International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:5, No:12, 2011

Probabilistic Modelling of Marine Bridge Deterioration

P.C. Ryan and A.J. O' Connor

***Abstract*—**Chloride induced corrosion of steel reinforcement is the main cause of deterioration of reinforced concrete marine structures. This paper investigates the relative performance of alternative repair options with respect to the deterioration of reinforced concrete bridge elements in marine environments. Focus is placed on the initiation phase of reinforcement corrosion. A laboratory study is described which involved exposing concrete samples to accelerated chloride-ion ingress. The study examined the relative efficiencies of two repair methods, namely Ordinary Portland Cement (OPC) concrete and a concrete which utilised Ground Granulated Blastfurnace Cement (GGBS) as a partial cement replacement. The mix designs and materials utilised were identical to those implemented in the repair of a marine bridge on the South East coast of Ireland in 2007. The results of this testing regime serve to inform input variables employed in probabilistic modelling of deterioration for subsequent reliability based analysis to compare the relative performance of the studied repair options.

***Keywords*—**Deterioration, Marine Bridges, Reinforced Concrete, Reliability, Chloride-ion Ingress

List of Abbreviations

C_{crit}	Critical Chloride Content
CoV	Coefficient of Variation
D_{app}	Apparent diffusion coefficient
GGBS	Ground Granulated Blastfurnace Slag
OPC	Ordinary Portland Cement
PDF	Probability Density Function
RC	Reinforced Concrete
TCD	Trinity College Dublin
T_i	Time to initiation of corrosion

I. Introduction

IT is widely accepted that chloride induced corrosion is one of the most serious problems encountered in reinforced concrete (RC) structures [1]-[3]. Researchers have been trying to understand chloride induced corrosion since it was first modelled by Collepardi in 1970 [4]. Since then, the true scale of the problem has become apparent with many of the world's aging structures deteriorating rapidly due to the effects of chloride induced reinforcement corrosion. It is estimated that the annual cost of corrosion effects on bridges in the United States alone is of the order of $8.3 billion [5]. Conventionally, the chloride induced corrosion process was described by Tuutti's [6] model which involved two stages – corrosion initiation phase and crack propagation phase. This model is illustrated below in Fig. 1

P.C. Ryan is a Ph.D candidate in Trinity College Dublin, Ireland. (phone: 00353-87661-2537; e-mail: ryanp7@tcd.ie).
A.J. O'Connor is an Associate Professor in Trinity College Dublin, Ireland. (e-mail: alan.oconnor@tcd.ie).

