CHALMERS

Evaluation of Durability of Concrete with Mineral Additions with regard to Chloride-Induced Corrosion

TANG LUPING
INGEMAR LÖFGREN

Department of Civil and Environmental Engineering
Division of Building Technology
Building Materials
CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden, 2016

Report No. 2016-4

REPORT

Evaluation of Durability of Concrete with Mineral Additions with regard to Chloride-Induced Corrosion

TANG LUPING
INGEMAR LÖFGREN

Department of Civil and Environmental Engineering
Division of Building Technology
Building Materials
CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden 2016

Evaluation of Durability of Concrete with Mineral Additions with regard to Chloride-Induced Corrosion
TANG LUPING, INGEMAR LÖFGREN

Report / Department of Civil and Environmental Engineering, Chalmers University of Technology 2016-4
ISSN 1652-9162

Department of Civil and Environmental Engineering
Division of Building Technology
Building Materials
Chalmers University of Technology
SE-412 96 Gothenburg
Sweden
Telephone: +46 (0) 31-772 1000
http://www.chalmers.se

Evaluation of Durability of Concrete with Mineral Additions with regard to Chloride-Induced Corrosion

Abstract

This report presents the results from a study of modelling of chloride ingress and corrosion initiation in concrete with Portland cement and with fly ash and ground granulated blast-furnace slag as mineral additions. Durability of concrete with mineral additions and general models for chloride ingress in concrete are briefly reviewed. Chloride threshold values for corrosion initiation are discussed. The ClinConc model was employed to model the chloride ingress profiles after exposure under marine and road environments for 100 years. The model was validated using the field data after exposure in the Swedish seawater for about 20 years. The results show that the addition of mineral additions in general increases the resistance of concrete to chloride ingress. In consideration of both chloride resistance and alkalinity, the concrete with mineral additions still reveals sufficient margin to allow a significantly lower chloride threshold for initiation of corrosion of reinforcement steel in concrete. Based on the results from this study, some values of minimum cover thickness are suggested for different exposure environments depending on different types of binder used in concrete to achieve a service life of 100 years.

Keywords: Cement, concrete, chloride, corrosion, durability, mineral addition, service life

Sammanfattning

I denna rapport presenteras resultat från en studie av modellering av kloridinträngning och korrosionsinitiering i betong med portlandcement och med flygaska och mald granulerad masugnsslagg som mineraliska tillsatsmaterial. Beständigheten hos betong med tillsatsmaterial och allmänna modeller för kloridinträngning i betong granskas kort. Kloridtröskelvärden för korrosionsinitiering diskuteras. ClinConc-modellen användes för att modellera klorid inträngningsprofiler efter exponering i marin och vägmiljöer i 100 år. Modellen validerades med fältdata från exponering i det svenska havsvattnet i ca 20 år. Av resultaten framgår att tillsatsmaterial i allmänhet ökar betongens motstånd mot kloridinträngning. Med hänsyn till både kloridmotstånd och alkalinitet ger betong med tillsatsmaterial fortfarande tillräcklig marginal för att tillåta ett signifikant lägre kloridtröskelvärde för korrosionsinitiering. Baserat på resultaten från denna studie föreslås en del värden för minsta täckande betongskikt för olika exponeringsmiljöer beroende på olika typer av bindemedel som används i betong för att uppnå en livslängd på 100 år.

Nyckelord: Betong, beständighet, cement, klorid, korrosion, livslängd, mineraliska tillsatsmaterial

Preface

Nowadays as a worldwide tendency more and more mineral additions are used as cementitious replacement materials in concrete in order to reduce the emission of carbon dioxides and improve carbon footprint in concrete production. For reinforced concrete structures exposed to severe environments it is essential to understand the long term performance of concrete with mineral additions. This study was carried out with such a purpose, on the request of Thomas Concrete Group AB (TCG), as a part of Trafikverket project financially supported by the Swedish Transport Administration.

As the first author I appreciate the pleasant collaboration with Prof Ingemar Löfgren. I also acknowledge Dr Anders Lindvall at TCG for his constructive comments and suggestions in reviewing the draft report.

Tang Luping

March 2016

Contents

1 Introduction

1.1 Background

Chloride induced reinforcement corrosion is still a big durability problem of reinforced concrete structures such as bridges and tunnels in road infrastructures, especially in Sweden due to its long coastal line and cold climate with intensive use of de-icing salt. At the present, the specification of durability is mainly based on the establishment of various constraints to the mixture proportions of the concrete, such as cement type, water-binder ratio, and entrained air-content, together with requirements on the cover thickness as function of the severity of the exposure. This approach does not consider the actual performance of concrete materials with different types of cement and mineral additions added to the cement or directly to the concrete.

As towards the sustainable development of society and construction industry, it is a tendency to use more and more cementitious replacement materials such as fly ash (FA) and ground granulated blast-furnace slag (GGBS) in concrete mixtures in order to reduce the emission of carbon dioxides and improve carbon footprint in concrete production. Therefore, performance-based service life design and calculations are desired instead of “deem-to-satisfy” rules. With the help of more sophisticated durability models more robust structures can be designed with expected service life and reduced consumption of materials. This report intends to evaluate the service life of reinforced concrete with binders blended with FA and GGBS regarding chloride-induced corrosion of reinforcement steel, based on the current knowledge and available models. Moreover, the aim is to provide recommendations with respect to requirements on minimum concrete cover for different concrete compositions, with main focus on bridges and tunnels with a service life of 120 years.

1.2 Scope of this study

The purpose of the work presented in this report was to investigate the effects of binders blended with FA and GGBS on chloride ingress in concrete and chloride-induced corrosion of reinforcement steel. The investigation work includes the following parts:

- Brief literature review with focus on the important influencing factors related to the resistance of concrete against chloride ingress and threshold value.
- Modelling of service life of selected types of concrete for two types of structures, submerged in the seawater (on Swedish west and east coast, respectively, as exposure class XS2 in the European standard SS-EN 206-1) and exposed to splashing salt water such as columns and side beams of a road bridge (as exposure class XD3 in the European standard SS-EN 206-1).
- Discussion of the modelled results with regard to the uncertainties of the influencing factors, as well as with the minimum covers specified in the current building regulations.
- Providing recommendations for minimum concrete covers for road and marine environments for different concrete compositions

1.3 Limitations of this study

This study is limited to mineral additions of FA and GGBS with regard to chloride induced corrosion of reinforcement in concrete. The other durability problems such as carbonation and freeze-thawing attack will not be dealt with.

2 Literature review

This section presents some models for chloride ingress, threshold values and their important influencing factors.

2.1 Chloride ingress in concrete

Due to the global problems in chloride-induced corrosion of reinforcement in concrete structures, the topic of chloride ingress in concrete has been intensively studied in the past decades. An overview of durability of steel reinforced concrete in chloride environments was given by Shi et al. (2012). In this overview the role of mineral additions such as FA, silica fume, GGBS and metakaolin, was also reviewed. In general, concrete with mineral additions reveals excellent resistance to chloride ingress in concrete by significant reduction of chloride diffusivity. In the review by Shi et al. (2012) different test methods for chloride diffusivity and challenges in assessing the durability of concrete from its chloride diffusivity were also discussed.

With regard to models of chloride ingress, many models can be found in the literature. Some typical models were summarized by Tang et al. (2011). According to Tang et al. (2011), the models can be categorized into two groups: empirical and mechanistic. Empirical or semi-empirical models often assume a diffusion process but use total chloride content as the driving force. Obviously, taking the total chloride content as driving force in the transport function is theoretically questionable, because it is only the free chloride ions that can move in the pore solution and contribute to chloride-induced corrosion of reinforcement in concrete. Therefore, mechanistic models often use the free chloride as the driving force and take the non-linear chloride binding into account. Some often used or mentioned models in the literature are briefly presented as follows.

The Simple ERFC Model
The abbreviation ERFC denotes the mathematical symbol erfc - ERror Function Complement. This model was first proposed in the early 1970's for modelling of chloride ingress in concrete (Collepardi M., et al., 1972[3]). The model uses an erfc solution to Fick's 2nd law of diffusion under the semi-infinite boundary condition:

$$C(x,t) = C_{\mathrm{i}} + \left(C_{\mathrm{s}} - C_{\mathrm{i}}\right) \cdot \mathrm{erfc}\left(\frac{x}{2 \cdot \sqrt{D_{\mathrm{a}} \cdot t}} \right) \tag{1}$$

where: C_{i} = the initial chloride content in the concrete (sometimes this chloride content is negligible), C_{s} = the surface chloride content, x = the depth, D_{a} = the apparent diffusion coefficient and t = the exposure duration. In this model the parameters C_{s} and D_{a} are assumed constant during the whole period of exposure.
In this model the key parameters are C_{s} and D_{a}, which have to be determined by curve-fitting of the chloride ingress profiles from field or laboratory exposure. It has been proven from many experimental data that this simplest model can only describe the chloride ingress under the exposure conditions for a short duration close to the conditions for which the input parameters are determined. As explained later, several modifications to equation (1) were proposed to try to expand the applicability range of such type of models.

