. It should be noted that the t_{ex} value for the Phase 1 testing was 7.2 weeks while the t_{ex} value for the Phase 2 testing was 10.5 weeks. The difference between the Phase 1 and Phase 2 t_{ex} times was due to the soaking period adopted in Phase 2 testing and the greater number of concrete mixes prepared in the Phase 2 testing, which necessitated a longer sample preparation time than Phase 1. The effect of this difference in t_{ex} is incorporated in the probabilistic analysis presented herein though the calculation of D_r using Equation 6.4. The other parameters used in the service life prediction were the same as those presented previously in Table 7.1.

Table 7.11 C_s and diffusion coefficient mean and COV values for probabilistic analysis

Concrete Type	D_{app}	D_r		C_s	
		Normal Dist.		Normal Dist.	
		μ	COV	μ	COV
Phase 1 OPC slabs	42.97	37.83	0.13	0.94	0.17
Phase 2 OPC slabs	67.58	61.99	0.11	1.04	0.05
Phase 1 OPC cubes	71.82	63.23	0.04	1.77	0.02
Phase 2 OPC cubes	84.15	77.19	0.09	1.16	0.09
Phase 1 PFA slabs	39.67	30.60	0.08	1.08	0.05
Phase 2 PFA slabs	45.74	38.34	0.09	0.90	0.07
Phase 1 PFA cubes	43.38	33.46	0.09	1.95	0.06
Phase 2 PFA cubes	47.03	39.42	0.03	1.17	0.12
Phase 1 GGBS slabs	16.26	11.81	0.08	1.27	0.07
Phase 2 GGBS slabs	26.26	21.14	0.06	0.87	0.03
Phase 1 GGBS cubes	39.18	28.48	0.11	1.70	0.04
Phase 2 GGBS cubes	27.55	22.18	0.09	0.97	0.07

The probabilistic analysis of the relative performance of the PFA SCCs was carried out by calculating T_i PFA/T_i OPC for 100,000 simulations. Thus a distribution reflecting the performance of PFA relative to OPC was developed for each set of specimen results. An identical process was followed for the calculation of the GGBS RM distributions. Figure 7.12 below compares the CDF plots for the PFA RMs for exposure condition 1 and exposure condition 3. Figure 7.13 compares the CDF plots for the PFA RMs for exposure condition 2 and exposure condition 3. In each case the drier exposure condition is shown by the dashed line with the wettest exposure condition (No. 3) shown by the solid line. The legend in each plot indicates the sample set which was used in the analysis with the exposure condition number shown in brackets.

The median T_i values from the probabilistic analysis for the OPC SCC ranged from 6.2 years to 18.8 for the different exposure conditions. The PFA SCC median T_i values ranged from 25.9 years to 45.1 years for the different exposure conditions. The exact 50% probability, or median, RM values for each comparison are presented in Table 7.12 together with the COV values associated with each distribution of RMs. The RM values obtained from the experimental results are also presented. Comparison of the experimental results and the probabilistic results facilitates investigation into the influence of long term effects and age of concrete at exposure, on the change in RM across different exposure conditions.

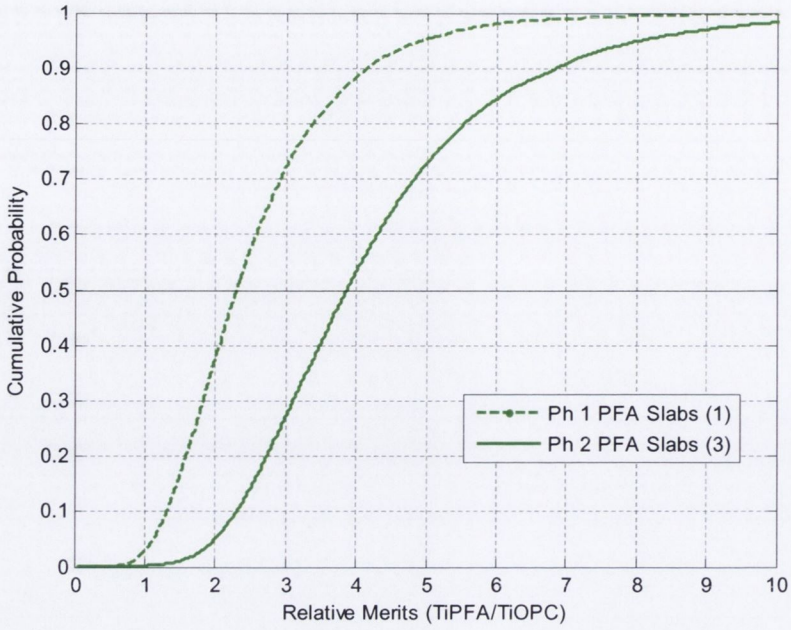


Figure 7.12 CDF of PFA relative merits values for exposure condition 1 and 3

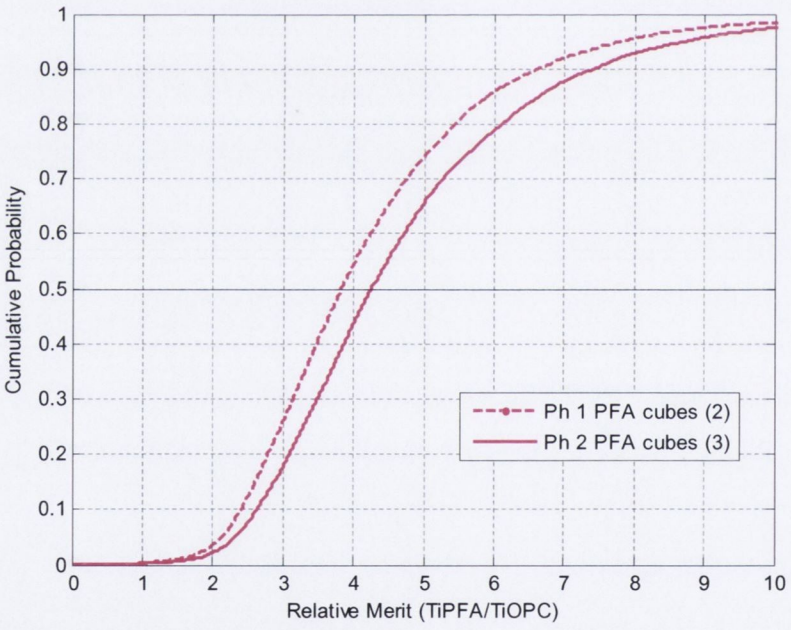


Figure 7.13 CDF of PFA relative merits values for exposure condition 2 and 3

Table 7.12 Probabilistic and experimental results of PFA relative merits for the two sets of exposure condition comparisons

Exposure Condition	Median RM	Percentage Reduction in RM	RM COV	Experimental RM	Percentage Reduction in RM
3 - Slabs	3.86		0.47	1.60	
1 - Slabs	2.30	43 %	0.49	0.95	41 %
3 - Cubes	4.25		0.45	1.76	
2 - Cubes	3.82	10%	0.45	1.60	9 %

Firstly, considering the Exposure condition 3 to 1 comparison, it is noted that the effect of change in exposure condition on RM is almost identical for the probabilistic modelling (43%) and the experimental testing (41%). The RM obtained from the probabilistic modelling have increased when compared to the experimental testing. Again, this is due to the incorporation of the effects of the time dependant diffusion in the probabilistic service life predictions. However, these time dependant effects, combined with the effect of incorporation of age of concrete at exposure.