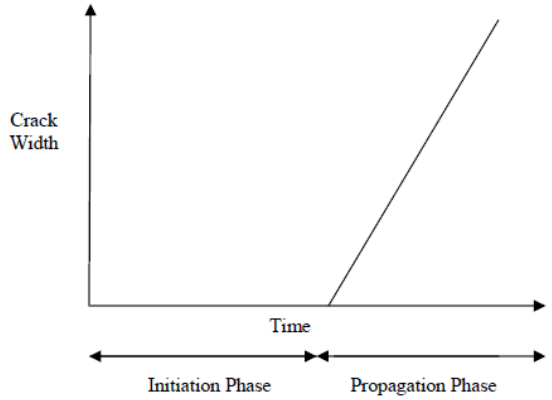

Fig. 1 Tuutti model for chloride induced reinforcement corrosion

More recently however, following work carried out by Weyers et al [7] it is recognised that there are three stages in the chloride induced corrosion process; 1) the corrosion initiation phase, 2) the crack initiation phase and 3) the crack propagation phase. The initiation phase is considered to have commenced when chlorides from the external environment have penetrated to the depth of the reinforcement in a sufficient quantity to break down the passive layer on the surface of the reinforcement. The protective passive layer develops due to the high alkalinity of the concrete surrounding the reinforcement bar after concrete casting. The high alkalinity in the concrete is a result of the presence of highly alkaline sodium, calcium and potassium hydroxide ions. Corrosion cannot take place while the pH remains high around the reinforcement. In a marine environment the levels of hydroxide ions, and subsequent pH levels, are controlled by the presence of aggressive chloride ions [8]. Once the chlorides are in sufficient concentration at the depth of reinforcement the pH level surrounding the concrete is lowered, the passive layer is destroyed and provided there is sufficient supply of oxygen and moisture at the level of the reinforcement, corrosion commences. The concentration of chlorides which brings about the initiation of corrosion is known as the critical chloride content or the threshold chloride level.The crack initiation phase introduced by Weyers et al [7] is the period between the initiation of corrosion and the time to crack development. During this period corrosion products fill the voids and pores surrounding the reinforcement and some migrate away from the concrete steel interface through concrete pores [7]. This phase ends upon development of the 1st chloride induced crack in the concrete. This signals the beginning of the crack propagation phase which spans from the development of cracking to the end of the service life of the structure.

This paper solely explores the initiation phase of reinforcement corrosion. Typically, this phase has a far longer duration than either of the other two phases with the result that

it has the most significant impact on a structure's overall service life. The paper predicts the time to initiation of corrosion through the utilisation of a probability based deterioration model. The model incorporates the results of accelerated chloride ingress testing carried out at Trinity College Dublin (TCD). The testing compares the performance of two repair options which were utilised in the repair of an actual bridge on the South East coast of Ireland in 2007 as will be discussed in Section II. The two repair options examined in this paper are:

1. *CEM 1 Ordinary Portland Cement (OPC) concrete*
2. *CEM 1 OPC with 60% Ground Granulated Blastfurnace Slag (GGBS) as a partial cement replacement.*

The term CEM 1 above relates to the definition of Ordinary Portland Cements in [9] which deals with 27 products in the family of common cements. These common cements are grouped into five main cement types. CEM 1 is one of these five main types of cement and is specified in [9] as 95 - 100% clinker and 0 - 5% minor additional constituents. The GGBS utilised in the experimental program was manufactured in accordance with [10], and is approved for use in the manufacture of concrete and concrete products by [11].

II. Experimental Program

As indicated the basis for the experimental programme was derived from the repair of Ferrycarrig Bridge, a marine bridge on the South East coast of Ireland. In 2007 the bridge underwent substantial repairs. Although the deterioration of Ferrycarrig Bridge itself was not due to reinforcement corrosion, the nature of the bridge repairs afforded the Irish National Roads Authority a unique opportunity to gather information regarding the efficiency of typical alternative concrete repair options in Irish marine environments. Consequently, the cover concrete on the bridge's seven crosshead beams was removed beyond the level of the reinforcement and one of five different repair strategies was employed on each of the bridge's crosshead beams. The repair of Ferrycarrig Bridge is discussed in detail in [12].

The process of chloride-ion ingress and the resultant reinforcement corrosion is very slow. It could therefore be 20 years or more before any real conclusions can be drawn on the relative efficiencies of the repair methods from Ferrycarrig Bridge. Consequently, an experimental program is being carried out at TCD in order to investigate the repair methods utilised at Ferrycarrig Bridge in the short to medium term through accelerated laboratory testing. This allows information to be obtained on typical repair options while they are still widely used. As previously mentioned this paper focuses on two of the repair options utilised on Ferrycarrig Bridge; an OPC repair option and an OPC + GGBS repair option.The experimental testing was carried out in the TCD laboratories using a salt fog chamber. Three 300mm x 300mm x 120mm slab samples were poured for both the OPC and OPC + GGBS mixes. The water/binder ratio for the mixes was 0.44 in accordance with the XS3 exposure class in EN206-1:200 [11]. The OPC + GGBS mix comprised of 60% GGBS and 40% CEM 1 OPC by weight. A 10mm course aggregate was used for both mixes and the proportion of fine to coarse aggregate was 2.3 : 1. A plasticiser and a super-plasticiser were used in the mix. The mix design details for the test samples were identical to the mix design details used on site at Ferrycarrig Bridge. The testing was focused on the initiation phase of corrosion and thus is solely concerned with the ability of the repair options to resist the ingress of chlorides. Consequently, no reinforcement was utilised in the OPC slabs or in the OPC + GGBS slabs. The samples were cured in a curing tank at 20 ± 2ºC for 28 days in accordance with the relevant standard [13]. Compressive strength tests were carried out on 100mm cubes at 7 and 28 days for each of the mixes. The results of these tests can be seen in Table I.