Mejlbro-Poulsen's Model
This mathematical model was developed in Denmark through Danish national project HETEK in the middle of 1990's (Frederiksen J.M., et al., 1996). The model assumes the total chloride as the driving force, and considers both the surface chloride content C_{s} and the apparent diffusion coefficient D_{a} as time-dependent functions, that is,

$$D_{\mathrm{a}} = D_{\mathrm{aex}} \left(\frac{t_{\mathrm{ex}}}{t + t_{\mathrm{ex}}} \right)^{\alpha} \tag{2}$$

and

$$C_s = C_i + S(D_a \cdot t)^p \tag{3}$$

where: D_{aex} = the apparent diffusion coefficient at the time of exposure t_{ex}, α, S and p are the constants. An analytic solution to Fick's 2nd law with time-dependent C_s and D_a was given in the form as follows (Mejlbro L. 1996[5]):

$$C(x,t) = C_i + (C_s - C_i) \cdot \Psi_p\left(\frac{x}{2\sqrt{D_a \cdot t}}\right) \tag{4}$$

where: Ψ = a series of Γ-functions (Gamma-functions). When $p = 0$, equation (4) becomes the same form as equation (1). Because it is impossible to measure D_{aex} at the time of exposure (no ingress profile is available for curve-fitting), the model needs some experimental data from short term exposures (e.g. 1 year exposure and at a later storage) to estimate the values of D_{aex}, α, S and p. Updating from later available infield data may result in different values of these empirical parameters (Frederiksen and Geiker 2000; 2008).

DuraCrete Model

The DuraCrete project (Engelund S., et al., 2000) recommended the following equation to express the apparent diffusion coefficient in equation (1):

$$D_a = k_{e,cl} \cdot k_{c,cl} \cdot D_{RCM,0} \cdot \left(\frac{t_0}{t + t_{ex}}\right)^{n_{cl}} \tag{5}$$

where: $D_{RCM,0}$ = the chloride migration coefficient measured by e.g. the Nordtest method NT BUILD 492 (Nordtest, 1999), at the age t_0 = 28 days, $k_{e,cl}$ and $k_{c,cl}$ are constants considering the influence of environment and curing, respectively, on chloride ingress, t_0 is the reference period (concrete age of 28 days) at which $D_{RCM,0}$ is measured and n_{cl} is the age factor describing the time-dependency of the apparent diffusion coefficient.

It can be seen that a difference between equations (2) and (5) is that D_{aex} in the former is from the same field exposure conditions as D_a, while $D_{RCM,0}$ in the latter is from the laboratory conditions which is different from the actual exposure conditions. The model uses an empirical factor of $k_{e,cl}$ to try to bridge the gap between the laboratory and the field conditions. Again, it needs a lot of qualified infield data to establish the proper values of $k_{e,cl}$ for actual service life design. The principle of the DuraCrete model was also adopted in a guidance of Concrete Society (Bamforth, 2004) and the *fib* model code for service life design (*fib*, 2006), in which the factor $k_{e,cl}$ is specified by the following equation:

$$k_{e,cl} = \exp\left[b_e\left(\frac{1}{273+T_{ref}} - \frac{1}{273+T_{real}}\right)\right] \tag{6}$$

where: b_e = the regression variable varying between 3500 and 5500, with 4800 as the mean value and 700 as standard deviation, T_{ref} = the reference temperature in °C at which the chloride migration coefficient is measured and T_{real} = the real exposure temperature in °C.

ACI Life 365 Model

In North America, an empirical model has been developed by ACI TC-365 in the beginning of 2000's (Thomas and Bentz, 2001). The model utilizes Fick's 2nd law of time-dependent diffusion as the transport function with the total chloride content C as the driving force:

$$\frac{\partial C}{\partial t} = D(t)\frac{\partial^2 C}{\partial x^2} \tag{7}$$

where

$$D(t) = D_{ref}\left(\frac{t_{ref}}{t}\right)^m \tag{8}$$

where D_{ref} = the apparent diffusion coefficient at the reference time of exposure t_{ref} and m = the constant. In order to prevent the diffusion coefficient indefinitely decreasing with time, the relationship shown in equation (7) is only valid up to 30 years. Beyond this time, the value at 30 years (D_{30y}) calculated from equation (7) is assumed to be constant throughout the rest of the analysis period. The temperature effect on the apparent diffusion coefficient has also been taken into account in this model. The model selects the rate of chloride build-up and the maximum surface content based on the type of exposure (and structure) and the geographic location. The model also gives various α values for different additions of puzzolanic materials. Obviously, the model is semi-empiric. On the other hand the software of the ACI model has integrated chloride ingress, initiation and propagation of corrosion, repair schedule, and life-cycle costs together, so as to give the user a simple tool for maintenance planning of concrete structures. A numerical approach is followed in the model for the time integration. Therefore, special software is needed for application of the model.

An important difference that often confuses the readers/users of the models, is that, $D(t)$ in equation (8) is different from D_a in equations (2) and (5). The former is the instantaneous diffusion coefficient as conventionally defined in Fick's law, while the latter is the average diffusion coefficient during the period from t_{ex} to $(t + t_{ex})$ (Poulsen, 1993).

ClinConc Model

The model ClinConc (Cl in Concrete) was first developed in the middle of 1990's (Tang and Nilsson, 1994; Tang, 1996). The ClinConc model consists of two main procedures: 1) Simulation of free chloride penetration through the pore solution in concrete using a genuine flux equation based on the principle of Fick's law with the free chloride concentration as the driving potential, and 2) Calculation of the distribution of the total chloride content in concrete using the mass balance equation combined with non-linear chloride binding. Obviously, the ClinConc model uses free chloride as the driving force and takes non-linear chloride binding into account. Thus it describes chloride transport in concrete in a more scientific way than the empirical or semi-empiric models. Later, this model has been expressed in a more engineer-friendly way (Tang, 2006; 2008) so as to make it possible for applications by practising engineers. The free chloride concentration in the concrete at depth, x, is determined using the following equation:

$$\frac{c-c_i}{c_s-c_i}=1-\mathrm{erf}\left(\frac{x}{2\sqrt{\frac{\xi_D D_{6m}}{1-n}\cdot\left(\frac{t_{6m}}{t}\right)^n\cdot\left[\left(1+\frac{t_{ex}}{t}\right)^{1-n}-\left(\frac{t_{ex}}{t}\right)^{1-n}\right]\cdot t}}\right) \tag{9}$$

where: c, c_s and c_i = the concentration of free chlorides in the pore solution at depth x, at the surface of the concrete and initially in the concrete, respectively, D_{6m} = the diffusion coefficient measured by the RCM test, e.g. NT BUILD 492, at the age of t_{6m}, ξ_D is the factor bridging the laboratory measured D_{6m} to the initial apparent diffusion coefficient for the actual exposure environment, n is the age factor accounting for the diffusivity decrease with age, t_{ex} is the age of concrete at the start of exposure and t is the duration of the exposure.

Different from the empirical models, the factors ξ_D and n in the ClinConc can be calculated based on the physical properties of concrete including cement hydration, hydroxide content, water accessible porosity, time-dependent chloride binding, and the environmental parameters such as chloride concentration and temperature. The detailed descriptions of the factors ξ_D and n are given in (Tang, 2006).

The total chloride content is basically the sum of the bound chloride, c_b, and free chloride, c, expressed as:

$$C=\frac{\varepsilon\cdot(c_b+c)}{B_c}\times 100 \qquad \text{mass\% of binder} \tag{10}$$

where ε is the water accessible porosity at the age after the exposure, B_c is the cementitious binder content, in kg/m$^3_{concrete}$, and c_b, is the bound chlorides expressed as the same unit as free chloride. The bound chlorides can be calculated by different equations, e.g. Langmuir (Sergi et al., 1992), Freundlich (Tang and Nilsson, 1993), or other regression equations. The detailed descriptions of the use of Freundlich equation for calculation of bound chlorides are given in (Tang, 2006).

It should be noticed that the ClinConc model was primarily developed for the submerged environment, where the chloride solution is constantly in contact with the concrete surface. When the model is used for the atmospheric zone or the road environment, certain modifications are needed.

Both the DuraCrete and the ClinConc models use the diffusion coefficient measured by the RCM test, e.g. NT BUILD 492, as an input parameter, but care should be taken that this parameter is tested at different concrete ages, i.e. 28 days in the former while 6 months in the latter. Among the above mentioned models, only the ClinConc model treats the material properties and the exposure environment in a separate way.

Other Models

Besides the above mentioned models, there are many other models based on either empirical equations or physical and chemical/electrochemical processes. Some of them are more or less similar to the Life-365 or the DuraCrete model, whilst some of them need special software such as the STADIUM® to calculate the complicated mathematical iterations (e.g. Samson et al., 1999; Truc et al., 2000; Johannesson, 2000; Meijers, 2003; Petre-Lazar et al., 2003; Nguyen et al., 2006). More information about various models can be found in (Tang et al. 2011). There are also various numerical models such as Multi-Environmental Time Similarity (METS) model (Jin and Xu, 2010), LIFEPROD model (Andrade et al., 2014), etc.

With regard to the factors influencing chloride ingress, an analytical evaluation of the key parameters involved in the error-function based models has been made by Tang et al. (2012). According to their analysis, the age factor in some models is the most sensitive one among all the input parameters.

2.2 Chloride-induced corrosion of reinforcement steel

It is well known that reinforcement steel in concrete is normally in a passive condition thank to the alkalinity of cementitious pore solution which facilitates the formation of a thin passive film on the surfaces of steel. When chloride ions penetrate into concrete and reach the steel surface at a certain concentration, the passive film is broken down and pitting corrosion is initiated. The exact mechanism of the passive film breakdown by chloride ions is not clearly understood yet although it can simply be considered as a function of the net balance between two competing processes; stabilization and repair of the film by OH^- ions, and disruption of the film by Cl^- ions (Bentur et al., 1997). It is, however, generally accepted that the active corrosion (depassivation) occurs when the chloride concentration reach a certain critical level, referred as the chloride threshold value C_{cr} (Bentur et al., 1997; Bertolini et al., 2004; Angst et al. 2009). The chloride threshold value depends on many parameters. Comprehensive literature reviews on the subject has been made by Angst et al. (2009) and Alonso et al. (2009), showing large scatter in the reported chloride threshold values with one order of magnitude (from 0.1% up to around 2% by mass of binder) found in the literature.

