Lifetime modelling of chloride-induced corrosion in concrete structures with Portland and blended cements

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ABSTRACT This article discusses mathematical modelling of the long-term performance of concrete with different supplementary cementitious materials in a maritime environment. The research was carried out in the light of the national Portuguese application of the CEN standards with mandatory requirements for a performance-based design approach. Laboratory investigations were performed on concrete compositions based on CEM I and CEM II/B-L in which the cement was partially replaced by either 0% (reference composition) or 50% of low calcium fly ash (FA). Concrete compositions were made with the objective to achieve service lives of 50 and 100 years with regard to steel corrosion. Test results of compressive strength, chloride potential diffusion and electrical resistivity are reported for different curing ages of 28, 90, 180 and 365 days. Chloride diffusion results were used for the implementation of modelling equations in order to estimate the design lifetime regarding reinforcing steel corrosion. A performance based approach using a probabilistic method was carried out and the results obtained are compared with the requirements according to the Portuguese prescriptive approach. The modelling results show that FA blended compositions have better performance compared to those with Portland cements, especially if curing ages beyond 28 days are considered.

1. Introduction

Concrete is the largest volume of concrete material used in the construction industry due to its wide availability and low cost of its components and the ease with which it can be prepared. As a result of the large volumes of concrete used, the production of Portland cement, the main binder of concrete, contributes to 5–8% of all produced CO2 (Sharp, Gartner, & MacPhee, 2010). Societies face the challenge of reducing the environmental impact of concrete without compromising the need for housing and infrastructure. From a concrete materials point of view, the main possible solutions are known and include (Flatt, Roussel, & Cheeseman, 2012) the following:

(1) Partial replacement of cement (clinker) by supplementary cementitious materials;
(2) Development of alternative binders;
(3) Wider implementation of concrete mix designs with reduced cement content;
(4) Recycling of demolished concrete in new concretes;
(5) Enhancement of durability (designing new infrastructures for a longer service life);
(6) Rehabilitation of existing infrastructures (extending the service life of existing infrastructures).

Predicting the service life of a concrete structure has always been a complex issue taking into account the presence of chlorides, CO2 and acids which will penetrate into a reactive porous milieu. Simultaneously, drying and wetting cycles modify the water content of the hardened concrete. Most chemical reactions occurring in the material are strongly related, and only in the simplest cases, the resulting behaviour can be correctly modelled.

According to ACI 365 (2000), the service life of a structural component is the period after construction when all the properties exceed the minimum acceptable values when routinely maintained. Predicting service life requires a mathematical model that simulates the behaviour of the structure under the prevailing environmental actions.

Such a model should take into account the relevant physical and chemical mechanisms for which the performance of the concrete in a certain environment must be assessed (Ferreira,

2004). Tuutti (1980, 1982) presented a conceptual model in which the service life for concrete structures with regard to reinforcement corrosion is subdivided into an initial and a subsequent propagation stage (Figure 1). The initiation stage refers to the penetration of the aggressive agents into the concrete cover, while the propagation stage is related to the evolution of different forms of deterioration after corrosion has been initiated. The sum of

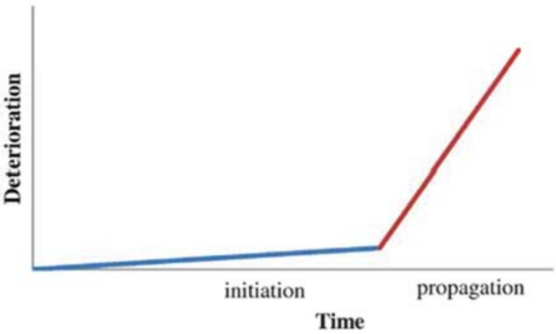

Figure 1. Tuutti's model for deterioration of structures.

the initiation and propagation equals the service life period of the structure. The length of both time periods depends on the characteristics of the environmental exposure conditions the structure or its members are subject to.

It has been shown that deterioration occurring long before the estimated design limits of reinforced concrete (RC) structures is of great concern nowadays (Folic, 2009; ICDCS, 2008; Mays, 2001; RILEM, 2009). In fact, Portuguese codes have evolved since the 1970s in order to take into account this important issue, for example by increasing the concrete cover by 10 mm for protected (interior spaces or protected with plasters or other barriers) RC elements and 20 mm for non-protected (concrete exposed directly to rain) elements (REBA, 1967) and 30—40 mm depend- ing on the chemical aggressiveness of the environment and the concrete compressive strength (REBAP, 1983). In Portugal, a significant number of structures that were designed and built in the 1980s and 1990s for a working life of 50 years are now close to their acceptable serviceability limits or have even passed beyond these, usually because of premature damage resulting from steel reinforcement corrosion (Costa & Appleton, 1998; Costa & Appleton, 2002).

Some of the previous codes (CEB-FIP, 1993; Eurocode 0, 2002; Eurocode 2, 2004) provided only qualitative definitions of exposure conditions and they fail to define the design life with respect to durability. The adopted approach is empirical and needs to be calibrated by physical models considering the variability of the environmental and materials parameters. For this reason, sev- eral authors have developed mathematical models for service life prediction (Mitchell & Frohnsdorff, 2004) which are now present in new benchmark codes Jfi Model Code, 2010).

This aspect is particularly important when RC structures are exposed to aggressive environments, such as marine environments (Maekawa, Ishida, & Chijiwa, 2007). In fact, in these situations, concrete is often found in a non-saturated condition rather than a saturated condition, with high levels of chlorides and relative humidity. Chloride penetration into non-saturated concrete structures is a complex phenomenon, involving various factors such as diffusion due to non-uniform distribution of chloride ions and convection due to moisture transport by capillary action into concrete. Consequently, RC structures in coastal environment or subjected to the presence of de-icing salts are significantly affected and have to be conceived taking these aspects into account (Gjorv, 2009). This has led to the definition of recent recommendations (Jb Model Code, 2010) and standards (ISO 16204, 2012) for this purpose, where, besides deemed-to-satisfy design and avoidance-of-deterioration design, performance may be assessed using a probabilistic approach or even a partial safety factor (semi-probabilistic) approach in the design stage.