TABLE I
Compressive Strengths of OPC and GGBS Mixes at 7 and 28 Days

Repair Type	7 Day Strength (N/mm^2)	28 Day Strength (N/mm^2)
OPC	57.0	68.9
OPC + GGBS	46.7	67.0

After the curing period the samples were painted with epoxy sealant paint on all but one face in order to rule out edge effects. Upon commencement of the exposure period the concrete test samples were subject to periodic wetting and drying cycles comprising of two days of salt fog and five days of drying per week. During the wetting cycles a 5% sodium chloride solution mist fills the chamber and saturates the surface of the samples. The SKWT4300 walk in fog chamber which was used for the experimental testing is shown in Fig.2.

Fig. 2 KWT4300 Walk in salt fog chamber

The samples were exposed to the aggressive accelerated chloride ingress environment for a period of 8 months. This duration was necessary to ensure penetration of chlorides to a sufficient depth to allow for the fitting of full chloride profiles. Once the 8 month testing had been completed dust samples were collected from each sample at 2.0mm depth increments using a profile grinder. A picture of the profile grinder and one of the concrete slab samples with dust samples extracted can be seen below in Fig. 3.

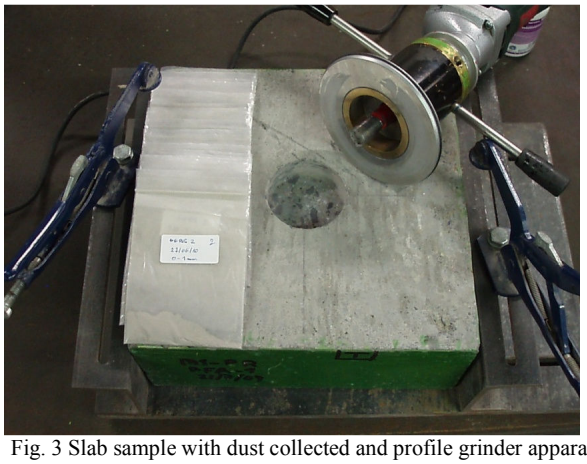

Fig. 3 Slab sample with dust collected and profile grinder apparatus

The dust samples were analysed using acid soluble potentiometric titration to calculate the total chloride content at 2.0mm depth increments. These values were used to create chloride profiles for each sample. An example one of the OPC chloride profiles can be seen in Fig. 4.

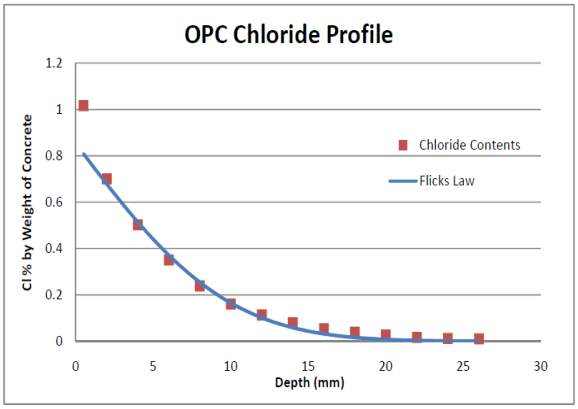

Fig. 4 OPC chloride profile with Fick's law fitted

As is common place in the literature Fick's second law with Crank's solution was fitted to the chloride profiles in order to obtain diffusion coefficients and surface chloride contents for each of the samples [2], [14], [15]. It is these values of surface chloride content, C_s, and apparent diffusion coefficient, D_{app}, which are used in the probabilistic deterioration model to compare the durability of OPC and OPC + GGBS in terms of the statistically determined time to initiation of corrosion.