There are many factors influencing C_{cr}. One of the decisive factors is the pH value of the pore solution which is strongly dependent on the type of binder (Bentur et al., 1997; Angst et al. 2009), because the passive film is formed and maintained under the alkali condition or the concentration of hydroxide ions. Hausmann (1967) suggested a Cl^-/OH^- of 0.6 whilst Diamond (1986) proposed 0.3 as a threshold chloride concentration, which will further be discussed in section 3.2.1. In both cases this quantitative data are based on the experiments in solutions. For reinforcement steel embedded in concrete additional factors such as moisture content, temperature, oxygen availability, defects on the concrete-steel interface are also important. Usually C_{cr} is expressed as the total or acid soluble chloride. In this case, the chloride binding capacity of cementitious hydrates has to be taken into account.

It should be noted that the large scatter in the reported data has partially be attributed to the variability in the testing methods (Angst et al., 2009; Alonso et al., 2009). According to Angst et al. (2009), the scatter in the C_{cr} data from the field investigations is relatively smaller than that from the laboratory studies. Frederiksen (2000) summarized the reported data and suggested an equation to the design values for chloride threshold level C_{cr}, that is,

$$C_{\mathrm{cr}} = K_{\mathrm{cr,env}} \exp\left(-1.5\mathrm{eqv}\left(\frac{w}{c}\right)_{\mathrm{cr}}\right) \qquad \text{mass\% of binder} \tag{11}$$

where $K_{cr,env}$ is the environmental factor and $\mathrm{eqv}\left(\frac{w}{c}\right)_{\mathrm{cr}}$ is the equivalent water-cement ratio with regard to chloride threshold.

The equivalent water-cement ratio was expressed as

$$\mathrm{eqv}\left(\frac{w}{c}\right)_{\mathrm{cr}} = \frac{W}{Cf_{\mathrm{p}}} = \frac{W}{C\left(1+\sum k_{\mathrm{p}}M_{\mathrm{p}}\right)} \tag{12}$$

where W and C are the content of mixing water and cement, respectively, f_p describes the effect of puzzolanic additives, k_p is the activity factor of puzzolanic additives with regard to chloride threshold, and M_p is the mass ratio of puzzolanic additive to cement. Frederiksen (2000) suggested the values of k_p -4.7 for silica fume and -1.4 for FA, but no value available for GGBS. It is noticed that this negative k_p value can dramatically increase the equivalent water-cement ratio, especially when the replacement level is high. For instance, when the fraction of FA is 41.6% of the binder, corresponding to the M_p value of 71.4%, the equivalent water-cement ratio will tend to infinite with k_p = -4.7, leading to C_{cr} = 0. This is, of course, not true. Therefore, the applicable range of Eq. (11) should be limited to maximum 10% silica fume and 20% FA in the binder, as the data Frederiksen (2000) based the equation on.

Again, it is the free chloride ions that contribute to chloride-induced corrosion of reinforcement in concrete. According to Hausmann (1967) and Diamond (1986) less chloride is needed to break down the passive film with a lower alkalinity. It is conventionally believed that the mineral addition in concrete results in lower chloride threshold value because of the pozzolanic reactions which consume $Ca(OH)_2$ from the cement hydration, resulting in a lower pH value in the pore solution (Tuutti, 1982). This is still questionable, because the initial pH (13-14) of the pore solution is mainly attributed to the alkaline oxides K_2O and Na_2O, as expressed by equivalent [Na_2O] in the binder whilst the long-term pH is dependent on the existence of portlandite in the hardened cement paste. According to Gruyaert (2011), concrete with 85% GGBS still kept some 2% portlandite by mass of binder after hydration for one year while the concrete with 50% GGBS contained more than 9% portlandite by mass of binder after hydration for 3 years. It is known that calcium leaching is a process much slower than chloride ingress. If there is no carbonation, very little amount of portlandite can keep the pH value of solution about 12.5 due to its low solubility (0.023 mol/l). However, the pH-value is not the only thing that influences the resistance against corrosion initiation. On the other hand, the higher chloride binding capacity, lower diffusivity and finer pore structure of concrete with mineral addition positively contribute to the resistance of concrete against corrosion initiation, as indicated in a study of reinforced concrete specimens after over 20 years' exposure in the Träslövsläge harbour (Boubitsas et al., 2014).

3 Modelling and Discussion

3.1 Modelling of chloride ingress

Based on the validation results from the concrete specimens after over 20 years' exposure in the Träslövsläge harbour (Boubitsas et al., 2014), the ClinConc model revealed the best agreement with the field data. Therefore, this model was used for modelling of chloride ingress in this study.

3.1.1 Concrete mixes

Concrete mixes designed and tested by TCG were primarily used for modelling. Six types of cement, two types of GGBS and one type of FA from different manufacturers in Europe were used in the study. The standard notations and available compound compositions of cement are listed in Table 1. The physical properties and available chemical compositions of cement and mineral additions are listed in Table 2. The concrete mixture proportions are listed in Table 3, where the aggregate was a granitic type, the superplasticizer (SP) was Glenium 51/18, and the air entraining agent (AEA) was MicroAir 100 (1:10). In all the mixes the air content was 4.5% by volume of concrete. No efficiency factors (*k*-factors) was used in the mixes with mineral additions. The additions of FA or GGBS were related to the total binder content by mass, e.g. "C2 + 20%GGBS" means that the binder is composed of 80% cement type C2 and 20% GGBS by mass.

Table 1. Standard notations and compound compositions of cement[1)]

ID	*Cement type according to EN 197-1*	*C_3S %*	*C_2S %*	*C_3A %*	*C_4FA %*
C1	CEM I 42.5 N MH/LA/SR3	57.8	18.5	2.2	13.7
C2	CEM I 42.5 N MH/LA/SR3			1.1	
C3	CEM I 52.5 N	56.0	15.8	6.7	8.9
C4	CEM III/A 42.5 N/NA				
C5	CEM II/A-V 42.5 N MH/LA/NSR			3.0	
C6	CEM II/B-S 52.5 N			5.3	

1) According to the manufacturers' information C4 contains about 49% GGBS; C5 is based on the clinker of C1 and contains about 13% FA; and C6 contains about 33% GGBS.

Table 2. Physical properties and chemical compositions of cement and mineral additions[1)]

Mineral ID	*Density* kg/m^3	*Blaine* m^2/kg	*CaO* %	*SiO_2* %	*Al_2O_3* %	*Fe_2O_3* %	*MnO* %	*SO_3* %	*Cl^-* %	*Na_2O_{eq}* %
Cement C1	3200	330	64	22	3.7	4.5	0.9	2.6	0.01	0.51
C2	3160	330	64	22	3.3	4.6	0.7	2.5	0.07	0.45
C3	3140	420	63	19	4.3	3.1	3.4	3.4	0.07	0.90
C4	3000	450	52	28	8.9	1.2	3.1	3.2	0.08	0.70
C5	3040	370						3.0	0.01	0.85
C6	3060	460	56	25	6.3	2.1	4.0	3.1	0.07	0.80
GGBS S1	2900	420	40	35	12	0.7	7.0	0.2	0.01	1.20
S2	2920	500	31	34	13	0.3	16.5	0.3	0.01	0.90
Fly ash FA	2100	[2)]	<5						0.01	2.4

1) According to the manufacturers' information GGBS and FA fulfil the requirements of SS-EN 15167-1 and SS-EN 450-1, respectively, although some values of chemical compositions are not available.

2) The fineness of FA is expressed by 16% fraction of particles of size ≥45 µm.

Table 3. Mix proportions of various types of concrete, kg/m^3 (if not otherwise stated)

Mix ID	*Binder type*	*Cement content*	*GGBS or (FA)*	*Water-binder ratio*[1)]	*Aggre-gates*	*SP, % by mass of binder*	*AEA, % by mass of binder*
Ref 1	C1 0.45	400		0.45	1710	0.50	0.20
Ref 2	C2 0.45	400		0.45	1705	0.50	0.20
Ref 3	C3 0.45	400		0.45	1703	0.50	0.20
#1	C2 +20%S1 0.45	320	80	0.45	1699	0.63	0.25
#2	C2 +30%S1 0.45	280	120	0.45	1696	0.71	0.29
#3	C2 +40%S1 0.45	240	160	0.45	1693	0.83	0.44
#4	C2 +60%S1 0.45	160	240	0.45	1687	1.25	0.50
#5	C1 +20%S1 0.45	320	80	0.45	1687	0.63	0.25
#6	C1 +40%S1 0.45	240	160	0.45	1703	0.83	0.33
#7	C6 0.45	400		0.45	1679	0.50	0.20
#8	C4 0.45	400		0.45	1688	0.50	0.20
#9	C2 +20%S2 0.45	320	80	0.45	1700	0.63	0.30
#10	C2 +40%S2 0.45	240	160	0.45	1693	0.83	0.53
#11	C5 0.45	400		0.45	1704	0.50	0.27
#12	C1 +20%FA 0.45	335	(84)	0.44	1674	1.45	1.05
#13	C5 0.40	425		0.40	1708	0.60	0.25
#14	C1 +20%FA 0.40	350	(87.5)	0.40	1686	1.45	1.05
#15	C1 +25%FA 0.40	350	(115)	0.39	1636	1.45	1.05
#16	C2 0.40	425		0.40	1709	0.60	0.25
#17	C2 +20%S1 0.40	340	85	0.40	1703	0.75	0.25
#18	C2 +30%S1 0.40	298	128	0.40	1700	0.86	0.29
#19	C2 +40%S1 0.40	255	170	0.40	1697	1.00	0.33
#20	C4 0.40	425		0.40	1690	0.70	0.17
#21	C6 0.40	425		0.40	1700	0.70	0.17

1) Binder here means the mass sum of cement and mineral additions without consideration of efficiency coefficient (k = 1).