2. Definition of design lifetime

2.I. Prescriptive definition

As an Annex to the NP EN 206-1 (2005), the prescriptive methodology LNEC E464 (2007) sets the requirements for the concrete constituents (maximum water/cement ratio, minimum cement dosage and cement type), the minimum compressive strength and the concrete cover thickness for a design working life of 50 years (target period). Eurocode 2 (2004) defines and describes environmental exposure classes in view of aggressive agents. In a marine environment, three exposure classes are defined with respect to chloride-induced corrosion which are described by:

- XS1: areas exposed to airborne salt but not in direct contact with sea water;
- XS2: permanently submerged elements;
- XS3: tidal, splash and spray zones.

In environments where chlorides are not originating from sea water, such as those where de-icing salts are present, the exposure classes follow the same principle although with a different designation: XD1; XD2 and XD3.

According to Eurocode 2, the same prescribed limits of the concrete composition and an additional 10 mm concrete cover allow an extension of the design working life to 100 years. The specification LNEC E465 (2007), following the guidelines of Eurocode 2 (2004), classifies the minimum durability concrete cover — c,, ,, — for RC structures according to struc- tural classes with which the design of the working life and type of the structures are associated (2010). Table 1 presents the defined limits according to the Portuguese specification (2007).

Table 1. Prescriptive limits for working life of 50 years LNEC E464 (2007).

Cement type	CEM IV/A (Reference); CEM IV/B; CEM III/A; CEM III/B; CEM V; CEM II/B; CEM II/A-D			CEM I; CEM II/A;		
Exposure class	XS1	XS2	XS3	XS1	XS2	XS3
Minimum nominal cover (mm)	45	50	55	45	50	55
Maximum vr/c	0.55	0.55	0.45	0.45	0.45	0.40
Minimum cement dosage (kg/m")	320	320	340	360	360	380
Minimum strength class	C30/37 LC30/33	C30/37 LC30/33	C35/45 LC35/38	C40/50 LC40/44	C40/S0 LC40/44	C40/50 LC40/44

Note:"c,,, = c,;, g,, + 10 mm (Eurocode, 2004; LNEC E465, 2007)

3.2. Performance-based method

Specification LNEC E465 (2007) has introduced a performance-based approach that involves a thoughtful and realistic assessment of the correlation between design, durability along with future maintenance and repair. The objective of this approach is to ensure that the required performance will be maintained throughout the intended life of the structure along with the optimisation of the inherent lifetime costs (Narashiman & Chew, 2009). Based on this approach, the design of concrete compositions can, therefore, be carried out through performance-based indicators as an alternative to define strict limits to the concrete mix proportions. In fact, since the relevant properties of each composition have to be assessed through testing and the results will be evaluated, in principle there are no limits whatsoever for the type and quantity of constituents.

Taking into account the conceptual deterioration model for corrosion of steel into concrete (Tuutti, 1950), in which two periods are clearly defined: during the *initiation period* where external agents penetrate into concrete until they reach the reinforcing steel in such an amount to result into the onset of corrosion; subsequently, there is the propagation period characterised by the development of damage to the concrete structures due to the onset of corrosion. The performance-based approach criterion relies on the probability of a deterioration agent attaining a certain depth and/or quantity or the probability of its effect resulting in a certain level of deterioration.

In either case, this probability (of failure) cannot surpass the values associated with the so-called limit state, defined in standards or codes. Eurocode 0 (2002) and the Portuguese specification LNEC E465 (2007) set the maximum values of probability of failure *p* and the corresponding minimum reliability index *{'y* for three different reliability classes RC3, RC2 and RC1 (Eurocode 0, 2002). The probability *p* is related to the reliability index J based on the standard normal distribution (Equation (1)) (Ferry-Borges & Castanheta, 1955; Thoft-Christensen & Baker, 1982).

$$p_f \quad \Phi(-\beta) \tag{1}$$

For corrosion deterioration, the ultimate limit state (collapse) is not considered by the LNEC E465 (2007). This specification considers the limit state associated with serviceability for reliability class RC2, which means = 1.5 corresponding to *p* — 6.7*/. In view of this, test results are included in mathematical models in order to perform a lifetime estimation regarding the type of action – chlorides (Baroghel-Bouny, Nguyen, & Dangla, 2006) or carbonation (Thiery, Villain, Baroghel-Bouny, & Dangla, 2006). The following sections will present the existing service life predictive models based on durability indicators for chloride-induced corrosion and the following criteria of acceptance or failure according to the Portuguese standards (LNEC E465, 2007; NP EN 206-1, 2005).

2.2.I. Modelling of the initiation period for chlorides ingress

The initiation period concerns the penetration of chlorides into concrete and is based on the mathematical model of/fi bulletin 34 (2006), now in /fi Model Code (2010), which has been adopted in the specification LNEC 465 (2007) with the required adjustments regarding the Portuguese exposure environment. For the analysed exposure classes, the propagation period is discarded by this specification since the corrosion process is rapid. Consequently, the end of the initiation period is taken as the accepted limit for the performance of a specific RC element.

The chloride content known as the threshold for the initiation of corrosion is defined by the performance-based specification (LNEC 465, 2007) as C, and corresponds to a penetration depth that is equal to the cover depth. The parameter C is defined by mean values of 0.4%i of binder mass for classes XS1 and XS2 and 0.3% for class XS3 (LNEC 465, 2007). According to available references, these values for the chloride threshold C_R defined by the Portuguese specification E46S are extremely low to be considered as mean values (Angst, Elsener, Larsen, & Vennesland, 2009; Bertolini, 2011; CS TR61, 2004; /6 Model Code, 2010), whereas mean values of 0.6% are more commonly considered.