To assess the role of the different t_{ex} values for the two phases of testing, two further probabilistic analyses were carried out with t_{ex} = 10.5 weeks for both sets of samples in the first analysis, and t_{ex} = 7.2 weeks for both sets of samples in the second analysis i.e. the different t_{ex} values are not accounted for, as was the case in the experimental results comparison in Chapter 5. For both of the probabilistic analyses a percentage reduction in RM of 45% from exposure condition 3 to 1 was identified. Thus, the incorporation of the variation in age of concrete at exposure in the analysis presented in Table 7.12 actually leads to a drop in percentage RM reduction of 2%. This reduction is to be expected given that exposure at a slightly earlier age in the Phase 1 testing (drier exposure condition) is a disadvantage to the PFA SCC which has a higher *m* value than the OPC SCC to which it is being compared. Therefore, the incorporation of the time dependant effects in the analysis lead to a 4% increase in percentage RM reduction, while the incorporation of the t_{ex} factor leads to a 2% drop in RM reduction. This results in a net increase in percentage reduction from experimental analysis to probabilistic service life prediction of 2%. These effects are small in this case, however, as will be seen below, the percentage influences of t_{ex} and time dependant diffusion are greater for some of the other exposure condition comparisons.

Examining the comparison between exposure conditions 2 and 3 reveals a similar pattern to that observed for the comparison of exposure condition 3 to 1 above. Again, the effect of exposure condition observed in the experimental testing (-9%) is maintained when examining relative

performance over service life durations using probabilistic methods (-10%). However, in this case when the role of the t_{ex} factor was explored with t_{ex} equal to 10.5 weeks for all specimens and t_{ex} equal to 7.2 weeks for all specimens, both analyses indicated a percentage reduction in RM of 18%. Consequently, the incorporation of the t_{ex} factor reduces the RM reduction by approximately 8% while the incorporation of the time dependant effects increases the RM reduction by approximately 9%. The influence of the two effects, therefore, leads to the 1% increase in RM reduction form experimental analysis to the probabilistic RM analysis.

The probabilistic analysis of the change in the GGBS performance across the same exposure conditions will now be discussed. Figure 7.14 presents the CDF plots for the 1 to 3 exposure condition comparison, while Figure 7.15 presents the 2 to 3 comparison. When considering the plots it should be noted that the x-axis range in the GGBS CDF plots is twice that utilised in the PFA CDF plots presented above. The 50% probability RM values obtained are presented in Table 7.13, together with the COV values associated with each RM distribution and the experimental RM results from Chapter 5.

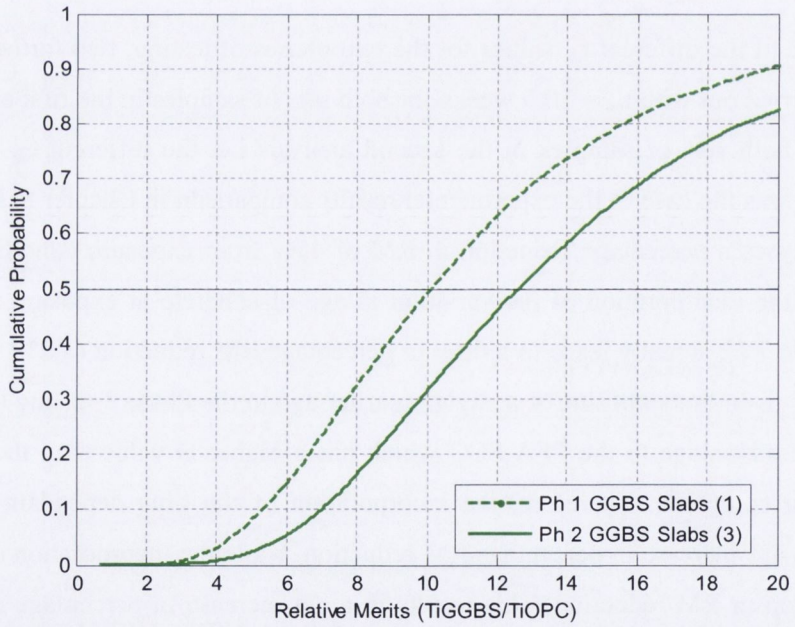


Figure 7.14 CDF of GGBS relative merits values for exposure condition 1 and 3

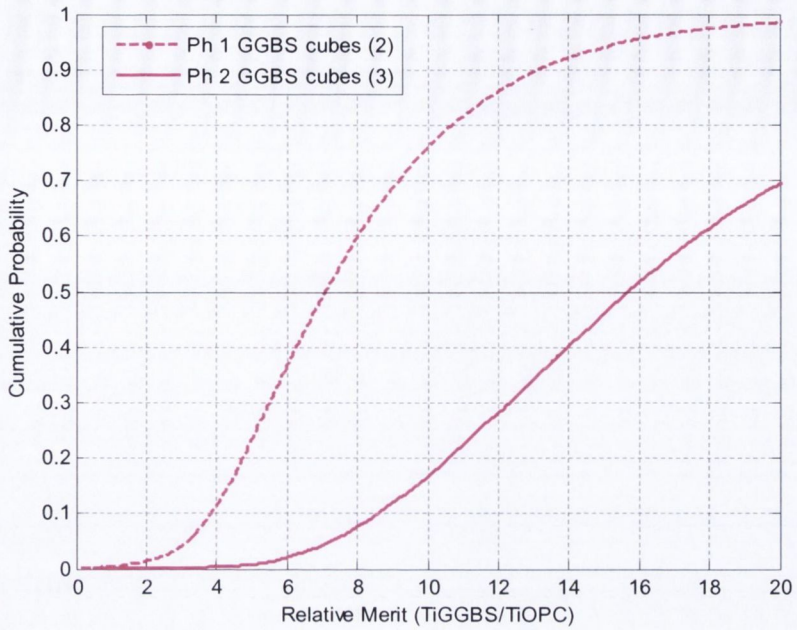


Figure 7.15 CDF of GGBS relative merits values for exposure condition 2 and 3

Table 7.13 Probabilistic and experimental results of PFA relative merits for the two sets of exposure condition comparisons

Exposure Condition	Median RM	Percentage Reduction in RM	RM COV	Experimental RM	Percentage Reduction in RM
3 - Slabs	12.64		0.50	2.81	
1 - Slabs	10.14	20%	0.52	2.27	19 %
3 - Cubes	15.71		0.50	3.30	
2 - Cubes	7.07	55%	0.51	1.80	43 %

As with the PFA SCC, the relative performance of the GGBS SCC obtained from the probabilistic modelling reduces with increasing level of drying in the exposure condition. However, in keeping with the experimental results discussed in Chapter 5, the reduction in performance is greater for the exposure condition 2 to 3 comparison than the 1 to 3 comparison. As discussed in Chapter 5, this is opposite to the expected effect. The same pattern was, however, identified by Ben Fraj et al. (2012) who found the performance of GGBS to reduce by a greater amount for exposure condition 2 to 3 than 1 to 3 (using the same terminology as used herein).

Examining the comparison of 1 year experimental results and the probabilistic modelling assessing performance over service life timeframes, the RM reduction from exposure condition 1 to 3 can be seen to be almost identical for both assessment methods. Investigating the role of the time

dependant diffusion and the t_{ex} effects in the probabilistic analysis it was found that the t_{ex} influence lead to a decrease in the RM reduction magnitude of 7%. Hence the time dependant effects contributed 8% to the magnitude of the RM reduction from exposure condition 1 to 3. For the 3 to 2 exposure condition comparison it is noted that the RM reduction in the probabilistic analysis is 12% greater than that identified in the experimental results analysis. This is due to the time dependant effects contributing 16% to the RM reduction, while incorporating the t_{ex} effect decreased the magnitude of the reduction by just 4%.