III. Probability Based Deterioration Model

The theoretical background for the probability based deterioration model utilises an adoption of Fick's second law of diffusion with Crank's error solution:

$$T_i = {x^2}/{4 * D_{app}} * \left[(erf - 1)\left({(C_{cr} - C_s)}/{(C_i - C_s)}\right)^2\right] \quad (1)$$

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. This formula provides an estimate of, T_i, the time to the initiation of corrosion and has been used throughout the literature in predicting the service life of concrete structures [16-18].

It is clear that the parameters in (1) are subject to variation. In order to represent the uncertainty associated with the variables, each variable in (1) was assigned a statistical distribution with a predefined mean and coefficient of variation (CoV). As indicated in Table II the mean and CoV values used for the model parameters are a combination of experimentally calibrated values and values adopted from the literature. Both normal and lognormal distributions were utilised, as suggested in the literature [19-22].

TABLE II
Statistical Parameters For Model Variables

Property	Units	Mean	COV	Dist.	Source
OPC Surface Chloride Content, C_s	[a]%	0.94	0.17	[b]N	Experimental Work
OPC Apparent Diffusion Coefficient, D_{app}	m^2/s	1.36E-12	0.13	[b]N	Experimental Work
OPC + GGBS Surface Chloride Content, C_s	[a]%	1.27	0.07	[b]N	Experimental Work
OPC + GGBS Apparent Diffusion Coefficient, D_{app}	m^2/s	0.52E-12	0.08	[b]N	Experimental Work
Critical Chloride Content, C_{crit}	[a]%	0.100	0.20	[c]LN	[19]-[21]
Cover, x	mm	Spec .+ 1.6	σ = 11.1	[b]N	[19], [20], [21]

[a] Percentage by weight of concrete, [b] N is normal distribution, [c] LN is log normal distribution

To facilitate computation a cover of 50mm was adopted in accordance with the relevant Eurocodes for a bridge beam in a marine splash zone [11], [23]. The mean cover value utilised in the model is thus 51.6mm with a standard deviation of 11.1mm or CoV of 0.22. The CoV and mean bias were adopted from [19], [20] where they were utilised in a probabilistic model based on data collected from RC beams in [22]. This CoV value of approximately 0.2 for cover depth is used in many cases in the literature [17], [24], others have adopted a higher value for the cover CoV of 0.29 [25] and 0.45 [26].

There is much debate about the exact value of the critical chloride content C_{crit}. Angst [3] conducted a comprehensive study of experimental testing and site measurements and

found that on the basis of the literature it was not possible to select a reliable range of chloride threshold values. Researchers such as Bastidas-Arteaga et al and So et al [16], [17] used a mean C_{crit} value of 0.0375% by weight of concrete in probability based deterioration models. Stewart in his earlier papers [18], [27] adopted a similar value, however in more recent papers by Stewart a higher value of 0.100% by weight of concrete has been utilised [19]-[21]. It is this mean value that is used for the purpose of this analysis. The CoV value adopted for the C_{crit} value in this analysis is 0.2. A value of 0.19 or 0.2 is broadly used across the literature for C_{crit} even when the value of the mean C_{crit} differs [16]-[21], [27].

The values utilised in the probability based deterioration model for surface chloride content, C_s for both OPC and OPC + GGBS were obtained from the accelerated corrosion tests detailed in Section II. As can be seen from Table II the mean C_s values for OPC and OPC + GGBS are 0.94% and 1.27% by weight of concrete respectively. The CoV values obtained from the experimental data for OPC and OPC + GGBS are 0.17 and 0.07 respectively. When compared to the literature it can be said that the mean C_s values are high and the CoV values obtained from experimental testing are low. Val and Stewart [28] and Bastidas-Arteaga et al [16] used a mean C_s value of 0.306% by weight of concrete with and a C_s CoV of 0.70. These values were obtained from a comprehensive field-based study carried out by McGee [29] which presented data from 1,158 bridges in the Australian state of Tasmania.

