Full probabilistic service life prediction and life cycle assessment of concrete with fly ash and blast-furnace slag in a submerged marine environment: a parameter study

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Abstract: Nowadays, more attention is being paid to sustainability in construction. Over the years, the concrete research community has developed a wide range of potential 'green' concretes. To reduce cement related CO_2 emissions, a considerable part of the traditional binder can be replaced with industrial by-products. However, as a result of the current focus on comparative durability assessment based on accelerated tests, sufficient knowledge on the actual service life and sustainability of these materials is still lacking. In this paper, we combined both approaches for concrete exposed to chlorides. Different mixes were subjected to a rapid chloride migration test. With the results obtained, probabilistic service life prediction was done. This service life together with the material's strength was used as input for life cycle assessment of an axially loaded column. Results show that the environmental impact of fly ash and slag concrete is less than half the impact of traditional concrete.

Keywords: service life prediction; life cycle assessment; LCA; concrete; fly ash; FA; blast-furnace slag; BFS; marine environment.

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1 Introduction

Since the aim for sustainability has become a major objective in our modern society, policymakers are encouraging scientists and technologists from all industrial branches, also the construction industry, to look for more environmentally friendly products. Given the considerable amount of Portland cement related CO_2 emissions – around 800 kg per ton (Josa et al., 2004) – more strong and durable concrete compositions with a lower cement content need to be developed. Partial replacement of the cement with industrial by-products such as fly ash (FA) or blast-furnace slag (BFS) has always been seen as a promising way to reduce concrete's environmental impact (Meyer, 2009). However, to date high cement replacement levels are not yet generally accepted because their impact on the concrete durability is still insufficiently known. True, quite some research has already been done to characterise the durability properties of both FA and BFS concrete (Malhotra and Mehta, 2005; Cox and De Belie, 2007; Baert et al., 2008; Lammertijn and De Belie, 2008; Van den Heede et al., 2011; Gruyaert et al., 2009, 2010, 2012). Yet, these investigations mostly comprised a mere comparative durability assessment simulating different deterioration mechanisms (carbonation, chloride and sulphate penetration, freeze-thaw, etc.) which not always allowed for an adequate service life prediction. Now, to evaluate the sustainability of a potentially 'green' concrete composition correctly, some reliable notion of the latter aspect is absolutely imperative. Certainly, it may be possible to design a concrete composition for which less CO_2 is emitted during the production process. If this composition is not so durable in its environment, rehabilitation actions (repair, replacement) will be necessary over time which will bring along additional concrete manufacturing and CO_2 emissions. In other words, when looking at both the production and use phase of the material, service life prediction counts as an undispensible input to the calculation of concrete's environmental benefit using the life cycle assessment (LCA) methodology, cf., ISO 14044 (ISO, 2006). In previous work, this procedure was already successfully applied to calculate an environmental score for high-volume fly ash (HVFA) concrete exposed to carbonation (Van den Heede and De Belie, 2010, 2011; Van den Heede et al., 2010). With the current paper, we tried to reduce this existing lacuna by estimating the service life of both FA and BFS concrete permanently exposed to chlorides and calculate their global warming potential (GWP) accordingly using the LCA framework.

Firstly, these concrete mixes were subjected to non-steady state migration tests, cf., NT Build 492 (1999) to evaluate whether the performance of FA and slag concretes can be considered as equivalent to ordinary Portland cement (OPC) concrete (regular comparative durability assessment). This is the appropriate methodology to follow when one wants to apply concrete compositions with high amounts of type II additions (FA, BFS,…) in more demanding environments, e.g. exposure class XS2 (permanently submerged environments with corrosion induced by chlorides from sea water cf. Belgian standard NBN B15-100, BIN, 2008). However, as already pointed out earlier, this mere comparison between concrete mixtures is insufficient, as it gives no indication on the potential life span of the materials. Therefore, we also used the obtained chloride migration coefficients as input to the full probabilistic service life prediction model for chloride exposed concrete as prescribed in the Fib Bulletin 34 (Fib, 2006). Probabilities of failure (P_f) and reliability indices (β) were calculated for all concrete mixtures. The input parameters of the model with an important impact on the maximum service life were first assessed for the reference concrete.

In the end, the environmental impact of each mix was calculated using a four step LCA procedure. An axially loaded column was chosen as functional unit. This way, the effect of strength and durability differences between concrete compositions was incorporated in the LCA outcome.

2 Materials and methods

2.1 Concrete mixtures

In total, six concrete mixtures were manufactured (Table 1). Two of these concrete compositions are seen as a reference concrete. T(0.45)a is an OPC concrete with the minimum cement content and the maximum W/C ratio conforming to Belgian standard NBN B15-001 (BIN, 2004) for a XS2 environment. The same holds true for mix T(0.45)b. Despite the somewhat different granulometry of the sand and aggregates, and the slightly higher cement content (350 kg/m^3 instead of 340 kg/m^3) both references are quite similar. Mix T(0.45)a was used as a reference while investigating the chloride resistance of concrete with various amounts of FA. The performance of the blast furnace slag containing mixes, was evaluated in perspective with the results obtained for reference mix T(0.45)b.