The compressive strength tested in accordance with EN 12390-3 and the chloride migration coefficient measured using NT BUILD 492 at different ages are listed in Table 4.

Table 4. Compressive strength and chloride migration coefficient of concrete

Mix ID	*Compressive strength, MPa*			*Chloride migration coefficient, $\times 10^{-12}$ m^2/s*		
	28 days	*56 days*	*180 days*	*28 days*	*56 days*	*180 days*
Ref 1	45.5	52.2	58.2	17.6	14.5	13.9
Ref 2	46.9	53.7	59.2	20.0	14.9	14.6
Ref 3	42.8	48.6	50.1	10.9	9.0	8.6
#1	45.6	54.1	63.5	11.4	8.7	6.1
#2	39.4	48.8	56.0	11.5	7.2	4.3
#3	37.5	49.1	59.9	15.2	6.5	2.9
#4	37.0	48.3	66.6	8.9	4.7	2.5
#5	45.4	52.7	58.4	9.4	6.4	4.7
#6	36.8	46.3	55.5	7.6	4.1	3.8
#7	47.0	52.4	59.4	8.6	6.2	5.8
#8	50.4	57.8	66.4	5.0	3.3	2.3
#9	48.6	57.8	63.2	12.0	8.5	5.6
#10	38.7	49.1	57.9	11.1	6.1	3.6
#11	45.8	50.2	63.2	15.5	11.5	4.8
#12	38.8	46.7	-	22.8	14.0	6.2[1)]
#13	50.7	54.8	64.6	12.5	8.6	4.0
#14	45.8	53.7	-	16.9	8.9	3.0[1)]
#15	49.1	58.4	-	16.4	9.3	3.6[1]
#16	50.5	57.4	61.5	19.0	14.9	13.1
#17	50.0	57.0	66.8	12.9	7.8	5.7
#18	48.4	56.8	68.7	10.4	5.5	4.5
#19	45.6	57.0	72.6	9.3	5.3	3.5
#20	54.7	61.0	68.3	4.7	3.5	2.9
#21	59.2	61.7	68.6	6.3	4.5	4.4

1) Estimated from the data measured at 28 and 56 days using exponent time-dependent relationship.

3.1.2 Exposure conditions and input parameters

In Sweden there are west and east coasts, both are representative for XS2 and XS3 exposure conditions whilst the highway like Rv 40 with deicing salt spread in the winter is a typical road environment representative for XD3 exposure condition. The chloride concentration in Swedish east coast along the Baltic Sea is quite low, about 0.4% by mass of seawater, whilst the chloride concentrations in the seawater along Swedish west coast vary from southern to northern part due to the fact that brackish water from the Baltich Sea is transported northwardly from Öresund along the coastline. Therefore, in the modelling of the marine environment the chloride ionic concentration of 14 g/l and the annual mean water temperature of +11°C at Träslövsläge harbour were used for Swedish west coast (Tang, 2003) whilst 4 g/l and +9 °C in the southern part of the Baltic Sea were adopted for Swedish east coast (Lindvall, 2007). For the road environment the chloride ionic concentration of 1.5 g/l and the annual mean air temperature of +10 °C were applied (Tang and Utgenannt, 2007). In this study the age factor due to desiccation effect under the road environment was not taken into account because of the uncertainty of this effect.

Besides the actual data of mix proportions in Table 1 and chloride migration coefficient measured at 180 days in Table 2, the effective alkali content (expressed as $\mathrm{eqv}[Na_2O]_{\mathrm{eff}}$) of each binder was calculated by the following equation (Locher, 2006):

$$\mathrm{eqv}\left[\mathrm{Na_2O}\right]_{\mathrm{eff}} = \left[1 - 1.8\left(\frac{S}{C+S}\right)^2\right] \cdot \mathrm{eqv}\left[\mathrm{Na_2O}\right] \tag{13}$$

where eqv[Na_2O] is the equivalent [Na_2O] content in % of binder which was given by the producers of cement and/or GGBS, and calculated from individual fractions of the materials in the blended binder (C = cement and S = GBBS by mass). For the concrete with the addition of FA, only the alkali from the cement was counted in the effective alkali content whilst the alkali content from FA was ignored for a more conservative estimation.

The actual concretes in this study contained very low initial chlorides because the only source of chlorides came from the cement with maximum value of 0.08% in C4, as shown in Table 2. It is more conservative, therefore, to assume an initial total chloride content of 0.1% by mass of binder. The free chloride concentration in the pore solution can then be found out using MS Excel Solver from equation (10) and the related chloride binding capacity as described in the ClinConc model. This initial free chloride concentration is much lower than the surface chloride concentration due to high chloride binding capacity. However, the porosity of concrete reduces with time, resulting in a small variation of the initial total chloride content in the deeper depths of concrete where the external chloride does not reach. Therefore, a small modification of the model was made, that is, if the calculated total chloride content at a depth is lower than the assumed initial value, the initial value i.e. 0.1% by mass of binder will be used.

The other input parameters involved in the ClinConc model were taken or calculated in accordance with the previous studies published in Tang (2003, 2006) and summarised in Appendix A in CBI Report 2:2012 (Tang et al., 2012).

As a national application of SS-EN 1992-1 (Eurocode 2) in Sweden, the values of minimum cover for various exposure classes are specified in Boverket's Building Regulations EKS 10 by Swedish National Board of Housing, Building and Planning (Boverket, 2016). It should be pointed out that recommendations of minimum covers are also given in SS-EN 1992-1-1 (Tables 4.3N and 4.4N), but the covers specified in EKS 10 are the ones that should be used for the actual design. It should also be noted that the values for XS2 and XS3 are based on the environment of Swedish east coast with lower chloride concentration (0.4% by mass of seawater). These values will, anyhow, be used in the following sections for comparison with the modelled values.

3.1.3 Validation of the model with the filed data

Although the model ClinConc has been validated using the field data measured in concretes with various types of binder including silica fume and fly ash after exposure in the Träslövsläge seawater for over 20 years (Boubitsas et al., 2014), no concrete with GGBS binder was tested in the previous projects. Fortunately, one concrete with CEM III/B, marked as concrete type "94-1" in Boubitsas et al. (2014) was exposed there since 1994. Its chloride profile measured after exposure for 19 years was used for comparison in this study. On the other hand, it can be seen from Table 4 that the chloride migration coefficient of concrete with C1 0.45 ("Ref 1") measured at 6 months is quite similar to that with C2 0.45 ("Ref 2"), indicating that the same type of cement (CEM I 42.5 N MH/LA/SR3) has similar chloride resistance although the manufacturers are different. Therefore, the chloride profiles of concrete type "7-40" (with Anläggningscement, *w/c* 0.40) and type "8-40" (with Slite cement CEM I 42.5 R, *w/c* 0.40) measured after exposure for 21 years (Boubitsas et al., 2014) were used for comparison with concrete #16 (C2 0.40). For the concrete with 13% and 20% fly ash ("C5 0.40" and "C1 +20%FA 0.40") the modelled profile was compared with the measured one from concrete type "H8" (with Anläggningscement +20%FA, *w/c* 0.30) in Boubitsas et al. (2014). The results are shown in Figure 1, where the minimum concrete covers under exposure classes XS2 and XS3 for maximum equivalent water-cement ratio of 0.40 in EKS 10 (Table D-1) are indicated.

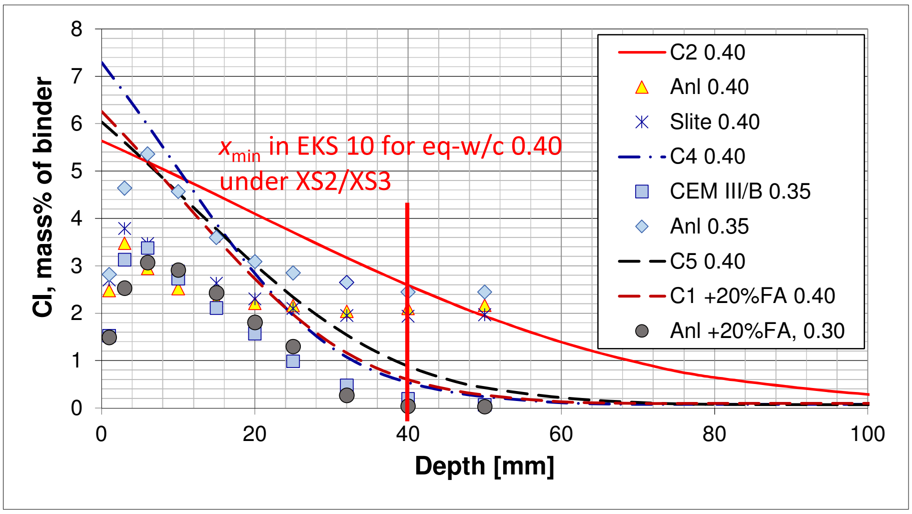

Figure 1. *Comparison of the modelled chloride profiles (lines) with the measured ones (marks).*

It is clear that after 21 years, the actual chlorides have already reached over 2% at the depth of 50 mm in the concrete with Anläggningscement at both *w/c* 0.40 and 0.35, and also in the concrete with Slite cement at *w/c* 0.40, in fairly good agreement with the modelled one with Infracement. It indicates that the ClinConc model does not overestimate chloride ingress in concrete with CEM I, but gives a reasonable prediction.

For the concrete with GGBS and FA, the measured chloride profiles are lower than the modelled ones, owing to the difference in water-binder ratio (0.30-0.35 for the measured and 0.40 for the modelled profiles) and in slag content (about 75% in CEM III/B and 49% in C4, CEM III/A) or fly ash content (about 13% in "C5" and 20% in "H8"). The higher modelled chloride profiles indicate that the ClinConc model does not underestimate the chloride ingress in the concrete with GGBS or FA but gives a reasonably conservative estimation.