In the case of the model defined for the estimation of the initiation period, chloride penetration is taken into account by means of diffusion through the concrete cover using Fick's second law (Mokhtar, Loche, Friedmann, Amiri, & Ammar, 2007), although considering that the diffusion coefficient diminishes with the exposure time (Costa & Appleton, 2002):

$$x \quad 2\mathrm{er}^{1}\left(1 - \frac{c\ ,t)}{C_s}\right)\sqrt{D(t)t} \tag{2}$$

where x is the chloride penetration depth (m) with a content of *C(x, I),- D(I)* is the coefficient of diffusion of chlorides in concrete, as a function of exposure time (m^2/s); *C(x, t)* is the chloride content in percentage of binder mass (%), at the depth x (m) after an exposure time *t* (s); C, is the chloride content in percentage of cement ('Fo) at the external concrete surface (x = 0) assumed to be constant over time; erf°' is the inverse of the error function.

The following equations allow the determination of $C_.$ and *D(t)* (LNEC E465, 2007):

$$C_s \quad C_b 2,5(w/c)k_{\mathrm{Vert}}k_{\mathrm{Hor}}k_{\mathrm{Temp}} \tag{3}$$

where C¿ is the surface chloride content that accounts for the salinity of the sea water at the Portuguese coast (21 g/l) with a temperature of (16 + 2) °C and is defined as 3% for exposure classes XS2 and XS3 and 2*/o for class XS1, *w/c* is the water/cement ratio of the concrete compositions (w/c = 0.45), *k, „* — 0.7 for class XS1 and 1.0 for class XS3 – and *kg,,* — 1.0 for 0 km of shore distance and 0.6 for I km from shore – are coefficients related to the environmental exposure and *k „„p* is the coefficient that accounts for the concrete temperature (Table 2) assumed as mean value of 15 °C through one year (k_T,p = 1.2):

Table 2. k„„ and k„as function of the concrete temperature.

Temperature	0 °C	10 °C	15 °C	20 °C	25 °C	30 °C	35°C
kp	2.2	1.5	1.2	1.0	0.8	0.7	0.6
kg	0.4	0.75	0.8	1.0	1.2	1.5	

$$D(t) \quad I_D \; k_D \quad k_{D\,T,}D \left(\frac{t_0}{t}\right)^n \quad \Leftrightarrow$$

$$D(t) \quad I_,D \left({t} \right) \tag{4}$$

where D is the potential diffusion coefficient obtained from laboratory testing according to NT Build 492 (1999) (m°/s) at a curing age f,, (usually 28 days), I_D is the factor that accounts for the curing influence — 2.4 for normal curing; 0.75 for permanent contact with water, $kp\ g_H$ accounts for the influence of the relative humidity — 0.4 for class XSI and 1.0 for classes XS2 and XS3, whereas $k_{D\,T}$ accounts for the temperature influence (Table 2). The ageing factor n expresses the effect of ageing on the chloride penetration in time and is defined by the specification LNEC E465 (2007) based on the internal experimental research of the Portuguese National Laboratory of Civil Engineering for the specific Portuguese environments. For classes XSI and XS3, rt is equal to 0.55 for CEM I and II, while for CEM III and IV, a value of 0.65 has been adopted.

Once the potential coefficient of diffusion and the concrete cover are known, and employing Equations (2) and (4), the initiation period f, which replaces f, may be obtained as follows:

$$t_i \quad \left| \left(\frac{2}{c} \mathrm{erf}^{-1} \left(1 - \frac{C - C,}{C - C} \right) \right)^{-z} \frac{j}{k\,D,\,t\text{“},} \right. \tag{5}$$

with c (m) being the concrete cover depth and C, the initial chloride content, by weight of cement, in the concrete composition.

2.2.2. Implementation of the probabilistic analysis

The probabilistic analysis of lifetime distribution was carried out using the limit state function with respect to chloride diffusion (Equation (6)) as well as the statistical parameters of the involved variables – mean and coefficient of variation (CoV) (Table 7).

The probabilistic calculus was implemented by means of the Monte Carlo method with 1,00,000 generated numbers for each random variable. The mean values of each variable were based on the experimental programme and the parameters of LNEC E465 (2007), while the values adopted for the standard deviation were based on h bulletin 34 (2006) and Val and Trapper (2005).

As outlined above, the concrete cover specified for a target period ft of 100 years is that specified for 50 years plus 10 mm. (Equation (6)) expresses the limit state function used for the implementation of the Monte Carlo method, where l is the lifetime design with $t_{t.}$— (initiation) t (propagation). In the present case, given the existing standards – LNEC E465 (2007), EN 1990 (2002), for those exposure classes concerning the presence of chloride in aerial (XSI) or tidal (XS3) zones, the propagation period t_p is discarded, given the short period in which corrosion propagates. Therefore, the criterion is to consider lifetime design depending only on the initiation period – t — $t;$ (initiation). The parameter 2 represents the uncertainty of the mathematical model:

$$g\,(x) \quad t_L - t_g \quad \lambda \left(2_x \,\mathrm{era'} \left(1 - \frac{C\ - C;}{C\ - Cl} \right) \right)^{-z} \frac{1}{k\,,,Dt,'} \quad - t_g \tag{6}$$

The probability of failure is expressed as the probability that the limit state function is negative:

$$p_f = p\left[g(x) < 0\right] \tag{7}$$

3. Experimental programme

3.1. Introduction

The purpose of the experimental part of this study is to evaluate the properties of blended FA concrete compositions compared to ordinary Portland cement (OPC, only clinker as binder) and PC—L (clinker and limestone filler as binder), concerning its durability with respect to chloride diffusion beyond 28 days of age. Cement types CEM I 42.5R (OPC) and CEM II/A-L 32.5 N (PC—L) are the most widely used cements in the Portuguese market. Two sets of reference concrete compositions with cement of high clinker content were defined, complying with EN 197-1 (2000), i.e. CEM I 42.5R (OPC) and CEM II/B-L 32.5 N (PC—L). Two other compositions were defined based on these two types of cement although now replaced by fly ashes (FA) in a proportion of 50% each of total binder. All compositions were designed in order to respect the prescribed limits of the specification E464 (2007) of NP EN 206-1 (2005), namely the use of no more than 50%i of FA as a binder.