Two compositions with FA were made. Mix F15 meets the requirements of the k-value concept specified by Belgian standard NBN B15-001 (BIN, 2004), as it has a maximum fly ash-to-binder (F/B) ratio of only 15% for a minimum total binder content. This concrete composition with FA can normally be applied in a XS2 environment. However, as the F/B ratio for mix F15 is rather low, the environmental benefit attributed to the partial cement replacement will be rather limited. Therefore, a HVFA composition with a 50% FA content (F50), was developed. For the same reason, two concrete mixes with high amounts of BFS were also investigated. There, the cement replacement levels amounted to 50% (S50) and 70% (S70), respectively. The same percentages were used in previous research by Gruyaert et al. (2009, 2010, 2012).

Note that the HVFA mix design (F50) is characterised by a rather high total binder content (450 kg/m^3) and a very low water-to-binder (W/B) ratio (0.35). The authors made this particular choice to obtain an acceptable strength class after 28 days. Given the slow pozzolanic reaction of the FA, it will take a longer time to have a full strength development of the concrete. An increase of the binder content and a decrease of the W/B ratio, will increase the strength at early age. However, as can be seen from Table 1, the latter approach did not result in a strength class equal to the one of reference T(0.45)a. Other strategies to improve the early age strength performance of HVFA concrete (e.g., the addition of silica fume as ternary binder), are for the moment still under investigation. The same goes for the concrete compositions containing 50% and 70% BFS (S50 and S70), where the whole mechanical optimisation still needs to be done.

Table 1 Mix proportions and properties of the tested concrete mixtures

		T(0.45)a	*T(0.45)b*	*F15*	*F50*	*S50*	*S70*
1	Sand 0/4 (kg/m^3)	715	781	696	645	781	781
2	Aggregate. 2/8 (kg/m^3)	515	619	502	465	619	619
	Aggregate 8/16 (kg/m^3)	671	480	654	606	480	480
3	CEM I 52.5 N (kg/m^3)	340	350	317.6	225	175	105
4	Fly ash (kg/m^3)	-	-	56	225	-	-
5	Slag (kg/m^3)	-	-	-	-	175	245
6	Water (kg/m^3)	153	157.5	153	157.5	157.5	157.5
7	SP (ml/kg B)	3.0	1.4	2.0	4.0	2.9	2.9
	W/B (-)	0.45	0.45	0.41	0.35	0.45	0.45
	F/B or S/B (%)	0	0	15	50	50	70
	Slump	S4	S3	S4	S4	S3	S3
	Strength class	C45/55	C45/55	C40/50	C35/45	C30/37	C25/30

2.2 Curing and sample preparation

For the research on concrete with FA, 6 cubes with a 150 mm side were cast per concrete mix. After casting, the cubes were kept at a constant temperature and relative humidity (RH) of 20°C and 95%, respectively. Demoulding took place the next day whereupon the specimens were stored again under the same conditions until the age of 21 or 84 days. At that time, a core with a diameter of 100 mm was drilled out of each cube. Three of them were used for a rapid chloride migration test at the age of 28 days, while the other three were subjected to the same test after 91 days. The outermost 10 mm, containing the cast surfaces, were cut from the two ends of each core in correspondence with NT Build 492 (1999). Starting from these saw cuts, two cylindrical specimens with a thickness of 50 mm were taken from each core. As a result, six cylinders (n = 6) could be obtained for the accelerated chloride test at each testing age.

For the experimental program on concrete mixes with high amounts of BFS, four cubes with a 150 mm side were cast per concrete. They were cured under the same conditions as the FA mixes. Demoulding also took place the next day, except for the 70% slag mix. Those cubes were demoulded after two days because of the lower strength. The drilled cores were cut into three cylindrical specimens with a thickness of 50 mm, without cutting of the outermost 10 mm. This means that for each test four cylindrical specimens with one cast surface and one cut surface, and two specimens with two cut surfaces were used (n = 6). Whenever a cast surface was present, that surface was exposed to the NaCl solution during the rapid chloride migration test. The different concrete mixtures were tested after 28 days and 56 days of curing.

2.3 Rapid chloride migration test

The resistance to chloride penetration was evaluated experimentally using the rapid chloride migration test as described in NT Build 492 (1999). This method enables the calculation of a non-steady state chloride migration coefficient for a certain concrete mix. The following test procedure was adopted. First, the cylindrical specimens were vacuum saturated in a 4 g/l $Ca(OH)_2$ solution. After 18 ± 2 hours of immersion in this solution, the specimens were fixed inside silicon rubber sleeves with a 0.3 N NaOH (anolyte) solution on top. The bottom surface of the samples in the sleeves was brought in contact with a 10% NaCl solution (catholyte). Then, an external electrical potential was applied axially across each cylinder, which forces the chloride ions to migrate into the specimens. After a certain test duration (usually about 24 hours) the specimens were removed from the sleeves and split axially, whereupon a 0.1 M silver nitrate solution was sprayed onto the freshly split sections. When the white silver chloride precipitation had become clearly visible, the penetration depth was measured from the centre to both edges at intervals of 10 mm. From the chloride ingress obtained, a non-steady state migration coefficient can be calculated using the simplified formula of NT Build 492, using a predetermined chloride concentration (= 0.07 N) at the colour change boundary (1):

$$D_{nssm} = \frac{0.0239(273+T)L}{(U-2)t}\left(x_d - 0.0238\sqrt{\frac{(273+T)Lx_d}{U-2}}\right) \tag{1}$$

where D_{nssm}, U, T, L, x_d and t represent the non-steady state migration coefficient ($\times\ 10^{-12}$ m^2/s), the absolute value of the applied voltage (V), the average value of the initial and final temperatures in the anolyte solution (°C), the thickness of the specimen (mm), the average value of the penetration depths (mm) and the test duration (h), respectively.