Firstly, considering the CoV values, it is logical that the CoV for C_s would be far less for a controlled laboratory experiment than for the real marine environment. The data from 1,158 bridges would vary considerably about a mean due to factors such as different exposure conditions from site to site, tidal zone data, splash zone data and atmospheric exposure data compiled together, spatial variability on the structure itself, etc. The examination of the two repair materials in a controlled experiment allows a fair comparison to be drawn between OPC and OPC + GGBS without variation due to environmental conditions and material variation from bridge to bridge effecting the outcome i.e. a comparison can be drawn between the two materials will all other factors being equal. The purpose of estimating T_i for the two repair materials in this study is not to predict when exactly corrosion might initiate in a real marine structure. What is important in the analysis is the relative performance of the OPC repair concrete and OPC + GGBS repair concrete in resisting chloride ingress under the uniform laboratory conditions.

In terms of the mean C_s values, the nature of the accelerated tests results in mean C_s value that will be higher than those experienced in the real marine environment due to the frequent exposure of the samples to the 5% NaCl fog. Again it must be noted that it is the relative performance of the repair materials which is of interest for this study. It would be preferable to utilise real data collected from site, however there is very little data available for OPC + GGBS concrete bridges. In addition, every site has different exposure conditions making it difficult to perform a direct comparison between data collected from a bridge constructed with OPC concrete at one location and data collected from a bridge constructed with an OPC + GGBS concrete at another location. In addition the 8 month accelerated tests discussed in this paper allows results to be obtained over a total experimental program length of 13 months rather the real environment test duration of up to 20 years or more.

The mean D_{app} values obtained from the experimental testing for OPC and OPC + GGBS are 1.36E-12m^2/s and 0.52m^2/s. The D_{app} CoV values obtained from the experimental testing for OPC and OPC + GGBS are 0.13 and 0.08 respectively as can be seen from Table II. Morocous [26] utilised data collected from a field study whereby D_{app} was measured at 35 locations along the length of a bridge. The resultant mean D_{app} value utilised was 1.6E-12m^2/s with a CoV of 0.3. So in [17] used a mean D_{app} value of 1.032E-12m^2/s with a COV of 0.1. Rosowsky and Stewart [30] used a mean D_{app} value of 2E-12m^2/s with a CoV of 0.75. Thus the mean D_{app} values utilised for OPC concrete in probabilistic deterioration models in the literature is of the same order as the value obtained from the experimental testing and used herein.

The mean D_{app} value for OPC + GGBS found from the experimental testing is 0.38 times the OPC D_{app} value. A similar proportion of OPC to OPC + GGBS for D_{app} was also found by McPolin in [2] his study of these repair options where a proportion of 0.34 was reported. The CoV values for the D_{app} for both OPC and OPC + GGBS concrete are lower than the values used in the literature for the same reasons as outlined above for the CoV values of C_s.

IV. Results and Discussion

Fig. 5 below shows the frequency distributions generated via Monte Carlo simulation, for T_i for both OPC concrete and the OPC + GGBS concrete. Fig. 6 shows the same T_i values represented by probability density functions (PDF). The mean T_i values for the concretes are shown on the plot in Fig. 6 via vertical lines. For clarity, these values, along with standard deviations and CoV's are also presented in Table III. Fig. 5 and Fig. 6 clearly show that the OPC + GGBS concrete results in a larger time to initiation of corrosion than the OPC concrete. It can also be clearly seen in Fig. 5 & 6 that the spread of the OPC + GGBS T_i values is larger than the spread in the OPC T_i values. As shown in Table III however, both OPC and OPC + GGBS have similar COV values of 0.49 and 0.44 respectively.

The calculated mean T_i value is 2.1 times greater for OPC + GGBS concrete than OPC concrete i.e. based on the probabilistically modelled inputs of T_i it takes 2.1 times longer for corrosion to initiate in the OPC + GGBS concrete than in OPC concrete. In 2009 Melchers and Li published details of a comprehensive study of corrosion activation and initiation times in concrete structures exposed to severe marine environments [8]. The study, based on real structure data from corrosion and engineering literature, found that in general corrosion initiated in less than 5 years. It was found however, that for structures with GGBS the corrosion initiation period was at least 10 years and in many cases greater than 15 years. Thus, according to Melchers' and Li's study the relative merit for OPC + GGBS over OPC concrete was found to be between 2 and 3 +. This is in agreement with the value of 2.1 found in this analysis.