The determination of the compressive strength at an age of 2S days and the corresponding preconditioning were carried out following the standard EN 12390-3 (2009). The same procedure was followed for curing of 90, 180 and 365 days. For the chloride diffusion tests, cylindrical specimens of 100 mm in diameter and a thickness of 50 mm were used following the Portuguese specification LNEC E463 (200a) based on the Nordic standard NT Build 492 (1999).

3.2. Concrete compositions

The constituents and properties of the cement and FA used in this study are presented in Table 3. Each composition set has three mixes varying in total binder dosage: 330, 360 and 390 kg/m", respectively; and in all cases, the water/binder (w/fi) ratio was maintained at 0.45. All compositions were specified with the same aggregates and in similar proportions which included two fine silica-based aggregates and two coarse limestone aggregates (Tables 4 and 5).

4. Test results

Although it does not have a direct relationship to durability, concrete compressive strength is a reference parameter as regards the performance of a concrete composition and the tests were carried out following the standard NP EN 12390-3 (2009) at an age of 28, 90, 180 and 365 days. Regarding chlorides, the experimental procedure for the determination of the coefficient of potential diffusion followed the procedure adopted in NT Build 492 (1999), which included cylindrical specimens with 100 mm diameter and a thickness of 50 mm. Although not specified by NT Build 492, the specimens were subjected to 14 days of drying at 20 °C and 50% of RH, before being in a low pressure closed recipient and immersed in a solution of calcium hydroxide. Despite the differences in the pre-treatment of these specimens

Table 3. Portland cement OPC (CEM I 42.5R) and PC—L (CEM II/B-L 32.5N) and FA. Constituents and properties — wt96.

	CEM I (OPC)	CEM II/B-L (PC-L)	FA
Clinker (96)	95	60	
Lime filler (to)		35	
Loss on ignition (°é)	3.17	14.42	5.4J
SiO (%)	19.45	15.04	50.13
AI,O, (96)	4.17	3.33	22.20
Fe O (96)	3.5	2.96	9.74
Ca0 (é)	62.42	61.09	4.13
MgO (96)	2.20	1.30	1.44
CI ("é)	0.03	0.03	
so (No)	2.90	2.51	0.82
Ca0 free ("6)	1.39	0.89	0.47
Density (g/cm*)	3.11	2.99	2.46
Specific surface area (cm°/g)	4408	5491	3343
Compressive strength (iVIPo)			
2d	31.9	20.2	
7d	45.5	30.9	
28d	56.9	395	

Table 4. Concrete compositions with OPC (CEM I 42.5R) and PC—L (CEM II/B-L 32.SN) as binder — kg/m'.

	1330	1360	1390	II 330	II 360	II 390
Type of cement	CEM I	CEM I	CEM I	CEM II/B-L	CEM II/B-L	CEM II/B-L
Cement dosage	330	360	390	330	360	390
Fly ash dosage						
Sand 0.25—0.5 mm	260	240	220	260	240	220
Sand 1—2 mm	580	550	520	580	550	520
Gravel 8—10 mm	470	490	500	470	490	500
Gravel 14—25 mm	530	530	540	530	530	540
web	0.45	0.45	0.45	0.45	0.45	0.45

Notes: 1330 — concrete composition with type of cement CEM I with 330 kg/m^ dosage.
II 330 — concrete composition with type of cement CEM II/B-L with 330 kg/m" dosage.

Table 5. Concrete compositions with 509a of FA as binder — kg/m".

	1330 FA	1360 FA	390 FA	II 330 FA	II 360 FA	II 390 FA
Type of cement	CEM I	CEM I	CEM I	CEM II/B-L	CEM II/B-L	CEM II/B-L
Cement dosage	165	80	195	65	180	195
Fly ash dosage	65	80	193	65	80	195
Sand 0.25—0.5 mm	260	240	220	260	240	220
Sand 1—2 mm	580	550	520	580	550	520
Gravel 8—10 mm	470	490	500	470	490	500
Gravel 14—25 mm	530	530	540	530	530	540
web	0.45	0.45	0.45	0.45	0.45	0.45

Notes: 1330 FA– concrete composition with 330 kg/m' binder dosage: 165 kg/m' CEM I and 165 kg/m' FA. II 330 FA —concrete composition with 330 kg/m" binder dosage: 165 kg/m" CEM II/B-L and 165 kg/m" FA.

compared with the compressive strength specimens and how these influence the results, the authors opted not to change the procedure adopted in previous studies. The same test made it possible to measure the electrical resistivity.

4.1. Compressive strength

The experimental work included concrete compositions which were subjected to compressive tests at the ages of 2S, 90, 180 and 365 days. In view of the existing data (Wood, 1992), the results (Figures 3 and 4) show that concrete mixes with Portland cement have higher values of compressive strength at early stages of curing. Nevertheless, while composition with cement I (OPC) has the highest results for all ages, cement II/B-L (PC-L) presents the lowest values for 180 and 90 days of age, even when compared to the blended cement mixes. The results show that there is higher increase in the compressive strength for FA blended cement when the age of specimens is taken into account.

4.2. Chlorides ingress: potential diffusion coefficient

As already mentioned, the durability parameter investigated in this study herein refers to the penetration of chlorides into the concrete specimens. The test follows the Nordic standard NT Build 492 (1999) non-stationary method and consists of applying an external electrical potential difference axially across the specimens forcing the chloride ions contained in the exposure solution to migrate into the specimen. After the test, specimens are split and a silver nitrate solution is sprayed onto the freshly split sections. The depth of the white silver chloride precipitation can then be measured. From these measurements, the chloride migration coefficient can be calculated.