Although the ingress depth of the modelled profiles agree well with the measured ones, the measured chloride contents in the near surface zone are significantly lower than the modelled ones. One of the reasons is that, in the ClinConc model, the chloride binding capacity increases with the chloride contaminating time based on the field data after exposure for 10 years (Tang, 2003). After exposure for over 20 years, calcium leaching or partial carbonation may have reduced chloride binding capacity, resulting in lower total chloride contents in the near surface zone. Further investigation is needed in order to modify the model taking into account calcium leaching or partial carbonation. This is, however out of the scope of this study.

Based on the above results two conclusions can be drawn:

- The concrete with mineral additions such as GGBS and FA has significantly better resistance to chloride ingress.
- The ClinConc model also gives reasonable prediction of chloride ingress in concrete with mineral additions including GGBS or FA binder.

3.1.4 Modelled chloride profiles from the submerged seawater condition

The field data after over 20 years' exposure at the Träslövsläge harbor (Swedish west coast) show that the chloride ingress is more severe in the submerged zone than in the other zones, although in some occasional cases the chloride ingress may be severe in the splash zone close to the water level (Boubitsas et al., 2014). Therefore, the modelling under the submerged condition (exposure class XS2) can also be used for predicting chloride ingress under the seawater splashing condition (XS3). The modelled chloride profiles after exposure in Swedish west and east coasts for 100 years are shown in Figures 2 to 5, where the minimum concrete covers under exposure classes

XS2 and XS3 for maximum equivalent water-cement ratios of 0.40 and 0.45 in EKS 10 (Table D-1) are indicated.

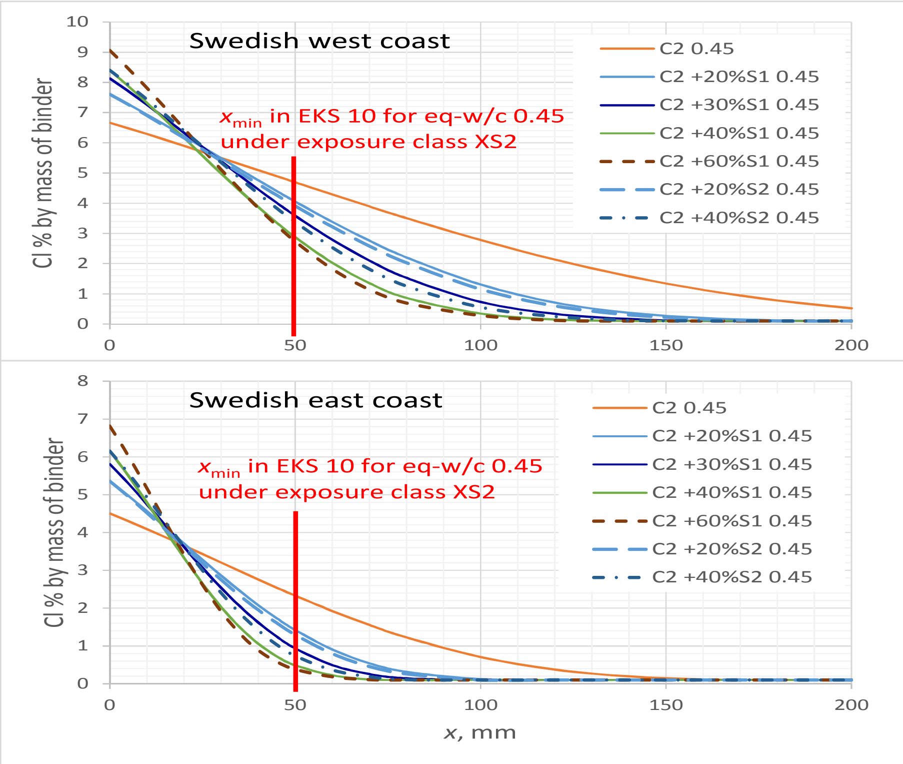

Figure 2. *Modelled chloride ingress after 100 years in concrete with cement type C2 and various additions of GGBS with w/b 0.45, under the exposure class XS2.*

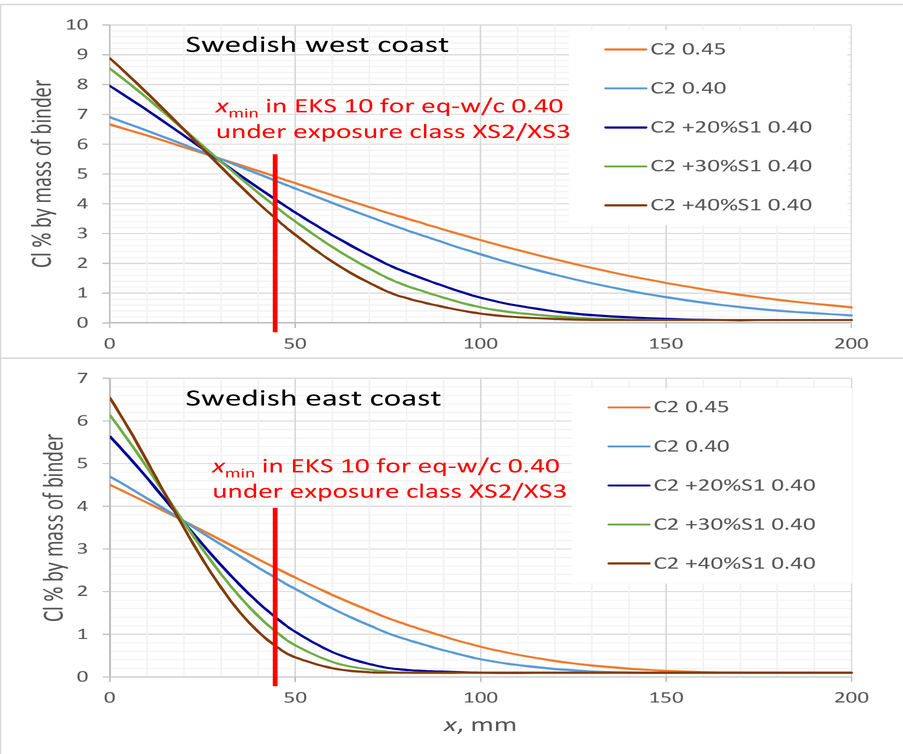

Figure 3. *Modelled chloride ingress after 100 years in concrete with cement type C2 and various additions of GGBS with w/b 0.40, under the exposure classes XS2 and XS3.*

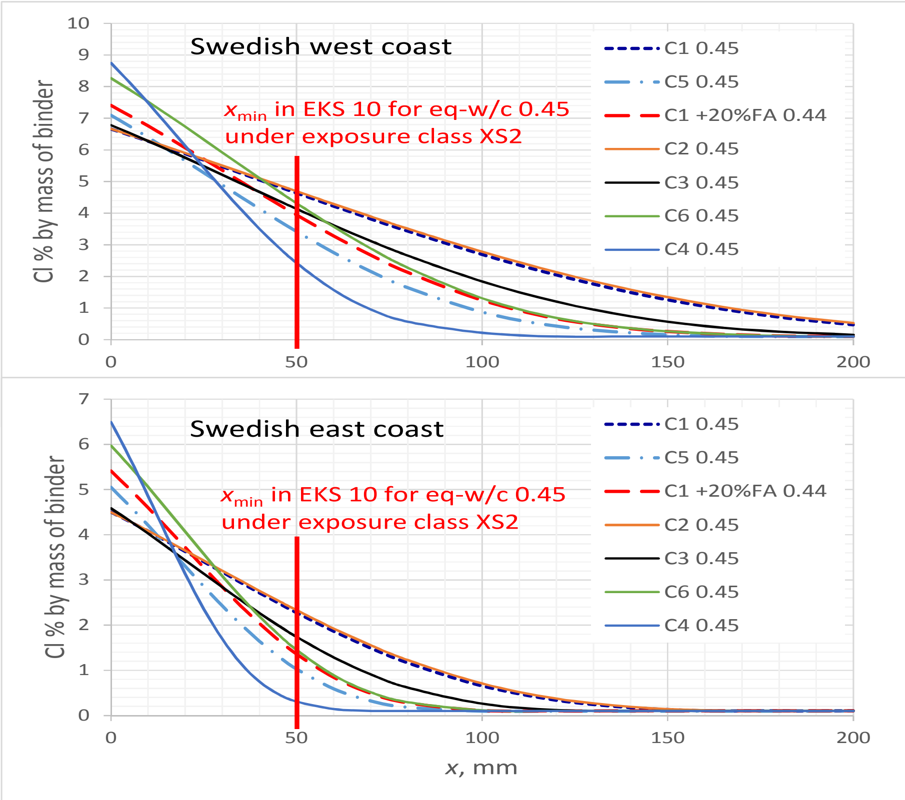

Figure 4. *Modelled chloride ingress after 100 years in concrete with different types of cement with w/b 0.45, under the exposure class XS2.*