2.4 Evaluation of the equivalent performance concept

Concrete mixes with 50% or 70% of the cement replaced by FA or slag are not in correspondence with the k-value concept, cf., Belgian standard NBN B15-001 (BIN, 2004). As a consequence, these concrete mixes can normally not be used in any of the more demanding exposure classes. Therefore, the Belgian standard NBN B15-100 (BIN, 2008) was developed. It provides a methodology for the assessment and the validation of the fitness for use of cements or additions of type II (e.g., FA, BFS,…) for concrete. According to this standard, high-volume fly ash or slag concretes can be applied in more severe environments when proof of an equivalent performance compared to the proper reference concrete exists. Regarding the resistance to chloride penetration, this proof can result from a diffusion test or alternatively from a non-steady state migration test. For this paper, the latter test method was used. The applicable evaluation criterion for equivalent performance states that the D_{nssm} values for the tested concretes should not exceed 1.4 times the value obtained for the reference concrete (Belgian standard NBN B15-100, BIN, 2008).

2.5 Service life prediction

The Fib Bulletin 34 (Fib, 2006) is a design code providing the necessary models for a full probabilistic prediction of the concrete's service life. In general, this design approach consists of defining a suitable limit state equation (2) containing the necessary load and resistance variables for the deterioration mechanism under investigation, in this case chloride induced corrosion of the steel reinforcements:

$$C_{cr} = C_0 + \left(C_{S,\Delta x} - C_0\right) \cdot \left[1 - erf \frac{d - \Delta x}{2 \cdot \sqrt{D_{app,C} \cdot t}}\right] \tag{2}$$

with

C_{cr}	the critical chloride content (wt.-%/binder)
C_0	the initial chloride content (wt.-%/binder)
$C_{S,\Delta x}$	chloride content at depth Δx (wt.-%/binder)
d	concrete cover (mm)
Δx	depth of the convection zone (mm)
t	time (years)
$erf(.)$	error function
$D_{app,C}$	apparent coefficient of chloride diffusion through concrete (mm^2/years).

The latter coefficient can be obtained from the experimental non-steady state migration coefficient using equation (3):

$$D_{app,C} = \exp\left(b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_{real}}\right)\right) \cdot D_{RCM,0} \cdot k_t \cdot \left(\frac{t_0}{t}\right)^a \tag{3}$$

with

b_e	a regression variable (K)
T_{ref}	the standard test temperature (K)
T_{real}	the temperature of the structural element or the ambient air (K)
$D_{RCM,0}$	the non-steady state chloride migration coefficient (mm^2/years)
k_t	a transfer parameter (-)

t_0 a reference point of time (years)

t time (years)

a the ageing exponent (-).

A combination of (2) and (3) enables an estimation of the time to depassivation in an environment with exposure to chloride induced corrosion. Table 2 gives a quantification of all the input parameters which are normally used in the model: distribution function, mean values and standard deviations.

Table 2 Quantification of the input parameters for the limit state equation defined by (2) and (3)

	Distribution	*Mean*	*Stdv.*	*Lower bound*	*Upper bound*
C_{cr} (wt.-%/binder)	Beta	0.6	0.15	0.2	2.0
C_0 (wt.-%/binder)	Constant	0	-	-	-
$C_{S,\Delta x}$ (wt.-%/binder)	Normal	3.0	0.8	-	-
d (mm)	Lognormal	40	8	-	-
Δx (mm)	Constant	0	-	-	-
b_e (K)	Normal	4,800	700	-	-
T_{ref}(K)	Constant	293	-	-	-
T_{real} (K)	Normal	283	5	-	-
$D_{RCM,0}$ (mm²/yrs)	Normal	Figure 1	Figure 1	-	-
k_t (-)	Constant	1	-	-	-
t_0 (yrs)	Constant	0.0767 (28d)	-	-	-
		0.1534 (56d)	-	-	-
		0.2493 (91d)	-	-	-
a (-)	Beta	0.30 (OPC)	0.12	0.0	1.0
		0.60 (FA)	0.15	0.0	1.0
		0.45 (BFS)	0.20	0.0	1.0

Probabilities of failure (P_f) and reliability indices (β) were calculated using the first order reliability method (FORM) available in the COMREL software. In compliance with Fib Bulletin 34 (Fib, 2006), these parameters need to meet the requirements for the depassivation limit state ($P_f \leq 0.10$ and $\beta \geq 1.3$) to qualify for use in a XS2 environment.

With a minimum cement content of 340 kg/m^3 and a maximum W/C ratio of 0.45, the reference concrete type T(0.45) specified in Belgian standard NBN B15-001 (BIN, 2004), should have a minimum service life of at least 50 years (Belgian standard NBN B15-001, BIN, 2004, European standard NBN EN 206-1, CEN, 2000). This 50 years time span is the indicative design service life for common buildings and other structures. An indication of the minimum concrete cover specified for these infrastructures, can be found in Eurocode 2. A XS2 environment normally requires a cover of at least 40 mm (CEN, 2005).