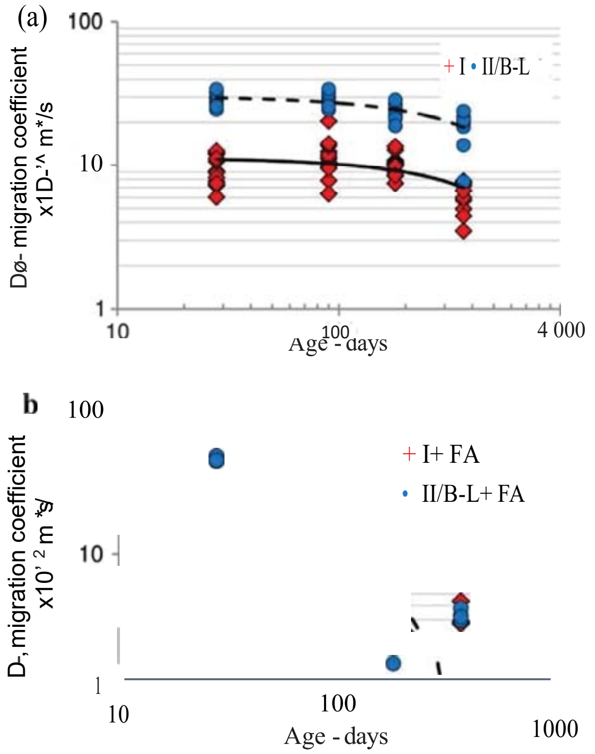

Figure 2. Chlorides migration coefficient, *D :* (a) cements I and II/B-L; (b) I+50%FA and II/B-L+509oFA.

The results of the four compositions for each age are shown in Figures 2(a) and (b) and in Table 6, where the compositions are grouped by binder type to obtain the corresponding potential diffusion coefficient *D,,*. Table 6 shows the individual results of the potential diffusion coefficient D,, for each dosage of 330, 360 and 390 kg/m'. Figures 2(a) and (b) present the results for the chloride potential diffusion coefficient of the concrete compositions for all ages. It can be seen that at an age of 28 days, the composition with CEM I (OPC) is the one that presents, distinctly, the lowest value for this coefficient, meaning less chloride potential diffusion. However, at 90 days of age, the compositions with FA — CEM I + 50'/oFA and CEM II + 50'/oFA already show less chloride diffusion than concrete composition CEM I. The trend of improvement at younger ages and when compared to CEM I can be demonstrated for ages of 180 and 365 days. Composition CEM I showed a negligible change for an age of both 180 days and 365 days. This aspect indicates that, for chloride action, the compositions blended with FA show a significant improvement in performance between 28 and 90 days of age, surpassing the composition of Portland cement in an evident way. With regard to the composition CEM II/B-L, the results indicate a small variation in the high values of chloride potential diffusion at different ages: 28, 90, 180 and 365 days.

Considering that the coefficient of diffusion varies with time, the results predicted by Equation (4) should be reasonably approximate to the experimental results expressed in Figures 2(a) and (b). However, it is possible to observe, on one hand, that these figures show a relationship between age and chloride diffusion far from linear in a logarithmic scale in both axes, which is the outcome if Equation (4) is implemented. On the other hand, even considering a rough approximation to linear behaviour to meet the experimental results, the prediction results using Equation (4) would have to consider ageing factors of *n* - 0.3 for the concrete compositions with CEM I and CEM II and n > 0.7 for concrete compositions with FA, which is extremely high and possibly unrealistic value (CS TR61, 2004; DuraCrete, 2000; Ferreira, 2004; Jfi Model Code, 2010). As presented before in this article, the ageing factor values defined by the Portuguese specification as input in a performance-based approach are significantly different, varying between 0.55 (Portland cement) and 0.6S (blended cement with FA).

4.3. *Chloride potential diffusion coeffĩcient, compressive* strength *and electrical resistivity*

The results of the chloride potential diffusion coefficient alongside compressive strength are shown in Figure 3. The results of electrical resistivity fi, measured on the specimens of each composition, are also presented in the same figure (Figure 3). The values of the concrete resistivity were calculated using Equation (8), where U (V) is the electrical potential difference across the thickness of the specimen, *A* (m^2) is the area of each specimen through which chloride ions migrate, *h* (m) is the depth of the specimen, and /,, is the mean current intensity (A) throughout the test:

$$\_ \frac{U A}{h I,,} \tag{8}$$

In order to understand if there is any kind of correlation between chloride potential diffusion and compressive strength of the studied compositions, i.e. D,, vs.,, both results were

Table 6. Test results of concrete chloride migration coefficient D, (x10—" m'/s) and compressive strength f, (MPa).

Concrete composition		28 days			90 days			180 days			365 days		
Binder type	Dosage (kg/m')	*D,,*	Temp. ("C)	*D,*	*D,,*	Temp. ("C)	*D,*	*D,*	Temp. ("C)	*D,*	*D,,*	Temp. ("C)	*D,*
	330	9.1	20.0		9.8	19.1		10.0	23.0		7.5	15.1	
	360	7.2	17.4	9.4	11.2	19.8	11.6	10.4	23.7	10.1	5.7	17.1	6.4
	390	11.9	18.8		13.7	22.6		9.8	25.3		6.0	15.0	
	330	30.2	19.4		29.5	19.1		23.9	23.0		13.8	15.1	
II/B-L	360	13.8	17.6	25.8	26.7	19.7	29.0	24.0	23.5	23.6	20.0	16.7	18.0
	390	33.5	18.5		30.8	22.7		22.9	25.0		20.2	IN.0	
	330	17.3	19.2		8.2	21.2		4.1	23.8		0.9	13.2	
I +5096 FA	360	17.6	18.0	17.1	5.5	22.3	6.8	2.3	24.4	3.0	3.9	14.2	1.9
	390	16.5	17.4		6.6	25.5		2.7	24.8		0.9	13.0	
	330	37.4	18.7		11.7	20.8		3.9	23.8		1.8	13.2	
II +5096 FA	360	42.0	18.2	39.1	9.4	22.0	10.5	2.7	24.5	3.1	4.3	14.0	2.5
	390	37.8	17.2		10.5	25.6		2.6	24.5		1.3	13.0	