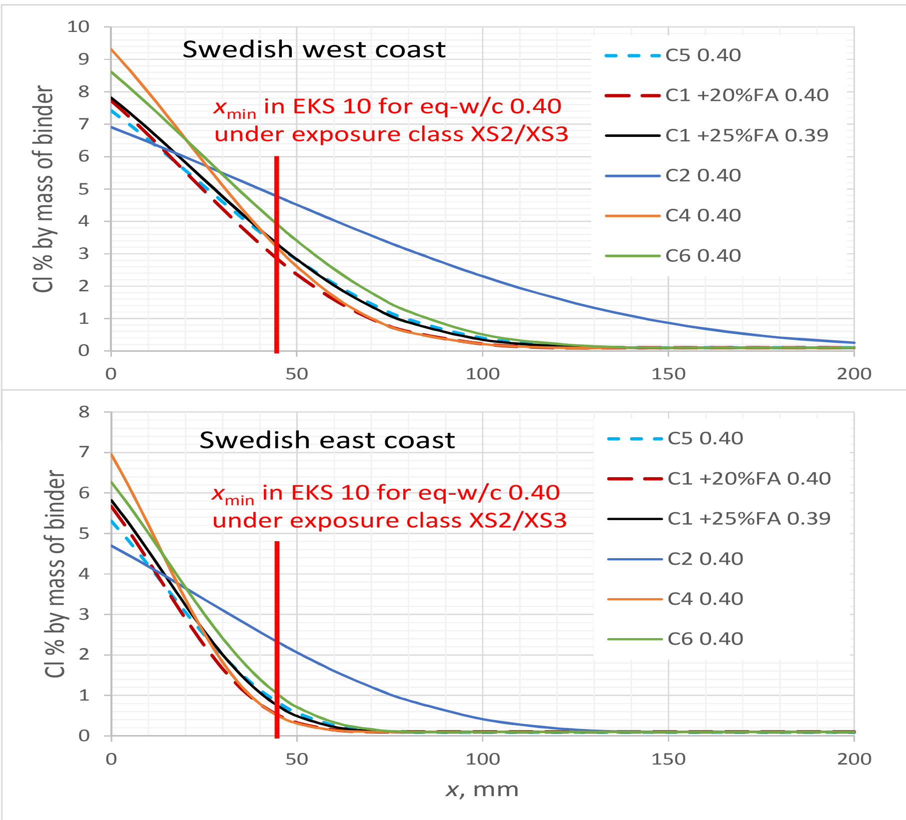

Figure 5. *Modelled chloride ingress after 100 years in concrete with different types of cement with w/b 0.40, under the exposure classes XS2 and XS3.*

It is clear that lower chloride concentration in the Swedish east coast seawater leads to lower chloride ingress depths. From the modelled results it can be seen that the chloride resistance of concrete increase with the addition of GGBS, independent of types of GGBS (Figure 2). The effect of GGBS addition by 20% is better than that of reduction of water-cement ratio from 0.45 to 0.40 (Figure 3). Addition of FA by 13-25% (cement types "C5", "C1 +20%FA" and "C1 +25%FA") has an effect similar to that of GGBS by 33-49% (cement types C6 and C4), as shown in Figures 4 and 5. Similar types of cement by different producers have similar resistance to chloride ingress.

If the chloride threshold is C_{cr} = 1.0% by mass of binder, none of the concretes in this study can achieve a service life of 100 years under Swedish west coast environment with the minimum cover 45-50 mm as specified in EKS 10 (Table D-1). Under Swedish east coast environment, only those concretes which contain 30-49% GGBS or with 13-25% FA have the possibility to achieve the required service life. It seems that all the concrete with plain Portland cement fail to achieve a service life of 100 years, even under the Swedish east coast environment, i.e. the chloride content at the specified minimum cover depth exceeds the chloride threshold value.

3.1.5 Modelled chloride profiles from the road splashing condition

The modelled chloride profiles after exposure in a highway environment for 100 years are shown in Figures 2 to 5. According to EKS 10 (Table D-1) the minimum required concrete cover for a road splash environment (XD3) is 45 mm for maximum equivalent water-cement ratio of 0.40.

It is also clear that the addition of GGBS or FA can significantly reduce chloride ingress. According to EKS 10 (Table D-1), the maximum $(w/c)_{eq}$ is 0.40. It seems that, except the concretes with plain Portland cement, all the concretes with *w/b* 0.40 and mineral additions in this study can resist chloride ingress to a level lower than 0.4% Cl by mass of binder at the specified minimum cover depth, as shown in Figures 8 and 9.

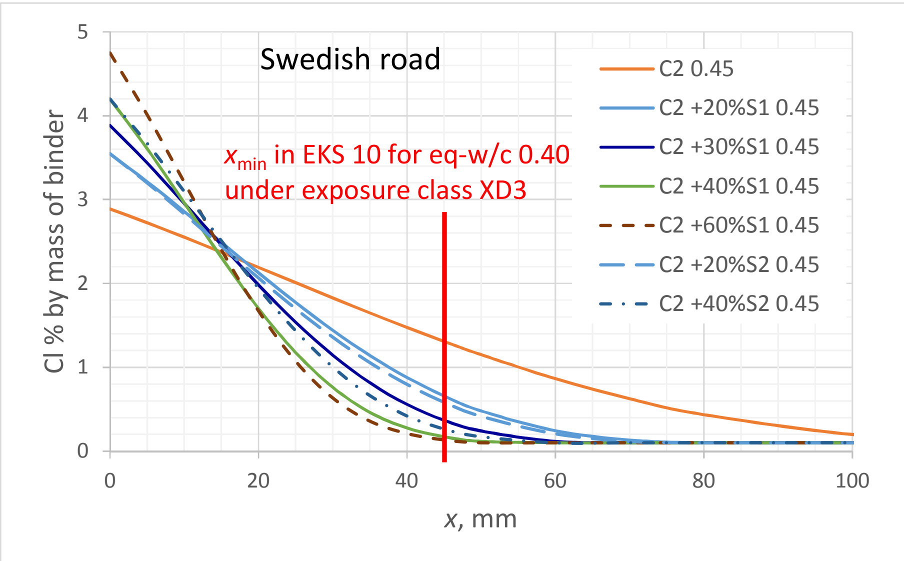

Figure 6. *Modelled chloride ingress after 100 years in concrete with cement type C2 and various additions of GGBS with w/b 0.45, under the exposure class XD3 (Note: the allowable maximum eq-w/c is 0.40 according to EKS 10).*

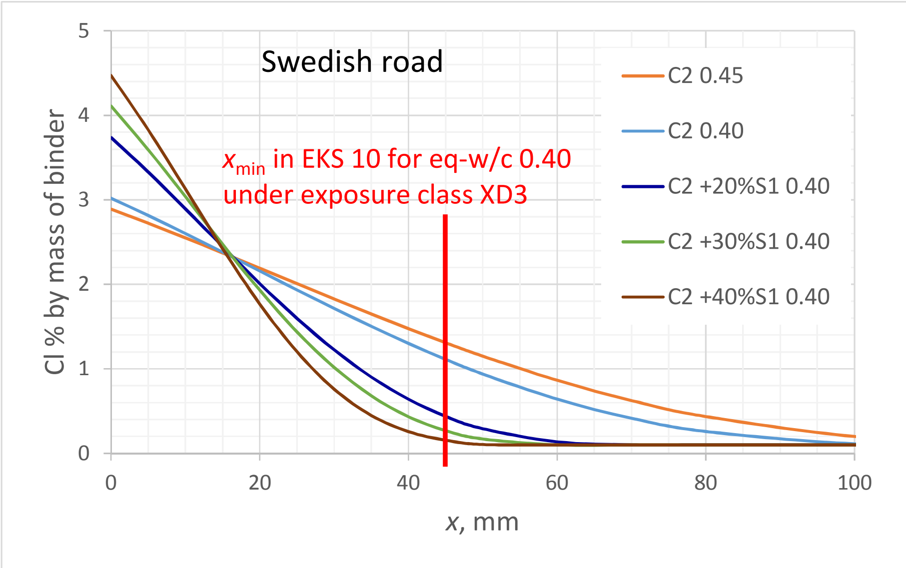

Figure 7. *Modelled chloride ingress after 100 years in concrete with cement type C2 and various additions of GGBS with w/b 0.40, under the exposure class XD3.*

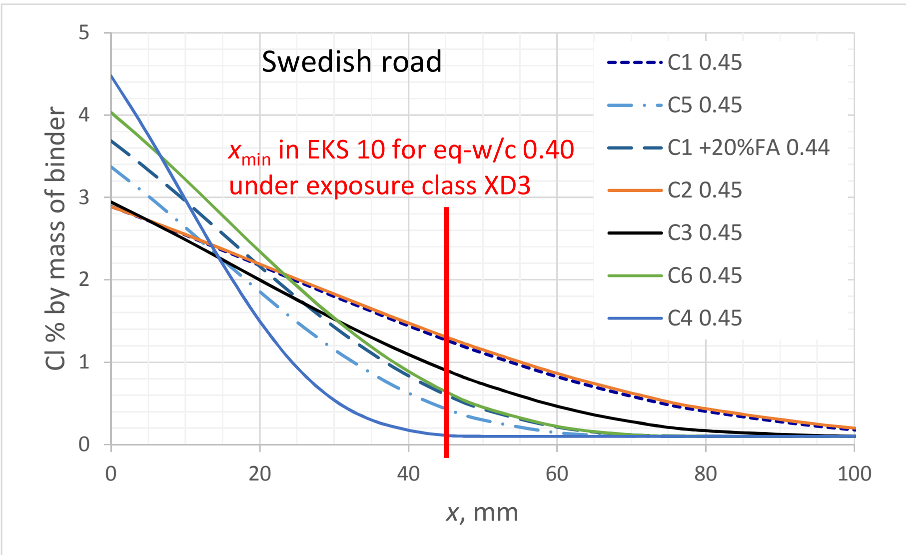

Figure 8. *Modelled chloride ingress after 100 years in concrete with different types of cement with w/b 0.45, under the exposure class XD3 (Note: the allowable maximum eq-w/c is 0.40 according to EKS 10).*