First, a preliminary service life prediction was performed for the OPC references T(0.45)a and T(0.45)b to check whether the minimum 50 years service life can be confirmed by the probabilistic design method. In addition, the effect of assuming different values for the concrete cover (d, Table 2), the ageing exponent (a, Table 2) and the critical chloride content (C_{cr}, Table 2) was investigated. Since a certain range of values for these parameters can be found in literature, this kind of parameter study is seen as very relevant. For instance, a 40 mm concrete cover is only a minimum value. Song et al. (2009) mention a design cover of 80 mm for a concrete tunnel box exposed to sea water. The ageing exponent for OPC concrete can easily range between 0.3 (Fib Bulletin 34, Fib, 2006) and 0.4 (CUR VC81, 2007). The same is true for concrete mixes containing FA (0.6–0.7) and BFS (0.45–0.5). The critical chloride content can also vary considerably. Duracrete (2000) gives much higher values than the 0.6 wt.-%/cement for permanently submerged OPC concrete structures. For W/C ratios between 0.5 and 0.4, C_{cr} values should range between 1.6 and 2.1 wt.-%/cement. This means that a value of about 1.9 wt.-%/cement can be assumed for reference T(0.45) with a W/C ratio of 0.45.

Once the definitive values for d, a and C_{cr} were agreed upon for the T(0.45) reference, the probabilistic service life prediction was also done for the concrete mixes with FA (F15, F50) and BFS (S50, S70). For each of these compositions, the probabilities of failure (P_f) and the reliability indices (β) were investigated for a time frame of 100 years. The same was also done now for the references T(0.45)a and T(0.45)b. This time period was considered as more appropriate for permanently submerged concrete in a marine environment, since the material is then usually incorporated in bridges or other civil engineering structures. There, the 100 years design service life usually counts as a minimum (Fib Bulletin 34, Fib, 2006).

Finally, it should be noted that the ageing exponents and the critical chloride contents used for the service life prediction of each concrete mix remain values taken from literature. In other words, they are not mix specific. The different ageing exponents apply to OPC, FA and BFS concrete in general and not to the exact concrete compositions under investigation. Moreover, the choice of the critical chloride content is only related to the W/C ratio of the concrete and no distinction in value was made between the different concrete types. Preferably, as many input parameters to the model as possible are mix specific and it is possible to determine them. To estimate the actual ageing exponents per concrete mix, it would be necessary to perform chloride diffusion tests on specimens directly taken from structures located in a submerged marine environment or after the immersion of specimens in realistic sea water solutions. Only if one has obtained the corresponding chloride diffusion tests at minimum three different ages, it is possible to quantify the ageing exponent by means of non-linear regression analysis. Additional experiments could also be performed to determine the critical chloride content per concrete mix. Therefore, specimens with embedded steel reinforcement bars would need to be manufactured on which the corrosion potential of the steel reinforcement bars is measured as a function of the Cl^- concentration near the steel bars. However, within the early stages of optimising potentially ‘green’ concrete types these experiments are often not yet performed, because they take quite some time. Instead, it is easier to subject the initial concrete mix designs to the less time consuming chloride migration tests and use the default values of the Fib model as preliminary input to get a first impression of the service life. Based on these results, the mix designs first need to be improved further on, before starting up a more detailed experimental program in which all input parameters of the service life prediction model are measured specifically for each of the fully optimised mix designs. A first attempt to measure mix specific ageing exponents for FA concrete was recently done by Van den Heede et al. (2012). For the BFS concrete, this research is ongoing. Mix specific C_{cr} values for both concrete types also still need to be determined.

2.6 Life cycle assessment

In correspondence with ISO 14040, the LCA consisted of four major steps: the definition of goal and scope, the inventory analysis, the impact analysis and the interpretation (ISO, 2006).

2.6.1 Definition of goal and scope

This LCA was conducted to quantify the environmental benefits of partially replacing OPC with FA. Special attention was paid to its influence on the reduction of greenhouse gas emissions (GHGs), since this is the reason why concrete mixes with considerable amounts of FA and BFS were developed in the first place.

Since both the strength and durability aspects were included in this LCA study, an axially loaded column carrying a design load of 1,500 kN for 100 years was chosen as the functional unit. The experimental strength classes given in Table 1 were used for the column design. All calculations regarding concrete and steel reinforcement dimensioning were done in accordance with Eurocode 2 (CEN, 2005). Logically, a higher concrete strength class reduces the column dimensions. Since more or less the same amount of reforcement steel was used in every column, its environmental impact was not considered in this LCA.

Table 3 Overview of the life cycle inventory data used per m^3 of each concrete mix

Material data (kg)		*T(0.45)a*	*T(0.45)b*	*F15*	*F50*	*S50*	*S70*
1	Sand, at mine/CH S	715	781	696	645	781	781
2	Gravel, round, at mine/CH S	1,186	1,099	1,156	1,071	1,099	1,099
3	Portland cement, strength class Z 52.5, at plant/CH S	340	350	317.6	225	175	105
4	Fly ash*	-	-	56	225	-	-
5	Blast-furnace slag*	-	-	-	-	175	245
6	Tap water, at user/CH S	153	157.5	153	157.5	157.5	157.5
7	Superplasticizer (EFCA, 2006)	1.12	0.54	0.82	1.98	1.12	1.12
Transport data (tkm)							
1	Transport, barge/RER S	137.8	150.5	134.1	124.3	150.5	150.5
	Transport, van < 3.5 t/RER S	1.6	1.8	1.6	1.5	1.8	1.8
2	Transport, barge/RER S	228.5	211.8	222.7	206.4	211.8	211.8
	Transport, van < 3.5 t/RER S	2.7	2.5	2.7	2.5	2.5	2.5
3	Transport, van < 3.5 t/RER S	38.4	39.6	35.9	25.4	19.8	11.9
4	Transport, van < 3.5 t/RER S	-	-	2.1	8.6	-	-
5	Transport, van < 3.5 t/RER S	-	-	-	-	18.7	26.2
6	-	-	-	-	-	-	-
7	Transport, van < 3.5 t/RER S	0.13	0.06	0.10	0.23	0.13	0.13
Processing data (kWh)							
Electricity, low voltage, production BE, at grid/BE S		3.83	3.83	3.83	3.83	3.83	3.83

Note: *Seen as waste products with no environmental impact.