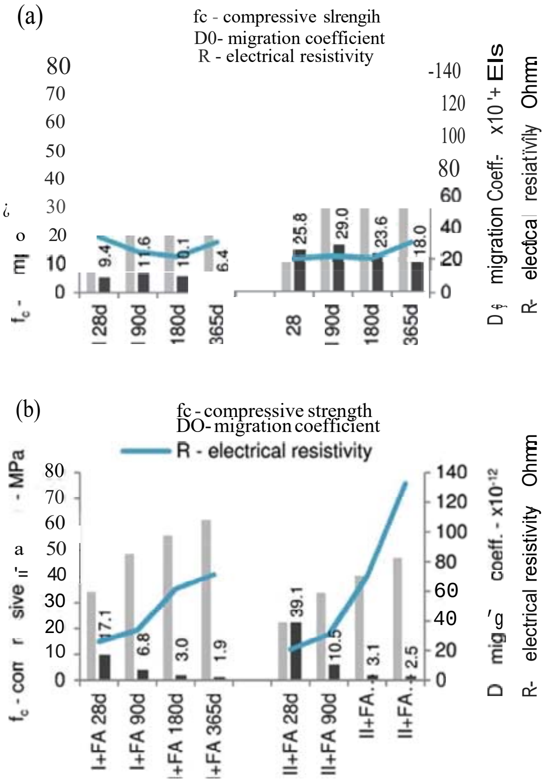

Figure 3. *D*, I, and P results of concrete specimens with different type of binder: (a) cement I and cement II/B-L; (b) cement I+5096FA and cement II/B-L+5096FA.

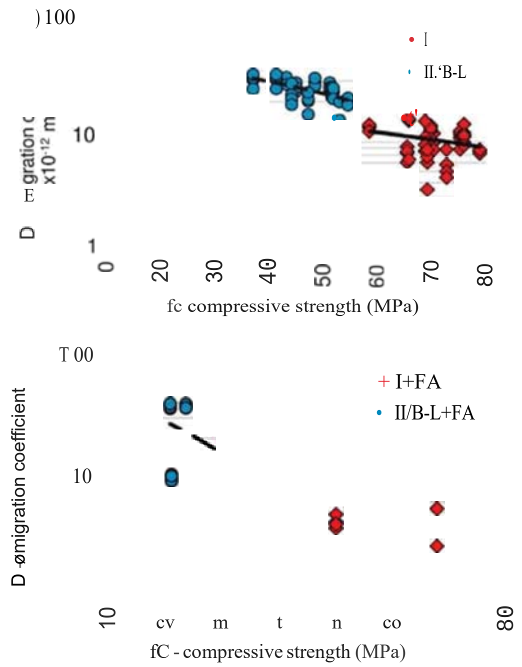

Figure 4. *D*, vs. I,: (a) cement types I and II/B-L; (b) I+509oFA and II/B-L+5096FA.

compared in Figure 4. It can be seen that CEM I and CEM II/B-L show significant differences between them (Figure 4(a)), although each composition shows low variation of *D* with (trend lines are of little inclination). For the FA compositions, CEM I + 50%FA and CEM II + 50%›FA, there is such a high variation of chloride potential diffusion with the compressive strength that both trend lines are approximately exponential (Figure 4(b)), which means that a little increment in compressive strength corresponds to a significant reduction in the chloride potential diffusion coefficient.

With respect to the relationship between the chloride potential diffusion coefficient D, and the electrical resistivity fi (Figure ñ), a similar trend is observed since there is no relevant change in,*D* with fi for the compositions without FA — CEM I and CEM II/B-L (Figure 5(a)) — while for concrete compositions with FA — CEM I + 50%›FA and CEM II + 50%›FA — the variation between these two properties seems to be greater (Figure 5(b)).

4.4. Oiscnssion oFfest *results*

For each type of binder, 3 dosages were used for the studied concrete compositions being 330, 360 and 390 kg/m', respectively. According to the results in Table 7, the relative difference, between 30 and 60 kg/m", does not seem to have had a relevant influence on the mentioned results of the potential diffusion coefficient. The experimental results show that the compositions with blended FA cement — CEM I + 50'?oFA and CEM II + 50*/oFA — have a lower chloride potential diffusion coefficient from 90 days of age and with significant improvement which derives possibly from the pozzolanic effect of FA. For specimens with 25 days of age, composition CEM I had the lowest, and best, potential diffusion coefficient though with no evolution for the other curing ages — 90, 180 and 365 days. The concrete composition with cement type CEM II/B-L maintained high values of potential diffusion coefficient for all curing ages compared to the other tested compositions. Moreover, there was a negligible improvement over time, as regards the curing age.

5. Design lifetime results

The estimation of the lifetime period using the probabilistic method, where all random variables have their distribution laws, the mean concrete cover values were taken as 45 mm (t_g = 50 years) and 55 mm (t_g = 100 years) for exposure class XS1 and 55 mm (t_g = 50 years) and 65 mm (t_g = 100 years) for exposure class XS3. The analysis was based on the assumption that the performance limit is expressed as reliability index J 1.5 or probability of failure p 7% (LNEC E465, 2007).

5. I. Input variables of the pro6o6i/istic approach

In the present recommendations and standards (DuraCrete, 2000; Jh Model Code, 2010; LNEC E465, 2007; RILEM, 1996), the estimation of the design lifetime of RC structures regarding reinforcement corrosion is based on test results of concrete specimens obtained at an age of 28 days. This study includes also the results obtained at 90, 180 and 36ñ days. As previously mentioned, the probabilistic method is based on the equations and parameters defined in the specification LNEC E465 (2007). However, the design lifetime results *t* are calculated taking into consideration the mean values of the random variables (Table 7) and their distribution laws according to existing references (DuraCrete, 2000; Ferreira, 2004; J6 Model Code,

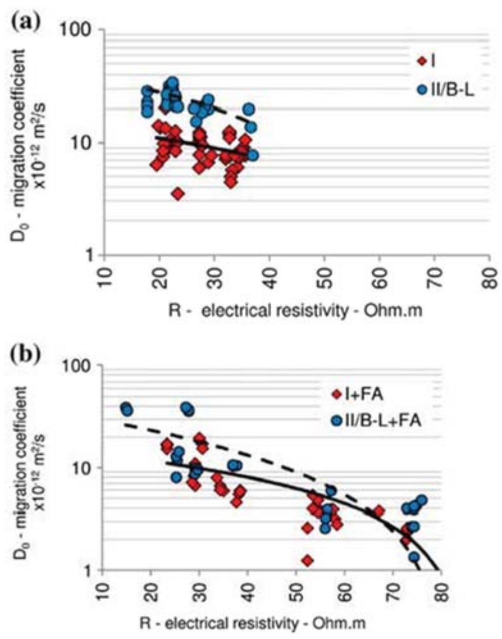

Figure 5. *D*, vs. *P:* (a) cement types I and II/B-L; (b) I+509oFA and II/B-L+5096FA.