Overall, the analysis above has examined the effect of exposure condition on RM values which were calculated based on probabilistic service life predictions for OPC SCC, GGBS SCC and PFA SCC. It was found that both the PFA SCC and GGBS SCC performance reduced with increased levels of drying for the two sets of exposure condition comparisons presented. The COV value for the analyses was substantial at approximately 0.5. This is due to the uncertainty associated with the chloride migration and reinforcement corrosion process over service life timeframes. The experimental results analysis and the probabilistic analysis showed similar RM percentage reductions with change in exposure condition, despite the fact that the probabilistic analysis incorporated both time dependant effects and age of concrete upon exposure. Further investigation into the roles of these two effects revealed that both the time dependant effects and the age of concrete upon exposure did, in some instances, the RM reduction, depending on the concrete type and exposure conditions being analysed. In general, however, the analysis found the influence of the two factors to be similar in magnitude, but opposite in effect on the RM reduction due to change in exposure condition. This resulted in the net change in the RM reduction being similar for the 1 year experimental results and the probabilistic analysis for service life timeframes.

7.5 SERVICE LIFE PREDICTIONS

The first two sections of this chapter were concerned with addressing Research Question 3, while the last section further investigated issues relating to Research Question 2. Having explored these research areas this section simply presents the prediction of the T_i values for the Ferrycarrig Bridge as a case study. This case study facilitates the first examination of the OPC with increased cover SCC option, which was used on Ferrycarrig Bridge crosshead No. 3. The performance OPC with Silane SCC option will also be examined in in a probabilistic framework in this section for the first time, although the examination of the OPC + silane SCC option is purely indicative, as will be discussed later in this section. It is noted that the analysis herein does not consider the effects of cracking due to structural loads or shrinkage on the ingress of chlorides. It is likely that these

effects would reduce the predicted service life. The crosshead beams considered in this section will be as follows:

- Crosshead 1 and Crosshead 7: OPC
- Crosshead 2: OPC: 70mm Cover
- Crosshead 3: OPC + Silane Treatment
- Crosshead 4 and Crosshead 6: OPC + 60% GGBS

No service life predictions will be made for Crosshead 5 which was constructed using OPC with a mixed in corrosion inhibitor. This is because the OPC with corrosion inhibitor durability option was only examined herein from a chloride ingress resistance point of view. As discussed in Chapter 4, the nature of this testing did not facilitate exploration into the primary means by which the corrosion inhibitor delays the onset of corrosion.

The results of two probabilistic analyses will be discussed in this section. First, the results of an analysis which uses the calibration factors employed in the previous section will be presented. Therefore, each crosshead beam will be compared for an equal level of exposure. This will allow the predicted long term performance of the OPC with increased cover option and the OPC with silane surface treatment option to be compared to the OPC and OPC + GGBS predicted long term performance for the first time. The second analysis will utilise the site investigation data analysed by Kenshel (2009) to its full capacity by implementing location specific calibration factors for the crossheads to make more accurate service life predictions. Given the need to calibrate the accelerated results, established in Section 7.2, the Phase 2 results will primarily be used for the Ferrycarrig Bridge service life predictions.

The CDF plot below presents the predicted T_i values for six of the seven crosshead beams. The summary statistics for the probabilistic analysis are presented in Table 7.14.

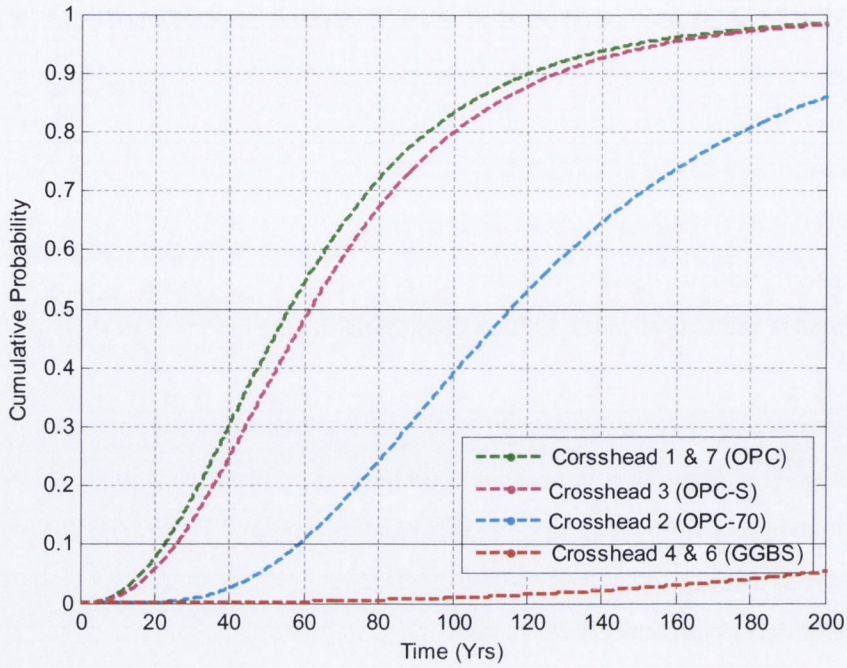


Figure 7.16 CDF plot of uncalibrated Ti distributions for PFA VC and OPC SCC

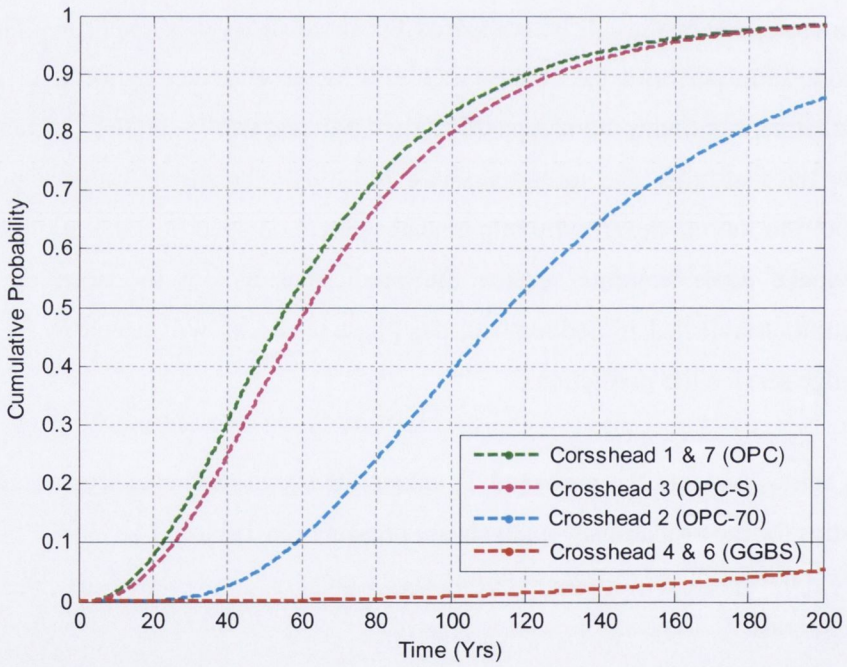


Figure 7.17 CDF plot of uncalibrated Ti distributions for PFA VC and OPC SCC

Table 7.14: Ferrycarrig Bridge crosshead beams predicted service life (uniform exposure)

Crosshead Beam	50% Probability	10% Probability	COV	R.M P(0.50)	R.M (0.10)
Crosshead 1& 7 (OPC)	55.7	22.5	0.68	1.00	1.00
Crosshead 2 (OPC 70mm)	115.7	58.5	0.56	2.08	2.60
Crosshead 3 (OPC + Silane)	61.2	25.4	0.64	1.10	1.13
Crosshead 4& 6 (GGBS)	697.4	262.9	0.83	12.53	11.70