2.6.2 Inventory analysis

Per concrete constituent, the necessary inventory data were collected from the Ecoinvent database (Frischknecht and Jungbluth, 2007). Their proper short descriptions as mentioned in the database together with the amounts transported and used to manufacture 1 m^3 of each concrete mix, are shown in Table 3.

In this case study, the secondary cementitious materials FA and BFS were considered as mere waste products from coal fired power plants and steel factories, respectively. Therefore, no environmental impact has been attributed to them. Inventory data regarding superplasticizers (SPs) were obtained from an environmental declaration published by the European Federation of Concrete Admixture Associations (EFCA, 2006).

The transport of the raw materials highly depends on the geographical location of the concrete mixing plant, in this case our laboratory where the research project was carried out. The search for an optimal plant location to minimise transport distances was not a goal in the current LCA study.

The processing data represent the amount of electricity (in kWh) used to mix 1 m^3 of each concrete composition (mix duration: 5 min).

2.6.3 Impact analysis and interpretation

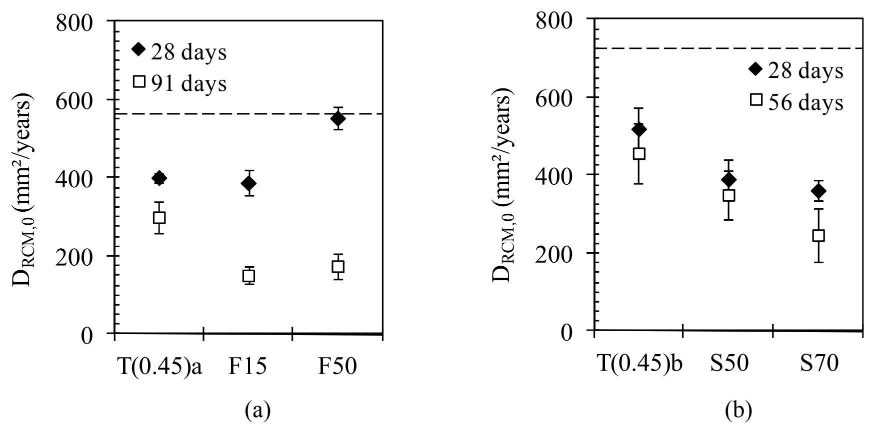
The IPCC 2007 GWP 100a impact method was adopted to calculate the concrete's GWP expressed in CO_2 equivalents for a time frame of 100 years. All calculations were done in the LCA software SimaPro 7.3.2.

3 Results and discussion

3.1 Validation of the equivalent performance concept

Figure 1 shows the non-steady state migration coefficients $D_{RCM,0}$ (mm^2/years) for both the fly ash [Figure 1(a)] and slag [Figure 1(b)] containing concrete mixtures in comparison with their proper references.

Figure 1 Non-steady state migration coefficients $D_{RCM,0}$ (a) for T(0.45)a, F15 and F50 and (b) for T(0.45)b, S50 and S70



With a FA content of 15% (F15), the migration coefficient obtained after 28 days of curing is very similar to the value recorded for the reference. Evidently, this kind of performance was expected, since mix F15 was designed in correspondence with the k-value concept of Belgian standard NBN B15-001 (BIN, 2004). Mix F50 on the other hand, is clearly less resistant to chloride penetration after 28 days than reference T(0.45)a. However, an evaluation of equivalent performance should be based on the test criterion mentioned in Belgian standard NBN B15-100 (BIN, 2008). The dashed line in the graph shows that the result for F50 is similar to 1.4 times the value of the reference after 28 days of curing. In other words, proof of an equivalent performance really exists for this concrete mix, at least after 28 days of optimal curing. Moreover, prolonged curing at a high RH seems to have a very beneficial effect on the chloride resistance of the FA containing concretes. For both F15 and F50, the decrease of the migration coefficient between 28 and 91 days of curing is more than twice the decrease obtained for T(0.45)a. Given the obvious difference between F15 and F50 at the age of 28 days, it is most remarkable that the 91 days migration coefficient for the two mixtures has become similar.

The concrete mixes with 50% (S50) and 70% (S70) slag show a quite different behaviour [Figure 1(b)]. In contrast with the FA containing concrete mixtures, the migration coefficient decreases when the slag content increases, whatever the curing age. The effect of prolonged curing is also clear from these results, especially for S70. However, the decrease in migration coefficient is not as pronounced as for the FA mixtures. S50 and S70 are clearly more resistant to chloride penetration than T(0.45)b and meet the test criterion mentioned in Belgian standard NBN B15-100 (BIN, 2008).