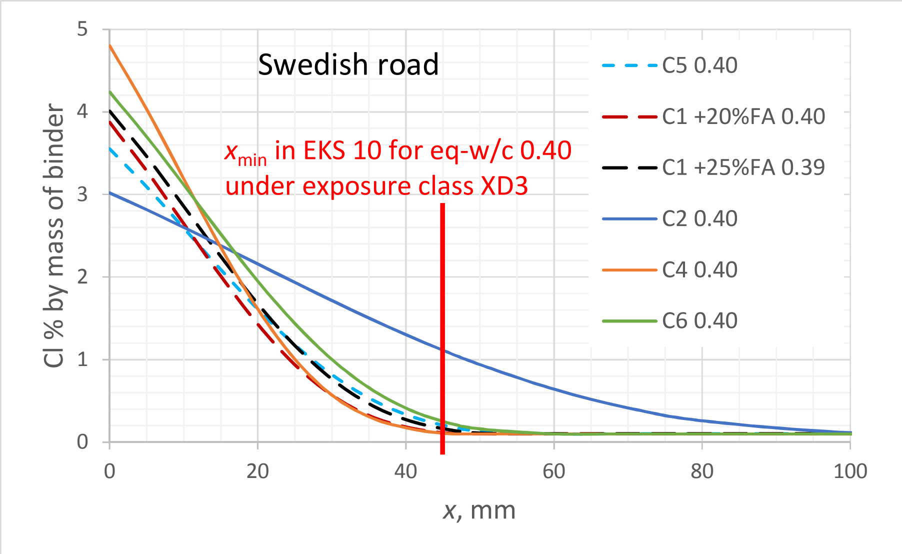

Figure 9. *Modelled chloride ingress after 100 years in concrete with different types of cement with w/b 0.40, under the road splashing condition.*

3.2 Estimation of minimum cover thickness

3.2.1 Chloride threshold values

Assuming a service life of t_L = 100 years, the minimum cover thickness x_c can be estimated from the modelled chloride profiles (as presented in 3.1.4 and 3.1.5) or the following equation in the ClinConc model, if the free chloride threshold value c_{cr} is given.

$$x_c = 2\sqrt{\frac{\xi_D D_{6m}}{1-n}\cdot\left(\frac{t_{6m}}{t_L}\right)^n\cdot\left[\left(1+\frac{t_{ex}}{t_L}\right)^{1-n}-\left(\frac{t_{ex}}{t_L}\right)^{1-n}\right]\cdot t_L}\cdot \mathrm{erf}^{-1}\left(1-\frac{c_{cr}-c_i}{c_s-c_i}\right) \tag{14}$$

In practice, the total chloride content, C_{cr}, in % by mass of binder is used. In this case MS Excel Solver can be employed to find out c_{cr} from equation (10). Garcia et al. (2014) reported that the chloride threshold value (0.6%-0.9% by mass of binder) for concrete with 60% GGBS was similar to that of reference concrete (0.775% by mass of binder as a mean value). According to Boubitsas et al. (2014), the estimated chloride threshold value from the field exposure is about 1% by mass of binder for most types of concrete with Portland cement and silica fume whilst the concretes with FA and GGBS did not show a corrosion tendency at a chloride content lower than 1% by mass of binder. Therefore, as mentioned in section 2.2, the conventional opinion of low C_{cr} for the concrete with mineral additions due to its lower alkalinity is questionable, because on one side there is no sufficient evidence of a significant lower pH value in the pore solution and on the other hand the improved microstructures in such types of concrete may prevail the weakness of low alkalinity, if it is.

On the other hand, the ratio of $[Cl^-]/[OH^-]$ is also used to express chloride threshold for pitting corrosion. With the ClinConc model, it is easy to obtain this ratio because the free chloride profile is calculated from equation (9), prior to the total chloride profile. The question is how to determine $[OH^-]$. According to the ClinConc model, the initial $[OH^-]$ was in a range of 0.3 mol/l (concrete #4, C2 +60%GGBS 0.45) and 0.98 mol/l (concrete "Ref 3", C3 0.45), calculated based on the effective equivalent $[Na_2O]$ in the binder according to equation (13), the binder content and the assessable water content in concrete. The calculated initial $[OH^-]$ concentration are shown in Figure 10, which is comparable with those reported by Vollpracht et al. (2015). The initial $[OH^-]$ for concrete "Ref 1" (C1 0.45) is 0.58 mol/l, corresponding to a pH value of 13.8. If this value is used, at $[Cl^-]/[OH^-]$ = 0.6 as Haussmann (1967) suggested, the free chloride concentration would be 0.35 mol/l or 12.3 g/l, corresponding to a total chloride content of over 6% by mass of cement! This is of course not the case in the reality. If we accept the value of total chloride 1.0% by mass of binder as C_{cr}, corresponding to a free chloride concentration of 0.44 g/l or 0.0124 mol/l, resulting in a ratio of $[Cl^-]/[OH^-] = 0.02$.

In this study for estimation of the minimum cover thickness, therefore, both the value of 1.0% chloride by mass of binder and $[Cl^-]/[OH^-] = 0.02$ will be used as criteria for concrete exposed under the marine environment, and as conventionally, 0.4% chloride by mass of binder and accordingly $[Cl^-]/[OH^-] = 0.01$ will be used as criteria for concrete exposed under the road environment due to the high availability of oxygen and possible carbonation. It is on the conservative side that much lower ratios of $[Cl^-]/[OH^-]$ than that by Haussmann (1967) were used in this study.

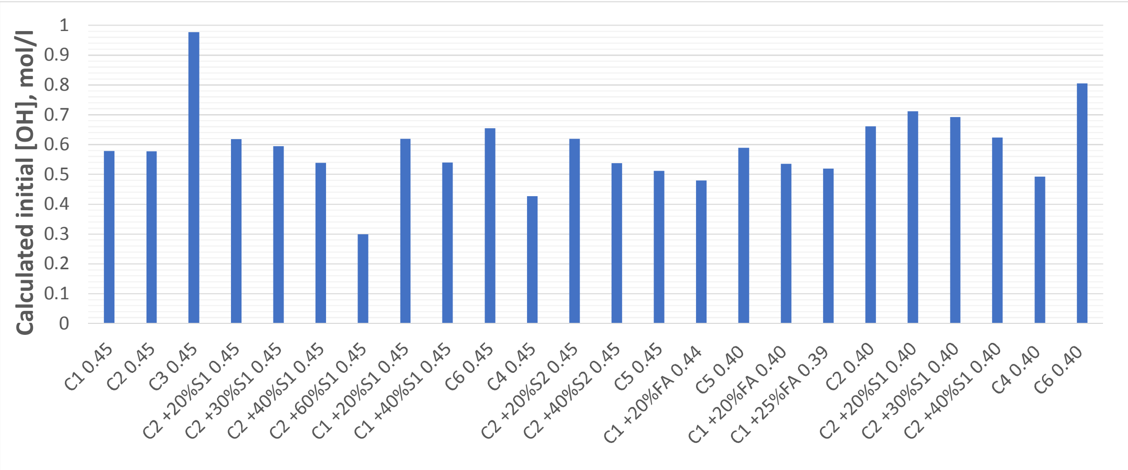

Figure 10. *Calculated initial [OH] in the pore solution of concrete with various types of binder.*

3.2.2 Estimated results

The results of minimum cover thickness estimated using two different C_{cr} criteria for concrete with a service life of 100 years are shown in Figure 11. It can be seen that the concrete with plain Portland cement (C1 0.45, C2 0.45, C3 0.45 and C2 0.40) needs a cover thickness of 120-170 mm or 70-90 mm for Swedish west or east coast environment and 60-80 mm for Swedish road environment. For the concrete with mineral additions, it needs in most cases less than 100 mm or 60 mm cover for Swedish west or east coast environment and 50 mm for Swedish road environment. These concrete covers can be compared with the requirements by Norwegian National Road Administration in its Bridge Projecting Handbook N400 (Statens Vegvesen, 2015), where minimum covers of 100 mm and 60 mm (plus 20 mm tolerance) are specified for the exposure environments of XS2/XS3 and XD3, respectively. The results in Figure 11 show that the minimum cover specified in the current standard EKS 10 (Table D-1) is not suitable for Swedish west coast environment. As stated in this standard, the specified values of minimum cover was based on the seawater of Swedish east coast where the chloride concentration is only 0.4% or 4 g/l, significantly lower than that in the seawater of Swedish west coast (about 14 g/l) and Norwegian coast (>20 g/l). Even in this case, the concretes with plain Portland cement or with less amount of mineral additions may not have sufficient resistance to chloride ingress with the minimum cover specified in EKS 10 (Table D-1). Therefore, further revision of Swedish standard EKS 10 (Table D-1) is needed.

Figure 11 also show that both chloride threshold criteria reveal similar results although the criterion $[Cl^-]/[OH^-]$ gives a little better benefit to the concrete with mineral additions. This is interesting because the initial alkalinity in concrete with high amount of GGBS is lower than that in concrete with plain Portland cement, as shown in Figure 10.