It is clear from Figure 7.17 that the Crossheads 4 and 6, constructed of OPC + 60% GGBS have by far the greatest predicted service life when considering both the 50% probability of corrosion and the service life as defined by the FIB Model Code. The predicted T_i values for the OPC SCC and GGBS SCC have, however, been discussed in detail in the Section 7.2. As mentioned, probabilistic output for OPC with increased cover and the OPC with silane surface treatment have not been discussed up to this point. Firstly, examining the OPC with increased cover, it is noted that the additional 20mm cover doubles the predicted T_i value from 55.7 years to 115.7 years. This results in a relative merit of 2.08. The percentage gain at the 10% probability of corrosion initiation is even greater with a RM of 2.6 obtained from the analysis. This increase in relative merit at the lower probability of corrosion is due to the relative magnitudes of the OPC–70 total cover and the rate of ingress. In the early stages of the progression of the chloride front, when the front is still at a comparatively shallow depth, the cover depth has a high influence over the probability of corrosion initiation. This leads to a high relative performance for the OPC-70 option at low probabilities of T_i. However, as the chloride front moves further into the concrete the probability of corrosion initiation becomes less influenced by the cover depth and more influenced by the other variables in Equation 6.8. This causes the relative merit of the OPC-70 to reduce with increasing probability of T_i. As highlighted by Ryan and O'Connor (2013). Thus, the advantage obtained from increasing cover is likely to be exaggerated when this concrete durability option is considered in the context of the FIB model code service life limit state of 10% probability of corrosion initiation.

Examining the Silane performance, it is noted from Figure 7.17 and Table 7.14 that this option results in only a marginal increase in performance over the plain OPC SCC utilised in Crosshead 1 and 7. This poor silane performance can be attributed to the moisture condition of the concrete sub-strata upon application of the silane surface treatment in the Phase 2 testing. As discussed in Chapter 5 the failure to allow the prescribed drying time before silane application in Phase 2 means that while the Phase 2 silane result may reflect the performance of silane in the tidal zone or lower splash zones, the result is not reflective of the OPC + silane performance in accordance with the manufacturers guidelines.

To facilitate some assessment of the silane in accordance with the manufacturer's guidelines the Phase 1 cube results will also be presented. Comparison of the Phase 1 OPC-S results from the two different chambers indicated that the silane performance did not change with changes in environmental exposure conditions. It is, however, recognised that the service life prediction using the Phase 1 results is highly approximate. It is merely presented as an indication of the OPC + silane performance when the surface is applied in accordance with manufacturer's recommendations. In fact, as noted in the Section 7.2 the service life predictions for OPC + silane in general are problematic for a number of reasons. Firstly, the silane hydrophobe only penetrates a few millimetres below the concrete surface. Consequently, the effectiveness of the silane treatment will reduce with time due to the effect of weathering and abrasion in aggressive environments. As discussed, there is a lack of knowledge surrounding the service life predictions of surface treated concretes (Stewart et al. 2012). In previous probabilistic service life assessments Ferreria (2008) assumed silane coated concrete to be impermeable at 0 years and to have the same properties of OPC at 7 years. Stewart et al. (2012) assumed silane treatments to be maintained throughout the structures service life. Given the already approximate nature of the silane surface treatment, and the lack of a detailed understanding of the length of the silane surface treatment's effective period, this assumption will also be made herein. Secondly, as seen from the Phase 1 experimental results, the application of the silane surface treatment resulted in significant reductions in the surface chloride content. The calibration of the silane results assumes the reduction in C_s, resulting from silane treatment, to be linear across the salt fog chamber and Ferrycarrig Bridge exposure conditions. This is unlikely to be a valid assumption. Given these assumptions, and the issues with the Phase 2 silane application, service life predictions relating to the OPC-S durability options are only indicative of possible performance. The analysis of the chloride profiles from Ferrycarrig Bridge Crosshead No. 3 will in time facilitate more realistic assessment of the effectiveness of silane for OPC SCC over time.

Bearing these issues in mind, the service life predictions presented in Figure 7.17 are presented again with the additional prediction for OPC-S, which is based on the Phase 1 cube sample results. The Phase 1 results were incorporated into the analysis by multiplying the Phase 2 OPC SCC C_s and D_{app} values by the ratio of the corresponding Phase 1 OPC values to the Phase 1 OPC-S values (OPC/OPC-S). The resultant C_s and D_{app} values were then calibrated. Thus, Figure 7.18 below presents two service life predictions for OPC-S, one based on the Phase 1 cube results and the second based on the Phase 2 cube results.

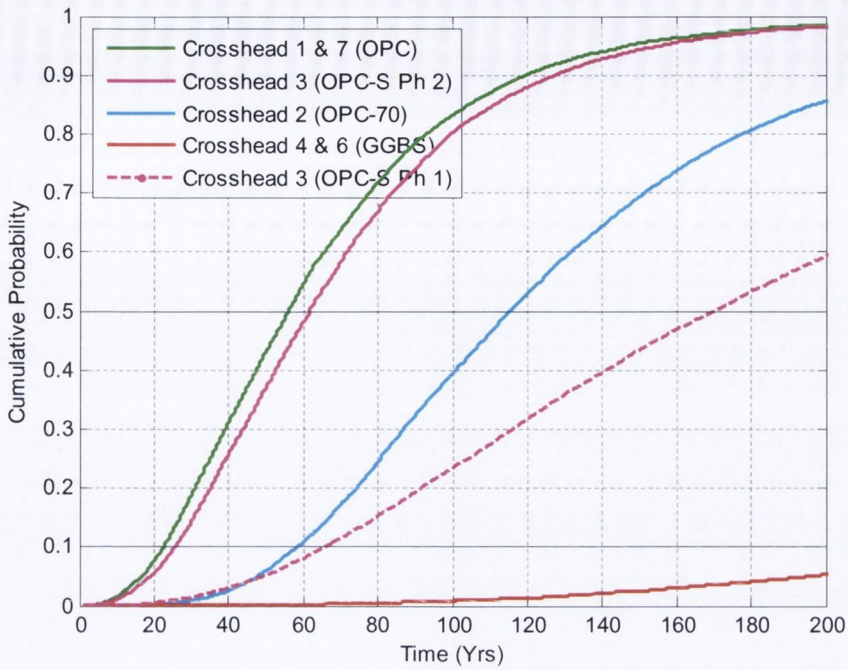


Figure 7.18 T_i predictions for Ferrycarrig crosshead beams with Phase 1 and Phase 2 silane results

Table 7.15: Ferrycarrig Bridge crosshead beams predicted service life with two OPC-S predictions (uniform exposure)

Crosshead	50% Probability	10% Probability	COV	R.M P(0.50)	R.M (0.10)
Crosshead 1 & 7 (OPC)	55.7	22.4	0.67	1.00	1.00
Crosshead 2 (OPC 70mm)	115.6	58.5	0.56	2.08	2.61
Crosshead 3 (OPC –S Phase 2)	61.2	25.3	0.64	1.10	1.13
Crosshead 4 & 6 (GGBS)	698.4	263.3	0.83	12.54	11.75
Estimate: Crosshead 3 (OPC –S Phase 1)	168.5	65.9	1.18	3.03	2.94

As can be seen from Figure 7.18 and Table 7.15 the duration of the OPC-S predicted service life has greatly improved when the Phase 1 results are used in the analysis. The predicted T_i value is 168.5 years while the predicted service life in accordance with the FIB Model Code is 66.0 years. It can be noted that the RM of the OPC-S has also increased to 3.0 compared to the RM value of 1.8 obtained from the experimental analysis. From Table 7.15 it is also noted that the COV value for the OPC-S Phase 1 prediction is far higher than the other Crosshead predictions. This is due to the fact that the silane treatment has resulted in a drop in the C_s value to levels which in some probabilistic runs are very close to the C_r value. When this occurs in the probabilistic analysis a very high value for predicted T_i is observed. This causes an increase in the T_i PDF distribution COV although the 50% probability of T_i value is only slightly effected. It is again re-iterated that

the Crosshead 3 (OPC-S Phase 1) prediction is an approximate prediction to be used as an indicator of performance only due to the previously discussed inherent assumptions relating to this service life prediction. Comparison of the two Crosshead 3 predictions does however give an idea of the potential influence of the moisture condition of the concrete sub-strata on service life of a structural element.