2010; Lindvall, 2003; Marques, Costa, & Lanata, 2010). The CoV of the model uncertainty was assumed in view of what some authors considered (Val & Trapper, 2008).

The probabilistic calculus for the design lifetime were carried out by means of the Monte Carlo method in which the random variables of the limit state function have been considered with probability distribution functions according to various reference documents Jfi Model Code, 2010; Lindvall, 2003).

5.3. Lifetime results and discussion of the probabilistic approach

The design lifetime results concerning the performance limit of $p_f = 7\%$ ($\beta = 1.5$) as failure probability – probability of $t_L = t_g$ – were obtained with the implementation of the Monte Carlo method using 1,00,000 generated values for each random variable. The results are depicted in Figures 6 and 7. For the marine atmospheric environment — exposure class XS1 (Figure 6), the target period t_o = ú0 years is only attained by composition CEM I at 28 days and despite CEM I + 50‘FoFA showing design lifetime value of nearly 40 years. For the target periOd ′ 100 years, only CEM I and CEM I + 50%FA reach or outdo this value. When analysing the performance of the FA blended compositions tested at 90, 180 and 365 days, the design lifetime results show an improving performance. It is evident that the results for the compositions with FA — CEM I + 50"%FA and CEM II + ñ0"/oFA – which highlights the possible pozzolanic effect of this addition, particularly from the age of 180 days. For concrete ages of 180 and 365 days, blended compositions result in calculated lifetime values that clearly surpass the target periods.

Regarding exposure class XS3 — tidal or spray zone environment (Figure 7), all the composition lifetime results are far from both target periods of 30 and 100 years, though with a tendency similar to class XSI. That is, the compositions blended with FA show increasing performance from 2S to 365 days. The concrete composition CEM II/B-L shows, for all ages, comparatively low lifetime results and with no change in relation to different curing ages. For the FA blended compositions, the increase in performance over time is evident. For the compositions without FA — CEM I and CEM II/L-B — there is one lifetime result with a decrease in performance. This aspect might be explained by the quantification of an ageing factor *n,* which models the

Table 7. Probabilistic variables for the calculus of the design lifetime.

Variable	Mean value	CoV	Distribution
D_0	Table 6	0.20	Normal
c (c_{nom})	45 mm (I, = 50 years)	0.25	Log-normal
	55 mm (f = 100 years)	0.20	
t_0	28; 90; 18a and 3ó5 days		Deterministic
C_R	0.4 (No cement mass)	0. 2	Normal
C_s	1.9 ("ü cement mass)	0.20	Normal
n	0.55 I/II	0.10	Normal
	0.65 I+5096FA/II + 50"AFA	0. 0	Normal
$k_{D,T}$	0.81	0.20	Normal
$k_{,,}$	0.40		Deterministic
k	2.4		Deterministic
Model uncertainty, X	1.0	0.15	Normal
D_0	Table 6	0.20	Normal
c (c_{nom})	55 mm (f, = 50 years)	0.25	Log-normal
	65 mm (f = 100 years)	0.20	
t_0	28; 90; 18a and 365 days		Deterministic
C,	0.3 (No cement mass)	0.12	Normal
C_S	4.1 ("ü cement mass)	0.20	Normal
n	0.55 I/II	0. 0	Normal
	0.65 I+5096FA/II +50"ÓFA	0. 0	Normal
$K_{D,T}$	0.81	0.20	Normal
$k_{„}$	1.0		Deterministic
k	2.4		Deterministic
Model uncertainty, \	1.0	0.15	Normal

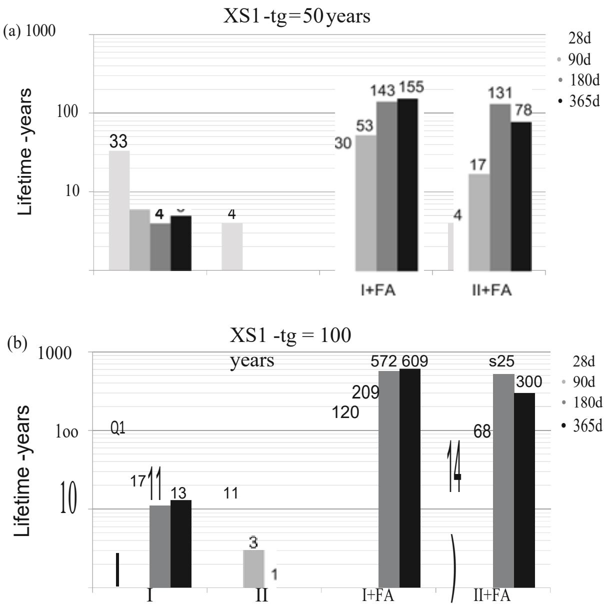

Figure 6. Lifetime results for the ages of 28, 90, 180 and 365 days: Exposure class XS1: (a) target period rd= 50; years (b) target period rd= 100.