3.2 Service life prediction

3.2.1 Parameter study

Table 4 presents the results of the parameter study conducted on the two reference concrete mixes. It shows how different values for the concrete cover d (40, 60 and 80 mm), the critical chloride content C_{cr} (0.6 and 1.9 wt.-%/cement) and the ageing exponent a (0.3 and 0.4), can affect the outcome of the service life (t_{SL}) prediction. The life spans mentioned in the table have probabilities of failure and reliability indices which are just in agreement with the limiting values ($\beta = 1.3$ and $P_f \approx 0.10$) for the depassivation limit state. When applying the minimum concrete cover of 40 mm and the Fib Bulletin 34 (Fib, 2006) values for C_{cr} (0.6 wt.-%/cement) and a (0.3), the estimated service life would be maximum 2 years. This observation is in sharp contrast with European standard NBN EN 206-1 (CEN, 2000), that mentions a life span of at least 50 years for a T(0.45) concrete mix when located in a XS2 environment. An increase of the concrete cover does not seem to bring satisfactory improvements. When assuming an ageing exponent of 0.4 instead of 0.3, the estimated service life is somewhat longer, yet still far below 50 years. Choosing a higher value for the critical chloride content has more important consequences. When C_{cr} is equated with 1.9 wt.-%/cement, a value corresponding with the Duracrete (2000) guidelines for submerged OPC concrete structures, in combination with a 60 mm concrete cover, the service life tends towards 50 years (42–62 years). More concrete cover can increase the life span to more than 100 years. With an ageing exponent of 0.4, a 100 years life span could already be ensured with 60 mm concrete on top of the steel reinforcements. The settings which resulted in a service life of around

50 years for the references were used further on for the service life prediction of the FA and slag containing concrete mixtures. This means that the concrete cover and the critical chloride content amount to 60 mm and 1.9 wt.-%, respectively, and that the ageing exponent is in agreement with the Fib Bulletin 34 (Fib, 2006) guidelines for the different concrete types (OPC: 0.3, FA: 0.6, BFS: 0.45). The question arises whether it is appropriate to assume the same C_{cr} value for concrete with FA and BFS. Extensive literature study conducted by Angst et al. (2009) shows that this is still an ongoing discussion. Some authors state that the presence of FA and slag lowers the critical chloride content (Thomas, 1996; Gouda and Halaka, 1970; Oh et al., 2003) while others claim exactly the opposite (Schiessl and Breit, 1996). On the other hand, a non significant effect was reported by Alonso et al. (2002) and Oh et al. (2003) for FA and slag, respectively. Keeping all contradictory findings in mind, the same critical chloride content [1.9 wt.-%/binder, cf., Duracrete (2000)] was adopted for all studied concrete mixes. Nevertheless, further investigation on the actual C_{cr} value for each concrete type is strongly recommended.

Table 4 Influence of the applied concrete cover, critical chloride content and ageing exponent on the estimated service life t_{SL} of the references T(0.45)a and T(0.45)b

d (mm)	*C_{cr} (wt.-%/c)*	*a (-)*	*T(0.45)a*		*T(0.45)b*	
			$t_{SL\ 28days}$ *(yrs)*	$t_{SL\ 91days}$ *(yrs)*	$t_{SL\ 28days}$ *(yrs)*	$t_{SL\ 56\ days}$ *(yrs)*
40	0.6	0.3	2	2	1	1
60			6	7	5	5
80			14	15	10	10
40	0.6	0.4	3	3	2	2
60			12	11	8	8
80			29	27	20	19
40	1.9	0.3	19	20	14	14
60			61	62	43	42
80			>100	>100	90	88
40	1.9	0.4	43	39	30	27
60			>100	>100	>100	93
80			>100	>100	>100	>100

3.2.2 Service life prediction

Figure 2 presents the outcome of the probabilistic service life prediction for reference T(0.45)a and the FA containing concrete compositions F15 and F50.

On the left hand side, the probabilities of failure are plotted for the 100 years time frame. On the right hand side, the corresponding reliability indices are shown with an indication of the limiting value ($\beta = 1.3$) for the depassivation limit state. As can be seen from graphs 2(a) and 2(b), a 100 years service life is ensured for the mixes F15 and F50 based on the non-steady state migration coefficient obtained after 28 days of curing. When the 91 days value for $D_{RCM,0}$ is used as input to the model, the same conclusions can be drawn [Figure 2(c) and Figure 2(d)]. The much lower probabilities of failure for

F15 and F50 after 91 days [Figure 2(c), detail] are attributed to the further densification of the pore structure with time due to the pozzolanic FA reaction and possible chloride binding. A 100 years service life does not exist for reference T(0.45)a, since the β value already drops below 1.3 after 61–62 years. As a consequence, the axially loaded column will need to be replaced once within the 100 years time period in order to guarantee the predefined service life for bridges and other important civil engineering structures situated in a marine environment.