Having examined the service life predictions using the global calibration factors, with particular attention paid to the OPC-70 SCC option and the OPC-S SCC option, the location specific analysis will now be presented. The plot below, developed by Kenshel (2009), shows the change in exposure severity for each of the Ferrycarrig Bridge crosshead beams. As can be seen from the plot the North Face of the crosshead beams was found to show a higher level of exposure. Given that this analysis is concerned with the onset of corrosion in each crosshead beam, the North Face results were utilised in the analysis presented below. It can also be noted from the plots that no north face data was collected for crossheads beams No. 1 (OPC), No. 5 (OPC + Corrosion Inhibitor) and No. 6 (GGBS). Consequently, these crossheads were omitted from the location specific service life predictions, however, this still facilitated assessment of one of each of an OPC, OPC-S, OPC + GGBS and OPC-70 crosshead.

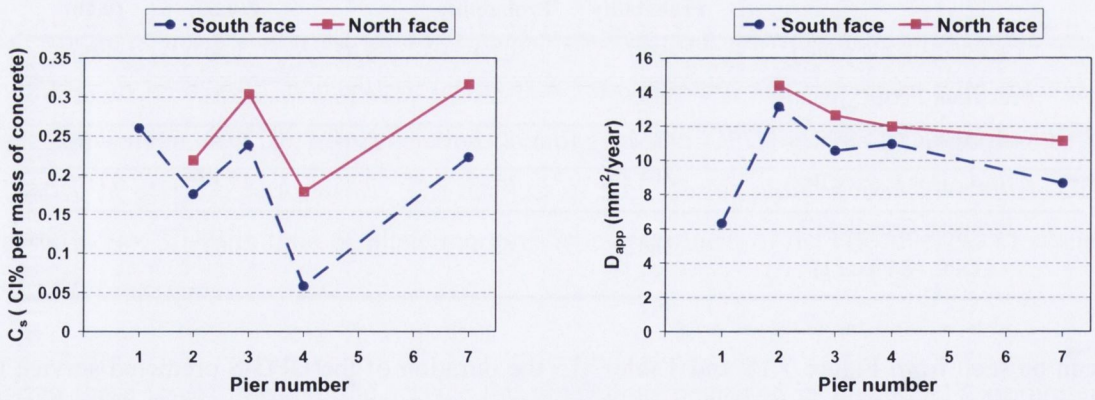


Figure 7.19. Location specific average transport properties (a) C_s and (b) D_{app} (Kenshel 2009)

When considering the calibration factors one must also consider what the probabilistic assessment is seeking to examine. Using the average C_s and D_{app} values from each crosshead will calculate the probability of corrosion at any given point on the north face of the crosshead. However, what is of concern herein is the time to corrosion initiation in each of the crosshead beams. To assess the time to first corrosion one must consider the most severely exposed points on the crosshead beam. Thus, the calibration values for the location specific probabilistic assessment were developed based on

the two most severe exposure points on the North face of each crosshead beam. These calibration values which indicate the severity of exposure for each crosshead beam are presented below in Table 7.16 i.e. higher calibration factors equate to more extreme exposure.

Table 7.16: Calibration values for each crosshead beam

Crosshead Beam	C_s Calibration Factor	D_r Calibration Factor
Crosshead beam 2 (OPC- 70)	0.23	0.53
Crosshead Beam 3 (OPC-S)	0.40	0.38
Crosshead Beam 4 (GGBS)	0.24	0.35
Crosshead Beam 7 (OPC)	0.39	0.34

The T_i distribution CDF plot for this analysis, which accounts for both material properties and location specific exposure, is presented below in Figure 7.20. The T_i values relating to the 50% and 10% probability of corrosion, together with the COV values for each crosshead are presented in Table 7.17.

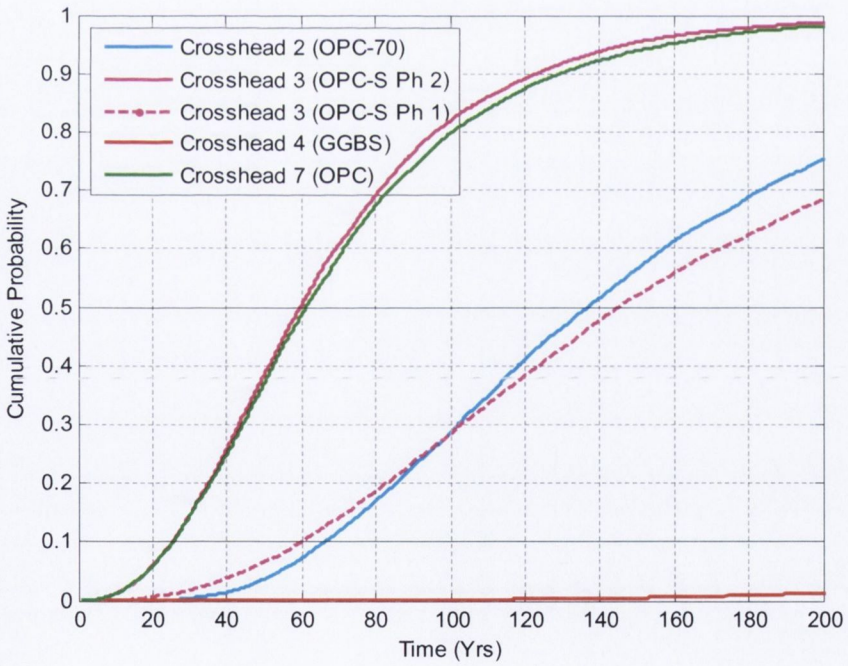


Figure 7.20 T_i predictions with Phase 1 and Phase 2 silane results

Table 7.17: Results of location specific probabilistic analysis

Crosshead Beam	50% Probability	10% Probability	COV
Crosshead 2 (OPC 70)	136.5	66.5	0.67
Crosshead 3 (OPC-S Ph 2)	59.1	24.9	0.62
Crosshead 4 (GGBS)	1208.2 (no corrosion)	446.1 (no corrosion)	0.91
Crosshead 7 (OPC)	60.8	25.2	0.65
Crosshead 3 (OPC-S Ph 1)	144.9	59.4	0.85

As can be seen from the CDF plot and Table 7.17 the probabilistic analysis indicated that corrosion will not initiate in the north face of crosshead beam No. 4 over the structures service life. The finding is a combination of the fact that a) crosshead 4 was constructed from the GGBS SCC option, and b) the crosshead is subject to the least severe exposure condition of the 4 crosshead examined in the site investigation in 2007. Kenshel (2009) stated that this reduced exposure severity at crosshead 4 was caused by chloride washout resulting from the bridge expansion joint directly above the crosshead 4. As no information is available regarding the exposure severity at crosshead no. 6 a location specific analysis was not possible away from the expansion joint. However, the results presented in Table 7.15 give a good approximation of the service life predictions for crosshead 6. It should be noted in the context of this finding that the analysis above is based on exposure condition 3 results. As shown in Chapter 5 and again in Section 7.4, the rate of ingress in the GGBS may increase depending on the level of wetting and drying experienced at the crosshead beam location.

Examining the other crosshead beam results it can be noted that the Crosshead 3 assessment, based on the Phase 1 results, shows similar service life predictions to Crosshead 2 (OPC-70). It is noted that this was not the case in the previous analysis which did not account for location specific exposure. The location specific analysis has resulted in predicted T_i values of 144.9 and 136.5 for OPC-S (Phase 1) and OPC-70mm, respectively. The service life predictions in accordance with FIB Model Code are also close at 59.4 and 66.5, respectively. However, it is re-iterated that that this prediction for OPC with silane is to be treated with caution.