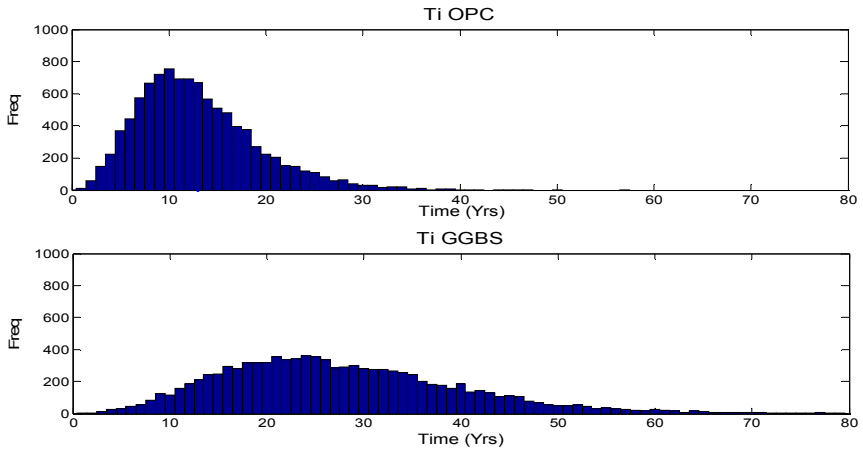

Fig. 5 Frequency distributions of T_i against years for OPC and OPC + GGBS

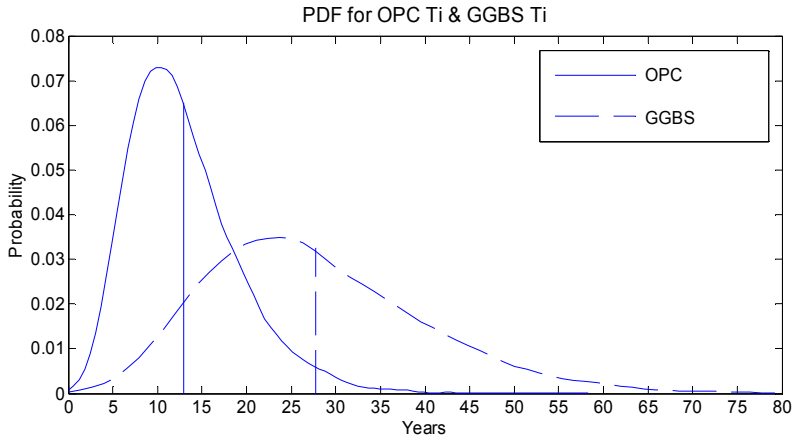

Fig. 6 Probability density functions of T_i against years for OPC and OPC + GGBS

TABLE III
T_i STATISTICAL PARAMETERS FROM MONTE CARLO SIMULATION

Repair Option	Mean Ti	St Dev. of Ti	CoV of Ti
OPC	13.01	6.36	0.49
GGBS	27.75	12.24	0.44

The benefits of using the OPC + GGBS concrete mix over the OPC concrete mix according to the analysis conducted herein can again be seen in the cumulative probability distribution shown below in Fig. 7. This plot shows the probability of corrosion having initiated from year to year, over a period of 80 years. As can be seen from the plot the probability of corrosion at a given time in a marine bridge element is greatly reduced for the OPC + GGBS concrete. For instance, according to the probability based deterioration model, when a marine bridge element is 10 years old there is a probability of initiation of corrosion of 0.04 for OPC + GGBS concrete while the probability of initiation of corrosion for OPC concrete at 10 years is 0.37. When the marine bridge element is 30 years into it's service life, the deterioration model predicts a probability of corrosion of 0.62 for the OPC + GGBS concrete and a probability of corrosion initiation of 0.98 for the OPC concrete element.