Figure 2 Probabilities of failure (a, c) and reliability indices (b, d) for T(0.45)a, F15 and F50, calculated from the non-steady state migration coefficients obtained after 28 (a, b) and 91 days (c, d)

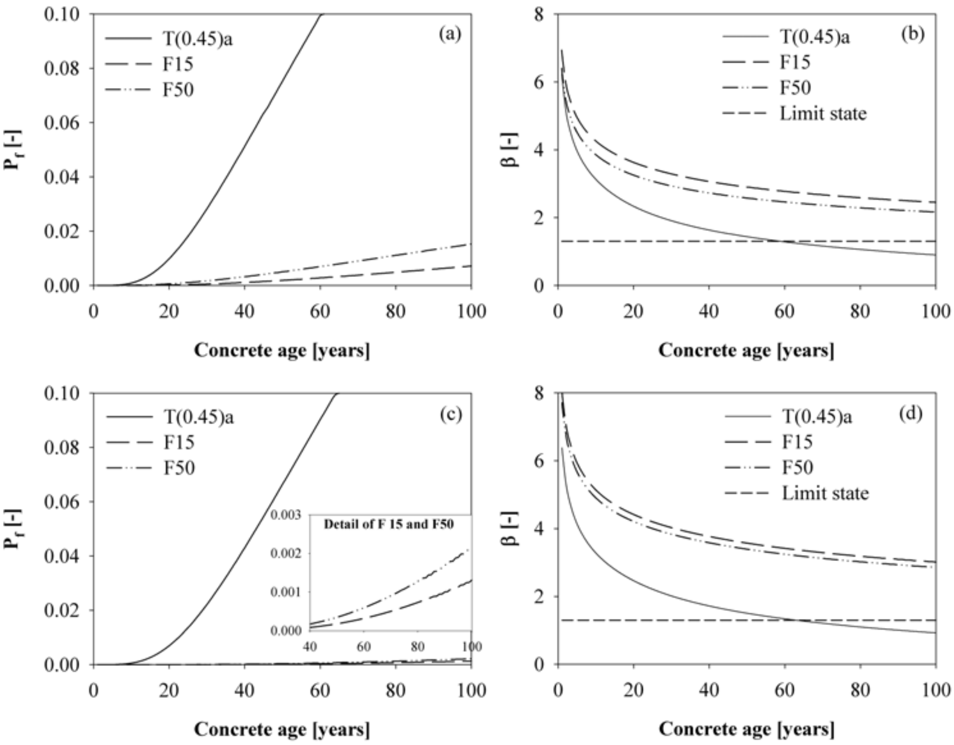
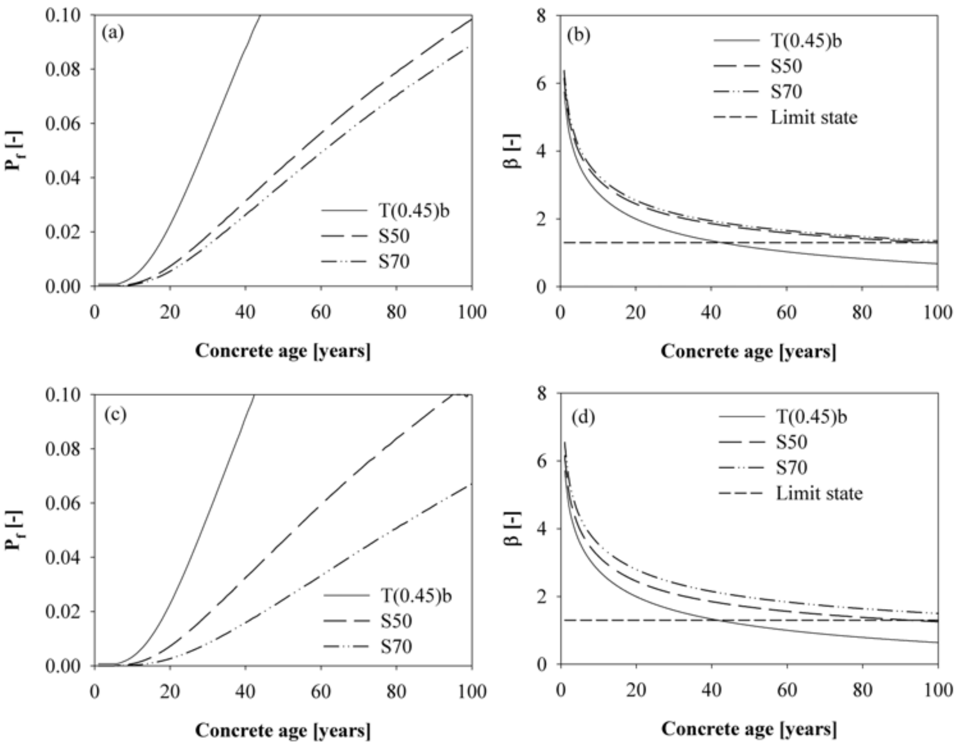


Figure 3 shows the results of the probabilistic service life prediction for reference T(0.45)b and for the slag concrete mixes S50 and S70. The graphs are presented in the same way as in Figure 2. From Figures 3(a) and 3(b) it is clear that a service life of 100 years is ensured for mixes S50 and S70 based on the non-steady state migration coefficient obtained after 28 days of curing. Using the 56 days value for the migration coefficient [Figure 3(c) and Figure 3(d)], the same conclusions can only be drawn for S70. For S50, a service life of 100 years is not ensured since the limiting value is already exceeded after 93 years. Compared with the FA mixtures, the probabilities of failure and the reliability indices approach the limiting values for the depassivation state at an earlier age. The β value for reference T(0.45)b drops below 1.3 after 42–43 years. This means that the axially loaded column will need to be replaced twice within the 100 years time period.