Examining the CDF plot for crosshead 3, based on the Phase 2 results, and crosshead 7 (OPC) it can be seen that any advantage from applying the silane surface treatment to a wet substrata has been eradicated by the slightly more severe exposure conditions at crosshead 3. These two CDF plots curves show the shortest service life predictions with similar predicted T_i values of

approximately 60 years and similar FIB model code service life predictions of approximately 25 years.

In summary then, this case study has facilitated examination of the relative performance of the increased cover option over service life timeframes. The relative merit of this option ranged from 2.1 to 2.6, with the relative merit reducing with increasing probability of T_i. The findings of the case study relating to the performance of the OPC-S SCC must be treated with caution due to the large number of unknown factors affecting the long term performance of silane surface treatment. These shortcoming in existing knowledge, such as the lack of understanding of the effective life of silane treatments, means that the OPC-S SCC predicted long term performance is purely indicative and cannot be considered a robust estimate. For the location specific analysis the Ferrycarrig Bridge data was used to full effect with prediction of T_i for Ferrycarrig Bridge made taking into account both the material properties and the specific exposure condition at each crosshead. It was found that the predicted times to initiation of corrosion at Ferrycarrig bridge can be expected to be in the region of 60 years for OPC, 140 years for OPC with increased cover and finally the analysis suggests that corrosion will not occur in the GGBS crosshead 4 over the service life of the structure.

7.6 CONCLUSIONS

The primary focus of this chapter was on addressing Research Question 3 which was developed in the literature review as follows;

Research Question 3: How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

This research question has been addressed in this chapter through probabilistic modelling incorporating the Phase 2 experimental test results in Sections 7.2, 7.3 and 7.5. For both the calibrated and uncalibrated analysis it was found that there was a notable increase in relative merits of the SCCs containing SCMs, when compared to the relative merits obtained from experimental testing. This was primarily due to the effect of time dependant diffusion coefficient. The GGBS SCC option experienced the greatest increase in relative performance from experimental testing to probabilistic modelling. The relative merit for this SCC option increased from a value of 3.25,

obtained experimentally, to a value of 12.5 for the calibrated probabilistic analysis. The predicted long term calibrated performance of the two OPC + PFA SCCs, relative to the OPC SCC, approximately doubled when compared to the from the 1 years experimental results. This information is significant when it is considered that existing information about the marine durability of different SCCs is solely based on short term investigations, with no long term data available to date, and no published studies utilising probabilistic modelling techniques to compare SCCs over long exposure. It was also found, however, that the COV values associated with the predictions herein were quite high. Thus, there is considerable uncertainty associated with the predicted long term performances of different concrete options. This further highlights the need to utilise a probabilistic approach, which allows uncertainty to be quantified and considered in the context of results. A sensitivity analysis was carried out to highlight how level of uncertainty could be reduced through further research. Examination of the time dependent diffusion process was identified as the most powerful means of reducing prediction uncertainty.

The probability based predicted long term performance of the SCCs and the VCs were also compared in this chapter. The analysis indicated that the OPC VC is substantially more durable than the equivalent SCC. However, the incorporation of SCMs results in a reduction in the difference between VC and SCC, meaning that given the susceptibility of the VC to workmanship errors the SCC could result in a more durable concrete on site. These findings were in agreement with the patterns identified in Chapter 5 when comparing SCC and VC, although it is noted the magnitude of the differences between the SCC and VC changed slightly for the long term predictions due to the influences of the time dependent effects.

The Ferrycarrig Bridge case study facilitated the comparison of OPC-70 SCC option with the other SCC options. The predicted long term relative performance of this increased cover option ranged from approximately 2.1 at the 50% probability of corrosion initiation, to 2.6 at the 10% probability of corrosion initiation. Thus based on the probabilistic modelling incorporating the Phase 2 results, increasing the cover depth by 20mm substantially increases long term marine durability, however, this measure is not as effective as incorporating SCMs in the SCC mix.

The final part of the Ferrycarrig Bridge case study involved predicting the likely time to initiation of corrosion in the Ferrycarrig Bridge crossheads based on the material properties of the crossheads beams and the extent of chloride exposure at each crosshead location. This analysis found that the predicted times to initiation of corrosion at Ferrycarrig bridge can be expected to be in the region of 60 years for OPC crosshead, 140 years for OPC with increased cover corsshead. The analysis

suggests that corrosion will not occur in the GGBS crosshead 4 over the service life of the structure.

Finally, it is noted that the probabilistic model was also utilised in this chapter to investigate if the effect of exposure condition on RM values identified in Chapter 5 still held firm over service life durations. This work constituted further investigation into Research Question 2. It was found that, although the effect of exposure condition was influenced by both time of concrete upon exposure and the time dependent effects, the influence of these factors were similar in magnitude but opposite in effect, meaning the net change in the RM reduction was similar for the 1 year experimental results and the probabilistic analysis for service life timeframes.

8. Conclusions and Future Work

8.1 INTRODUCTION

The literature review in Chapter 3 revealed a number of important shortcoming in the existing literature which deals the initiation phase of chloride induced reinforcement corrosion. Based on these shortcomings the following three core PhD research questions were developed:

Research Question 1: How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

Research Question 2: Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

Research Question 3: How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

The preceding chapters have presented the means by which this PhD work has addressed each research question. This chapter presents the conclusions of this PhD under these three core research questions. Subsequently, the recommendations for future work are presented.

8.2 RESEARCH QUESTION 1

How do different SCCs perform in the initiation phase of chloride induced corrosion when examining chloride transport properties under natural chloride migration conditions?

There are two parts to this research question; a) how do different SCC options compare to one another, and b) how does the performance of SCCs compare to the performance of traditional vibrated concretes? Both parts were addressed in this PhD through examining chloride transport properties obtained from natural chloride migration testing in salt fog chambers, a test apparatus which incorporates wetting and drying cycles.

Firstly, considering part a) of Research Question 1, five different SCCs were examined, each of which complied with the SCC workability requirements specified in EN 206-9:2010 (Centre Europeen de Normalisation CEN 2010). The SCCs options considered are listed below;

- OPC SCC
- OPC + PFA SCC
- OPC + PFA (k) SCC
- OPC + GGBS SCC
- OPC SCC + Silane surface treatment

The OPC + GGBS SCC was identified as the most durable SCC option across the three sets of exposure conditions examined, with a chloride resistance performance ranging from 1.9 to 3.3 times better than the OPC SCC, depending on the exposure condition and sample type examined.

The OPC + PFA SCC chloride resistance performance was found to decrease with increasing levels of drying during the chloride exposure period, as will be discussed in detail in Section 8.3. Consequently, the relative merit of the OPC + PFA SCC ranged between 1.0 for the driest exposure condition, and 1.8 for the exposure conditions with the least drying. This means that for the driest exposure condition the performance of OPC + PFA SCC was approximately equal to the performance of the OPC SCC, while for the wettest condition the OPC + PFA SCC chloride resistance was twice that of the OPC SCC.

PFA as a cement replacement with the k-value efficiency factor from EN 206-1:2000 was also investigated via the OPC + PFA(k) SCC mix, which was exclusively examined in the Phase 2 testing (least drying). It was found that the implementation of the *k*-value efficiency factor increased the relative performance of SCC containing PFA from a RM value of 1.8 to 2.6 i.e. 2.6 times more chloride resistance than the OPC SCC. It is noted however that the mix design modifications necessary to satisfy EN 206-1:2000, resulted in a notable reduction in SCC workability, with the OPC + PFA(k) SCC failing the J-ring passing ability test (Centre Europeen de Normalisation CEN 2010).