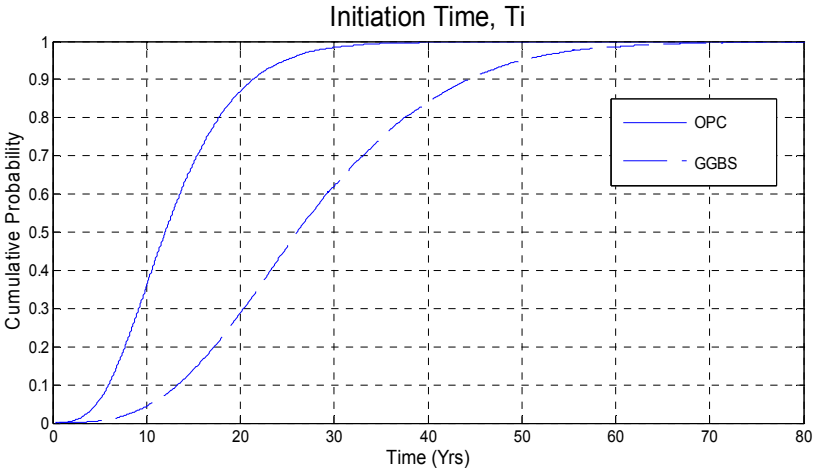

Fig. 7 Cumulative probability distribution of Ti against years for OPC and OPC + GGBS

V. Conclusion

The paper presented a statistical analysis of the time to initiation of corrosion for an OPC concrete and an OPC + GGBS concrete. The model utilised for the analysis incorporated results of accelerated chloride ingress testing carried out in a salt fog chamber. The analysis facilitated a probabilistic comparison of the durability characteristics of the two repair options in the initiation phase of corrosion. The results of the probabilistic deterioration model analysis found that OPC + GGBS concrete was more durable than OPC concrete by a factor of 2.1. This indicates that according to the deterioration model utilised the use of 60% GGBS as a partial replacement for OPC leads to a substantial extension of a marine bridge's service life.

Acknowledgment

The Irish National Roads Authority and the Irish Research Council for Science Engineering and Technology are gratefully acknowledged for the financial support provided in facilitating this research. The Authors would also like to thank the DurtiNet Project for it's support.

References

[1] V. Saraswathy, and H.W. Song, Evaluation of corrosion resistance of Portland pozzolana cement and fly ash blended cements in pre-cracked reinforced concrete slabs under accelerated testing conditions. Materials Chemistry and Physics, 2007. 104(2-3): p. 356-361.

[2] D. McPolin et al., Obtaining progressive chloride profiles in cementitious materials. Construction and Building Materials, 2005. 19(9): p. 666-673.

[3] U. Angst et al., Critical chloride content in reinforced concrete -- A review. Cement and Concrete Research, 2009. 39(12): p. 1122-1138.

[4] M. Collepardi, A. Marcialis, and R. Turriziani, The kinetics of chloride ions penetration in concrete, in Italian. Il Cemento, 1970. No.4: p. 157-164.

[5] G. Tosi, New Corrosion Control Legislation is Introduced in the house and Senate. Materials Performance, 2009. November 2009: p. pp 59.

[6] K. Tuutti, Service life of structures with regard to corrosion of embedded steel Metal Construction, 1979. 1(Compendex): p. 293-301.

[7] R.E. Weyers, Service life model for concrete structures in chloride laden environments. ACI Materials Journal, 1998. 95(Compendex): p. 445-453.

[8] R.E. Melchers, and C.Q. Li, Reinforcement corrosion initiation and activation times in concrete structures exposed to severe marine environments. Cement and Concrete Research, 2009. 39(Compendex): p. 1068-1076.

[9] European Standard Iistitution CEN, BS EN197-1:2000, Composition, specifications and conformity criteria for common cements 2000.

[10] European Standard Iistitution CEN, BS EN 15167-1:2006,Ground granulated blast furnace slag for use in concrete, mortar and grout — Part 1: Definitions, specifications and conformity criteria 2006.