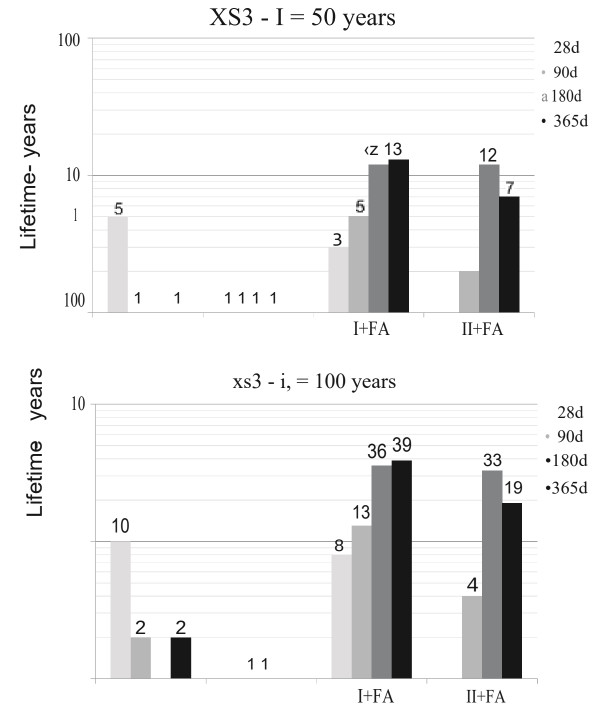

Figure 7. Lifetime results for the ages of 28, 90, 180 and 365 days: Exposure class XS3: (a) target period fg=50; years (b) target period ft=100.

evolution of concrete properties over time in addition to the variable time f_0 with regard to the testing age. This combination may affect in an unrealistic way the results of compositions with little performance improvement when tested at curing ages lower than 1 year.

Some of the predictions for exposure class XS3 do not reach 10 years of lifetime of which part does not even reach 1 year of lifetime. These predictions may be considered as unrealistic for RC elements with a concrete cover over 40 mm, although other authors using models with the same basis and different input variable values also present such low lifetime results for concrete compositions with ordinary and lime Portland cement (Bertolini, 2011; Ferreira, 2004). Additionally, some of the important RC structures in marine environment in Portugal have shown poor performance as to corrosion less than 10 years after construction, with a concrete cover of 40—50 mm (Costa & Appleton, 2002).

The modelling equation for the initiation period is based on the process of diffusion, even though in the environment corresponding to tidal or splash zones (class XS3), the absorption takes the most important part as regards chloride penetra- tion, which may also explain such a disparity of lifetime results between exposure classes XS1 and XS3. Such a difference may also be explained by the mean values defined in the Portuguese specification for the chloride threshold C — 0.4% and 0.3% for classes XS1 and XS3, respectively — which according to some references are seen as very low leading thus, eventually, to very low predictive values of lifetime.

It is also important to take into account that high modelling lifetime values — such as over 120—150 years— might be far from the reality of what existing RC structures have demonstrated so far and it may be, therefore, insufficient knowledge of the considered parameters or of the modelling equations. Additionally, the original modelling proposed by LNEC E465 is defined for the implementation of potential diffusion tests at the age of 28 days, as a reference age. This study aimed at taking further the implementation of this modelling with tests carried out at different ages. This was implemented despite the fact that the coefficient of difhision *D(t)* decreases over time due to different ageing effects. Equation (4), using testing results by of NT Build 492, only reflects the ageing effect regarding the binder continued reactions. This is why it is considered that the potential coefficient of diffusion from short-term laboratory on young specimens or deriving from young structures results in overdesign.

Another aspect that is worth to mention is the possibility that the criteria associated with failure due to corrosion may have a significant influence on the modelling results. In the studied cases, the modelling approach defined by E465 (2007) sets the lifetime limit as the end of the initiation period, not taking into account the onset of corrosion or the crack formation due to corrosion, i.e. different stages in the propagation period.

6. Conclusions

The concrete compositions in this study were designed to conform to the requirements of the NP EN 206-1 **(2005)** for RC structures with a working life of 50 and 100 years resisting steel corrosion in environments with the presence of chlorides. From the NP **EN 206-1** for RC structures and the results obtained from the testing programme and the lifetime analysis, the following conclusions may be drawn:

- According to the experimental results of the three analysed parameters — compressive strength, chloride potential diffusion coefficient and electrical resistivity — it is clear that lower compressive strength does not necessarily mean lower performance with respect to either chloride potential diffusion or electrical resistivity(higher potential diffusion and lower resistivity). It depends strongly on the type of binder. Furthermore, for FA blended compositions, there is a significant improvement in performance regarding these parameters over time, when compared to Portland cement compositions. This may be related to pozzolanic and associated filler effects of these FA mixtures.

- Among the concrete compositions studied here, it is evident that the best global lifetime performance is achieved with 50'Żo clinker and 50'Żo FA (CEM I + 50'ŻoFA) for both exposure classes XS1 and XS3. Concrete composition CEM II + 50%FA (with only 35% clinker, 15% of limestone filler and 50% FA) also performed better than compositions without FA for curing ages of 90, 180 and 365 days.
- Comparing the design, lifetime results f with both target periods I of 50 and 100 years for exposure class XS1, with the compositions studied here following the requisites of the prescriptive specification, only the FA blended compositions reach those target periods and in most cases for curing ages from 90 days on. In the case of exposure class XS3, none of the design values came close to approaching the target periods.
- Regarding the development of the performance over time, there is a gradual increase in the properties for the compositions with FA blended binders from 28 to 365 days of age. This shows the importance of considering the pozzolanic effect in the definition of the acceptance parameters of concrete compositions in marine environment. Compositions without FA (CEM I and CEM II/B-L) present design lifetime results where there is a reduction of performance in function of the curing age, which is not coherent with the test results of compressive strength, chloride potential diffusion and electrical resistivity, where there is improvement or at worst, stagnation of values. This aspect might be due to the simultaneous effect of the variables *n* (ageing factor) and f,, (curing age at which tests are carried out). This requires reflection and warrants further discussion.
- Considering some questionable assumptions by specification E465, such as the mean values attributed to the chloride threshold and the parameters $k_D k_D g$ *and* k_D affecting the diffusion coefficient, it is possible that some of the results are not realistic in view of the state of the art.

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