The Phase 1 testing indicated that the application of silane surface treatment to OPC SCC could notably increase the chloride resistance capacity of the concrete. The OPC + silane SCC option had a relative merit value of 1.8 when compared to the standard OPC SCC option. However, the variation associated with the Phase 1 OPC + silane performance from sample to sample was considerable. Comparison of the OPC + silane results from the literature also revealed considerable

variation in documented silane performance. The three sets of tests detailed in this thesis highlighted the extent of silane performance variation due to a) the manner of application of silane and b) the moisture condition of the concrete sub-strata upon silane application. These two issues, identified in controlled experiments, are likely to cause high variability of silane performance on site. This casts doubt on the appropriateness of this SCC enhance durability option.

The second part of Research Question 1 relates to the established need to compare SCC and VCs. This is essential in the consideration of the performance of SCC, as it compares this new concrete technology to the traditional concretes in a form of benchmarking process. Again these concretes were compared using natural chloride migration testing which incorporated wetting and drying cycles. It was noted that the vibrated concretes in this study were subject to ideal uniform compaction, unlikely to occur on site. The SCC vs. VC comparison was carried out for the following three concrete types:

- OPC SCC Vs. OPC VC
- OPC + PFA(k) SCC Vs. OPC + PFA(k) VC
- OPC + GGBS SCC Vs. OPC + GGBS VC

The experimental investigation revealed that, when comparing the OPC SCC to the equivalent OPC VC, the vibrated concrete had a chloride resistance performance 1.7 times greater than the SCC. This is an interesting finding in light of the fact that a number of researchers in the literature have stated that the use of SCC, instead of vibrated concretes, leads to an increase in concrete durability due to the elimination of issues relating to workmanship and improper concrete vibration (Kasemchaisiri and Tangtermsiriku 2008; Okamura and Ouchi 2003; Sahmaran et al. 2008). However, the relative performance values for the OPC concretes indicate that poor compaction and workmanship errors in the vibrated concrete would have to reduce durability by 42% before the performance of the OPC SCC and OPC vibrated concrete would be equal.

The performance of the vibrated concretes relative to the SCC was reduced for the concretes containing SCMs. The OPC + PFA(k) vibrated concrete performed 1.32 times better than the equivalent SCC, while the OPC + GGBS VC and SCC performances were very similar, with the vibrated concrete performing 16% better than the SCC.

8.3 RESEARCH QUESTION 2

Does the extent of wetting and drying in a chloride exposure condition effect the relative performance of different SCCs?

In order to address Research Question 2 in this PhD the performance of OPC + PFA SCC and OPC + GGBS SCC relative to OPC SCC, was examined across three different exposure conditions.

The experimental investigation revealed that, when compared to OPC SCC, the relative performance of both the GGBS SCC option and the PFA SCC option reduced with increased levels of drying in the chloride exposure environment. Absorption tests and drying tests indicated that this was due to a smaller difference between concretes containing SCMs and OPC concretes when considering absorption and drying, than when considering pure chloride diffusion in saturated conditions. This is a particularly interesting finding in light of the fact that the majority of accelerated testing published in the literature to date is carried out in fully submerged conditions, while the critical chloride induced corrosion zones in real structures occur in areas which are subject to cyclic wetting and drying.

Overall in terms of performance the results indicated that while OPC + GGBS concretes in particular are likely to be more durable than OPC concretes regardless of exposure conditions, it is important to investigate the scale of reduction in relative performance of these durable concrete options under various exposure conditions. The comparison of OPC and OPC + PFA SCCs however indicated a more critical situation, with the OPC + PFA performance found to be poorer than OPC in the Phase 1 SKWT 4300 testing (most drying), and 80% better than OPC in the Phase 2 SKWT4300 testing (least drying).

The effect of exposure conditions on predicted relative performance of OPC + PFA SCC and OPC + GGBS SCC was also examined in the probabilistic framework to investigate if the patterns identified in the 1 year duration experimental tests still held firm when considering long term exposure. It was found that the relative merit reduction for the different exposure conditions was similar for the 1 year experimental results and the probabilistic analysis for service life timeframes.

8.4 RESEARCH QUESTION 3

How do different concretes, particularly SCCs for which no long term marine durability data exists to date, perform when examined over service life durations using probabilistic modelling techniques? This issue should be examined with and without calibration of chloride transport properties.

Research Question 3 was addressed in this PhD through probabilistic modelling incorporating the natural chloride migration tests results obtained from the Phase 1 and Phase 2 experimental programmes. The analysis indicated that when considering timeframes applicable to service life there is a substantial increase in the relative performance of the SCCs containing SCMs when compared to the 1 year experimental testing. This improved performance of SCCs containing SCM, when compared to OPC SCC, was due primarily to the increased influence of the time dependant diffusion coefficient over greater exposure periods. The calibration of the experimental results with the data from Ferrycarrig Bridge was not found to have a notable effect of the relative performance of the SCCs containing SCMs, with the exception of the OPC + GGBS SCC relative merit, which reduced by approximately 20% due to calibration. The calibration did however result in large increases in the predicted T_i values for all the SCCs. This is to be expected when it is considered that the calibration represents a change in exposure condition from salt fog chamber conditions to Ferrycarrig Bridge conditions.

When considering the change in relative merit from 1 year experimental testing to probabilistic modelling, the OPC + GGBS SCC option experienced the greatest increase in relative performance. The relative merit for this SCC option increased from a value of 3.25, obtained experimentally, to a value of 12.5 for the calibrated probabilistic analysis. The predicted long term calibrated performance of the two OPC + PFA SCCs, relative to the OPC SCC, approximately doubled when compared to the from the 1 years experimental results. This information is significant when it is considered that existing information about the marine durability of different SCCs is solely based on short term investigations, with no long term data available to date, and no published studies utilising probabilistic modelling techniques to compare SCCs over long exposure.

The COV values associated with the probabilistic predictions were quite large, highlighting the considerable uncertainty associated with the predicted long term performances of the different concrete options. This emphasises the need to utilise a probabilistic approach, which allows uncertainty to be quantified and considered in the context of results. A sensitivity analysis was carried out to ascertain how the level of uncertainty could be reduced through further research.

Examination of the time dependent diffusion process was identified as the most powerful means of reducing uncertainty associated with long term performance.

The predicted long term relative performance of the SCCs and the VCs were also compared using the probabilistic model. It was found that the findings of the Phase 2 experimental comparison remained unchanged when comparing these two different forms of concrete over timeframes applicable to service life.

A probabilistic marine durability assessment of Ferrycarrig Bridge as a case study facilitated the comparison of OPC-70 SCC option with the other SCC options. The predicted long term relative performance of this increased cover option ranged from approximately 2.1 at the 50% probability of corrosion initiation to 2.6 at the 10% probability of corrosion initiation. Thus based on the probabilistic modelling incorporating the Phase 2 results, increasing the cover depth by 20mm does substantially increase long term marine durability, however, this measure is not as effective as incorporating SCMs in the SCC mix.

Finally, the Ferrycarrig Bridge case study allowed probability based predictions to be made as to the likely time to initiation of corrosion in the Ferrycarrig Bridge crosshead beams based on the material properties of the crossheads and the extent of chloride exposure at each crosshead. This analysis found that the predicted times to initiation of corrosion at Ferrycarrig bridge is expected to be in the region of 60 years for OPC and 140 years for OPC with increased cover. The analysis suggests that corrosion will not occur in the GGBS crosshead 4 over the service life of the structure. It should be noted, however, that the COV values associated with these long term predictions were high, ranging from 0.67 to 0.91.