Figure 3 Probabilities of failure (a, c) and reliability indices (b, d) for T(0.45)b, S50 and S70, calculated from the non-steady state migration coefficients obtained after 28 (a, b) and 56 days (c, d)



Note that the outcome of the probabilistic service life prediction for both FA and slag concrete can vary significantly with the chosen ageing exponent. Normally, higher values are used for these concrete types (0.6 for FA and 0.45 for BFS versus 0.3 for OPC), and this because of their better performance at later age. An evaluation of the correctness of these ageing exponents from the Fib Bulletin 34 (Fib, 2006) was not a goal in this study. However, it is worthwhile investigating in future research. For a given concrete composition, this would require the determination of the chloride migration coefficients at more than two ages and a non-linear regression analysis based on the values obtained as such.

3.3 Life cycle assessment

Table 5 shows the amount of concrete needed to manufacture the axially loaded column (design load 1,500 kN) based on the strength classes mentioned in Table 1, and the corresponding GWP of each concrete volume expressed in kg $CO_{2\,eq}$.

Logically, a higher strength class of the concrete reduces the column dimensions. It explains why only 0.15 m^3 was needed for the OPC reference (C45/55) column, while 0.27 m^3 was required in case of the S70 (C25/30) column. When the concrete durability aspect is not taken into account, the GWP scores in kg $CO_{2\,eq}$ are quite similar for all concrete mixes. However, the probabilistic service life prediction for t_0 = 0.0767 years

(28 days) already showed that 2 and 3 times the concrete volume will be necessary for reference T(0.45)a and T(0.45)b to achieve the required service life of 100 years. The act of replacement increases the environmental impact substantially. The impact of T(0.45)a is then about twice the impact of the mixes with FA. In comparison with the S50 and S70, the impact of T(0.45)b is more than tripled. Note that a comparison between the FA and slag containing concrete compositions is not recommended. From Table 5, it seems now that the use of slag concrete is clearly more beneficial since the impact is only about one third of the GWP of the reference. This is only true when a 100 years service life is considered. A comparison between Figure 2 and Figure 3 clearly show that very low probabilities of failure were obtained for the concrete compositions with FA after 100 years, which indicates that an even longer service life can be achieved with mixes F15 and F50. This additional beneficial effect was not further considered for this paper.

Table 5 Required volume (m^3) and the corresponding GWP (kg $CO_{2\ eq}$) per concrete mix

Mix	*T(0.45)a*	*F15*	*F50*	*T(0.45)b*	*S50*	*S70*
Volume (m^3)	*2* × 0.15	0.17	0.19	*3* × 0.15	0.22	0.27
GWP (kg $CO_{2\ eq}$)	*2* × 58 (116)	63	54	*3* × 60 (180)	56	52

4 Conclusions

In conclusion, to quantify the environmental benefit of concrete with high amounts of FA or slag correctly, a full assessment of its strength and durability is necessary. An evaluation of the latter aspect should be based upon test methods that enable not only a mere comparison between concrete mixtures, but also estimation for the service life of each mix. This twofold approach resulted in the following findings:

- Based on the results of the rapid chloride migration tests, an equivalent performance (≤ 1.4 times the reference value) compared to the applicable reference could be confirmed for the concrete compositions with 15% and 50% FA, and this after 28 days of curing. With prolonged curing (91 days), the chloride resistance of these compositions increased even more (at least × 2). For the slag containing concrete mixes, the migration coefficients obtained after 28 days were already lower than the values measured for the OPC concrete. This reduction clearly increases with an increasing cement replacement level.
- A parameter study prior to the service life prediction showed that some of the input parameters (concrete cover, critical chloride concentration and ageing exponent) to the Fib Bulletin 34 have an important influence on the estimation of the concrete's life span. To obtain a normal service life of approximately 50 years for the reference concrete mixes, a concrete cover of 60 mm, a critical chloride concentration of 1.9 wt.-%/cement and an ageing exponent of 0.3 were used as input parameters for the full probabilistic service life prediction. For the FA and slag concrete mixes, an ageing exponent of 0.6 and 0.45 was used, respectively.
- For the FA containing concrete mixes, a service life of 100 years is ensured when using the 28 days migration coefficient as well as the 91 days migration coefficient as input parameter.

- For the slag mixtures, a service life of 100 years is ensured when the 28 days migration coefficient is used. Using the 56 days migration coefficient, a service life of 93 years is obtained for the concrete mix with 50% slag while the service life of the concrete mix with 70% slag exceeds 100 years.
- LCA of an axially loaded column (design load: 1,500 kN, service life: 100 years), shows that the environmental impact of the compositions with FA is about half the impact of the OPC reference (~ 57 kg $CO_{2\ eq}$ on average versus 116 kg $CO_{2\ eq}$). The impact of the mixes with BFS is only one third of the reference impact (~ 54 kg $CO_{2\ eq}$ on average versus 180 kg $CO_{2\ eq}$). This beneficial effect regarding concrete sustainability is mainly attributed to the longer service life estimated for the concrete mixes with industrial by-products.

The future research perspectives are twofold. First of all, further attempts should be made to improve the early age strength of both FA and BFS concrete through the incorporation of ternary binders. Since concrete's strength controls the dimensioning of the structure and thus the amount of concrete manufacturing needed, it is imperative to obtain mechanical properties for FA and BFS concrete which are similar to OPC concrete. Then, a detailed service life prediction needs to be performed for the optimised concrete compositions where every relevant input parameter to the Fib Bulletin 34 model (ageing exponent, critical chloride content, etc.) is experimentally determined for each mix.

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