[11] European Standard Iistitution CEN, BS EN 206-1: 2000, Concrete, Part 1: Specification, Performance, production and conformity. 2000.

[12] P.C. Ryan, et al., Repair, Monitoring & Experimental Work Associated with Ferrycarrig Bridge, in Bridge Maintenance, Safety and Management: Proceedings of the Fifth International IABMAS Conference, R. Sause, D. Frangopol, and C. Kusko, Editors. 2010: Philadelphia.

[13] British Standards Institution, BS 1881-111:1983. Method of Normal Curing of Test Specimens. 1983, HMSO London.

[14] M.D.A. Thomas, and J.D. Matthews, Performance of pfa concrete in a marine environment--10-year results. Cement and Concrete Composites, 2004. 26(1): p. 5-20.

[15] M. Nokken, et al., Time dependent diffusion in concrete--three laboratory studies. Cement and Concrete Research, 2006. 36(1): p. 200-207.

[16] E. Bastidas-Arteaga et al., Probabilistic lifetime assessment of RC structures under coupled corrosion-fatigue deterioration processes. Structural Safety, 2009. 31(1): p. 84-96.

[17] K.K.L. So, M.M.S. Cheung, and E.X.Q. Zhang, Life-cycle cost management of concrete bridges. Bridge Engineering, 2009. 162(September 2009 BE3): p. 103-117.

[18] K.A.T. Vu and M.G. Stewart, Structural reliability of concrete bridges including improved chloride-induced corrosion models. Structural Safety, 2000. 22(Compendex): p. 313-333.

[19] M.G. Stewart and Q. Suo, Extent of spatially variable corrosion damage as an indicator of strength and time-dependent reliability of RC beams. Engineering Structures, 2009. 31(1): p. 198-207.

[20] Q. Suo, and M.G. Stewart, Corrosion cracking prediction updating of deteriorating RC structures using inspection information. Reliability Engineering & System Safety, 2009. 94(8): p. 1340-1348.

[21] K.A.T. Vu and M.G. Stewart, Predicting the likelihood and extent of reinforced concrete corrosion-induced cracking. Journal of Structural Engineering, 2005. 131(Compendex): p. 1681-1689.

[22] S.A. Mirza and J.G. MacGregor, Variations in Dimensions of reinforced concrete members. 1979. 105(Compendex): p. 751-766.

[23] European Standard Iistitution CEN, BS EN 1992-1-1:2004, Design of concrete structures — Part 1-1: General rules and rules for buildings. 2004.

[24] R.E. Melchers, C.Q. Li, and W. Lawanwisut, Probabilistic modeling of structural deterioration of reinforced concrete beams under saline environment corrosion. Structural Safety, 2008. 30(5): p. 447-460.

[25] D.E. Choe et al., Seismic fragility estimates for reinforced concrete bridges subject to corrosion. Structural Safety, 2009. 31(Compendex): p. 275-283.
[26] G. Morcous, Z. Lounis, and Y. Cho, An integrated system for bridge management using probabilistic and mechanistic deterioration models: Application to bridge decks. KSCE Journal of Civil Engineering, 2010. 14(Compendex): p. 527-537.
[27] M.G. Stewart, and D.V. Rosowsky, Structural safety and serviceability of concrete bridges subject to corrosion. Journal of Infrastructure Systems, 1998. 4(Compendex): p. 146-155.
[28] D.V. Val and M.G. Stewart, Life-cycle cost analysis of reinforced concrete structures in marine environments. Structural Safety, 2003. 25(Compendex): p. 343-362.
[29] R. McGee, Modelling of durability performance of Tasmanian bridges, in Applications of statistics and probability in civil engineering, Melchers RE and M. Stewart, Editors. 2000: Rotterdam: Balkema. p. p. 297-306.
[30] M.G. Stewart, and D.V. Rosowsky, Time-dependent reliability of deteriorating reinforced concrete bridge decks. Structural Safety, 1998. 20(Compendex): p. 91-109.