8.5 OVERALL RECOMMENDATION

OPC + GGBS displayed the highest marine durability of the SCCs examined herein, which include OPC, OPC + silane surface treatment, OPC with increased cover and OPC + PFA with and without the *k*-value efficiency factor. Evidence indicates the use of OPC + GGBS SCC, instead of an equivalent OPC SCC, will lead to an increase in durability by a factor of 2 in the short term, and by a factor as high as 12-15 over the service life of a marine structure. The predicted long term performance of OPC + GGBS also exceeds the next best SCC marine durability option, OPC + PFA with *k*-value applied, by a factor of approximately 2. The analysis also suggests that the use of

an OPC + GGBS SCC is likely to be more durable option than OPC + GGBS VC, when the role of workmanship is considered.

8.6 FUTURE WORK

The work carried out in this thesis has highlighted a number of areas where further research is recommended as follows:

1. Research into the time dependant nature of the chloride diffusion coefficient for different concrete types is required to accurately assess both the *m* value and the t_c parameter. This is a substantial undertaking which would require a large investment of time and finances. However, as indicated by the sensitivity analysis in Chapter 7, enhanced insight into these parameters would lead to the greatest reduction in uncertainty surrounding service life predictions for structures subject to chloride ingress.

2. In the future the work in this thesis should be built upon by utilising data from Ferrycarrig Bridge. The nature of the Ferrycarrig Bridge repair should allow investigation into a) time dependant diffusion for GGBS and OPC SCC, b) publication of the first real marine bridge data for different SCCs, c) further investigation into calibration factors for accelerated testing and d) incorporation of data from embedded probes and site investigations into the probabilistic model developed herein using techniques such as Bayesian updating.

3. Bases on the findings of both Research Question 1 (b) and Research Question 2 in this PhD SCCs and VCs should be compared under different chloride exposures conditions. The OPC vs. VC comparison herein indicated that the gap between SCC performance and VC reduced from OPC to OPC + PFA concretes and from OPC + PFA to OPC + GGBS concretes. This was thought likely to be due to fact that a) the cement paste occupies a greater proportion of the overall volume in the SCC than in the VC, and b) the chloride resistance of a given volume of cement paste increases from OPC to OPC + PFA to OPC + GGBS. However, the findings relating to Research Question 3 would suggest that the performance of the OPC + GGBS and OPC + PFA cement pastes, relative to OPC cement paste, would reduce for dryer exposure conditions. Thus, the performance of OPC + GGBS and OPC + PFA SCCs and equivalent VCs needs to be explored across exposure conditions which incorporate different levels of drying.

4. It would be very interesting to examine the effect of compaction on the relative performance of SCC and VCs. This could be done by comparing SCC to VC with varying degrees of compaction in VC specimens, from little or no compaction, to over compaction.

5. The effects of shrinkage cracking and structural cracking on chloride ion ingress were not considered in this thesis. Given the high fines content of SCCs, shrinkage cracking may increase the rate of chloride transport into SCC to a greater extent than in VC. The effect of shrinkage cracking on the marine durability of different SCCs for given levels of restraint needs to be investigated.

6. Finally, the probabilistic service life model developed herein could be improved upon in the future by incorporating the effects of a) cracking, b) two-dimensional chloride ingress for corner sections of structural elements, and d) the variation in surface chloride content with time. It is, however, noted that reliable, practical models for incorporating these effects do not exist in the literature at this point in time.

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APPENDIX A

A.1: CONCRETE MIXING PROCEDURE

- Weigh all the constituent materials
- Weight quantity of water required and place in clean bucket
- Add plasticiser and super plasticiser to the water. For the OPC-CI option, corrosion inhibitor was also added to the water at this point.
- Mix water and additives with a steel rod for 45 seconds and a constant rate
- Place the sand and coarse aggregate only in the palm mixer
- Mix for 30 seconds
- Add the cement, and SCM if applicable, and mix for 60 seconds
- Add the water to the palm mixer and mix for 1 minute
- Open the palm mixer and scrape the sides of the mixer to ensure all concrete is mixing fully
- Close palm mixer and mix again for 2 minutes
- Leave concrete to stand for 15 minutes before pouring into moulds
- In this intervening time carry out fresh concrete workability tests

A.2: CHLORIDE EXTRACTION PROCEDURE

- Weigh 5g of dried concrete dust sample using electronic scales correct to 2 decimal places
- Mix dust sample with 25ml of cold distilled water and 10ml of concentrated Nitric Acid (69%) in a 250ml conical flask
- Add 25ml of hot distilled water to the conical flask
- Heat the conical flask on a stirrer hot plate in the conical glass with while covered with a laboratory clock glass
- Once boiling point is reached boil for four minutes stirring throughout with magnetic stirrers
- Preform vacuum filtration on the heated sample using a 250 Flask Bucher, hardened ashless chloride free filter paper and a vacuum pump
- When the sample has been fully filtered rinse the filtrate a minimum of 3 times with the vacuum pump still switched on
- Pour the filtered solution into a 150mm Perspex test beaker
- Rinse the conical flask 3 times adding the rinse solution to the 150ml test beaker

A.3: PROCEDURE TO ENSURE ACCURACY AND PRECISION OF TITRATION ANALYSIS

The accuracy of the titration test system was confirmed by a verification test study of standard samples with known chloride concentrations. Samples in the three following chloride concentration by weight of concrete ranges were tested for OPC, OPC + PFA and OPC + GGBS concrete dust; ≈ 0.040% chloride by weight of concrete, ≈ 0.110% chloride by weight of concrete and ≈ 0.360% chloride by weight of concrete. In total 27 standard sample titrations were performed the initial system verification, with the maximum error obtained equal to 0.004% chloride content by weight of concrete.

To further ensure precision and accuracy in results duplicate dust sample analysis was carried out for each depth increment for the first 10 concrete specimens. After the first 10 specimens the variation associated with the analysis was assessed and found to be minimal. At this point it was decided that the repeatability of the analysis and the operation of the titration device was well established. Preforming duplicate analysis for each sample was not efficient in terms of lab time. Consequently, the accuracy of the analysis for the reminder of the analysis was ensured through the following measures:

- Three duplicate dust samples were tested per specimen
- Three spiked solutions samples were tested each morning which had a known chloride concentration to ensure the titration apparatus was working correctly
- If any unusual results were obtained during sample analysis three spiked solutions were tested and the dust sample in question was re-tested
- Three standard samples were tested once a week to verify accuracy

APPENDIX B

PUBLICATIONS LIST

Journal Publications

Ryan, P. C. & O' Connor A. (2013). "Probabilistic Analysis of the Time to Chloride Induced Corrosion for Different Self-Compacting Concretes." *Construction and Building Materials, 47, 1106-1116.*

Ryan, P. C. & O' Connor A. (In Press). "Examination of Self-Compacting Concrete for Marine Bridge Applications." *ASCE Journal of Bridge Engineering.*

International Conference Publications

Ryan, P. C., A. J. O Connor & J. Kelly, (2012). "Analysis of Five Self Compacting Concretes in the Marine Environment: A Probabilistic Approach." International Conference on Durability of Concrete Structures, Belfast.

Ryan, P. C., A. J. O Connor, (2012). "Probabilistic Modelling of Reinforced Concrete Bridge Repair Deterioration in Marine Environments." Bridge Maintenance, Safety and Management: Proceedings of the Sixth International IABMAS Conference. R. Sause, D. Frangopol and C. Kusko. Philadelphia.

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Salta, M., O'Connor, A., Schoefs, F., Ryan, P.C. (2010) "DuratiNet – Durable Transport Structures in the Atlantic Area". Proceedings of the Transport Research Arena Europe, Brussels.

National Conference Publications

Ryan, P. C., A. J. O Connor, (2010). "The Innovative Repair and Instrumentation of a Marine Bridge and Associated Research." Proceedings of the Bridge and Concrete Research of Ireland. Cork.