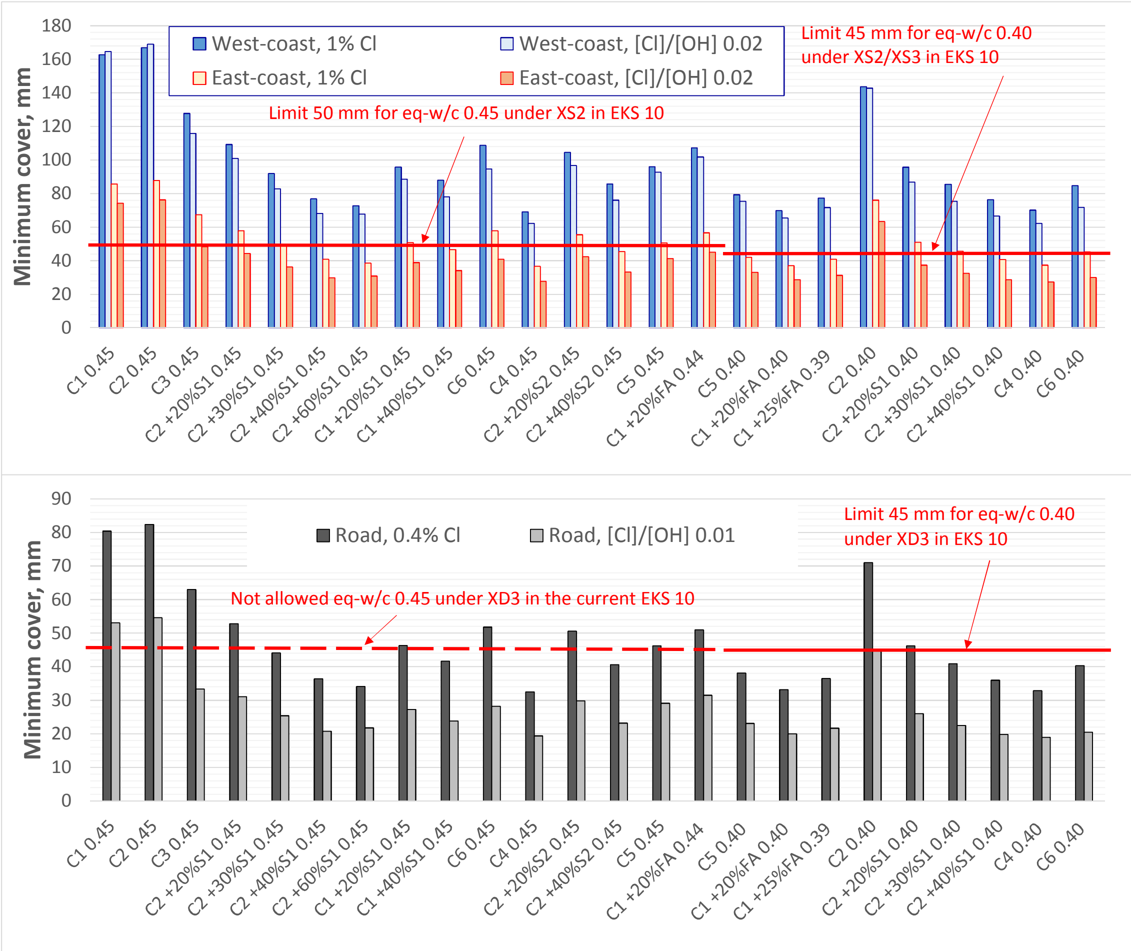

Figure 11. *Estimated minimum covers of concrete exposed to different environments for 100 years.*

In this study cement type C2 was used in most of the mixtures. The calculated minimum cover thickness of this concrete with *w/c* 0.45 ("C2 0.45") for Swedish west coast and road environment, respectively, was used as reference for comparison. Hence the relative cover thickness can be obtained as shown in Figure 12. It can be seen that for the concrete with 30-50% GGBS or 13-25% FA in the binder only about a half of the cover thickness is needed to reach the same service life as the concrete with plain Portland cement.

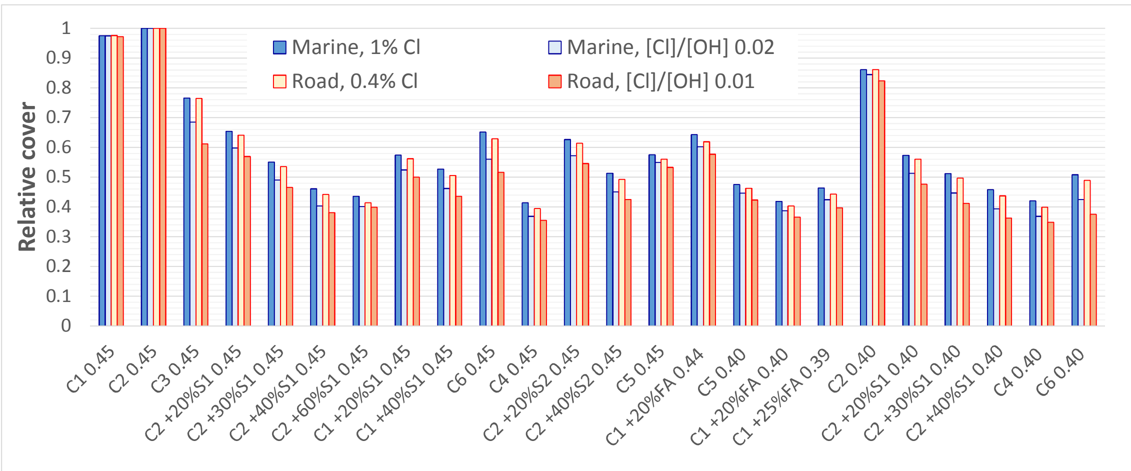

Figure 12. *Relative cover thickness of concrete exposed to marine (Swedish west coast) and road environments for 100 years.*

3.3 Discussion

3.3.1 Uncertainties in the modelling of chloride ingress

Because our basic result from modelling is chloride concentration c or content C, the uncertainty means the quantitative influence of each input parameter on the modelled value of c or C. The sensitivities of various input parameters in the error-function based models have previously been analysed by Tang et al. (2012). Based on their analysis, the sensitivity of surface chloride content c_s is constantly equal to 1, that is,

$$\frac{\Delta c}{\Delta c_s}\cdot\frac{c_s}{c}=1 \tag{15}$$

This means that any relative change in surface concentration c_s will result in an equal relative change in concentration c. The sensitivities of other parameters are dependent on the ratio of c/c_s. The sensitivity of D_{6m} or ξ_D can be expressed as:

$$\frac{\Delta c}{\Delta D_{6m}}\cdot\frac{D_{6m}}{c}=\frac{\Delta c}{\Delta\xi_D}\cdot\frac{\xi_D}{c}=\frac{1}{\sqrt{\pi}}\cdot\frac{c_s}{c}\cdot z\cdot e^{-z^2}=\frac{1}{\sqrt{\pi}}\cdot\frac{z\cdot e^{-z^2}}{\dfrac{c}{c_s}} \tag{16}$$

where

$$z=\frac{x}{2\sqrt{\dfrac{\xi_D D_{6m}}{1-n}\left(\dfrac{t_{6m}}{t}\right)^n\left[\left(1+\dfrac{t_{ex}}{t}\right)^{1-n}-\left(\dfrac{t_{ex}}{t}\right)^{1-n}\right]\cdot t}} \tag{17}$$

It is apparent that the sensitivity of D_{6m} and ξ_D is dependent on both c/c_s and z, the latter contains all the variations except for c_s. Their relationships are illustrated in Figure 13.

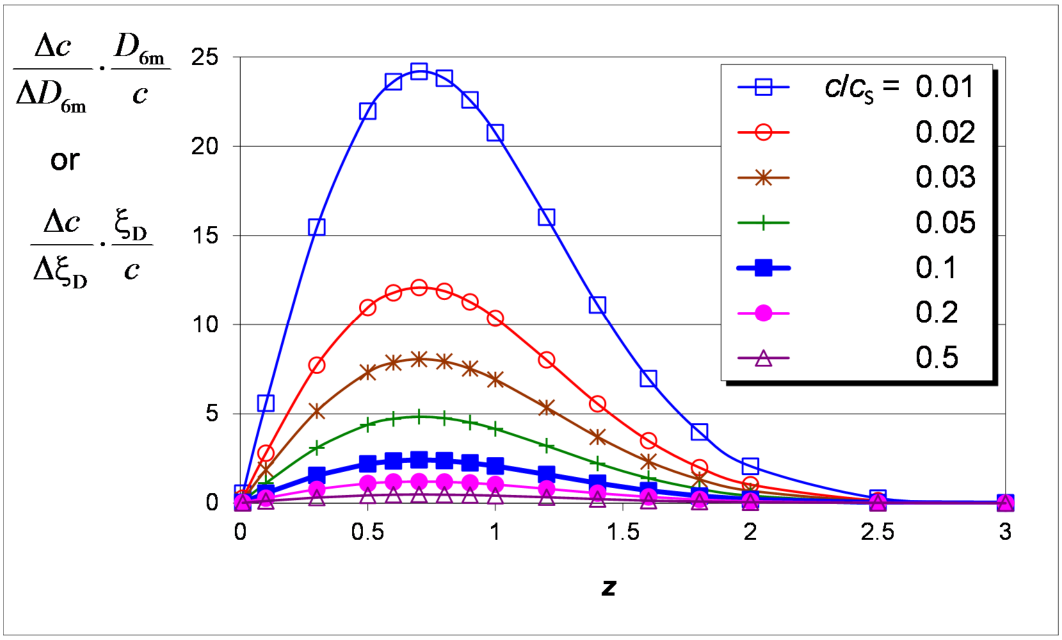

Figure 13. *Sensitivity of parameters* D_{6m} and ξ_D.

The sensitivity of n can be expressed as:

$$\frac{\Delta c}{\Delta n}\cdot\frac{n}{c}=\frac{n}{\sqrt{\pi}}\cdot\frac{z\cdot e^{-z^2}}{\dfrac{c}{c_\mathrm{s}}}\left[\ln\left(\frac{t_\mathrm{6m}}{t}\right)+\frac{1}{1-n}-\frac{\ln\left(1+\dfrac{t_\mathrm{ex}}{t}\right)\cdot\left(1+\dfrac{t_\mathrm{ex}}{t}\right)^{1-n}-\ln\left(\dfrac{t_\mathrm{ex}}{t}\right)\cdot\left(\dfrac{t_\mathrm{ex}}{t}\right)^{1-n}}{\left(1+\dfrac{t_\mathrm{ex}}{t}\right)^{1-n}-\left(\dfrac{t_\mathrm{ex}}{t}\right)^{1-n}}\right] \quad (18)$$

It can be seen that in both equations (16) and (18) the sensitivity of these input parameters are inversely proportional to the ratio of c/c_s. Our interesting concentration is, of course, the threshold concentration for corrosion initiation. When C_cr = 1% by mass of binder was taken as criterion, the value of c_cr varied between 0.13 g/l (concrete "C2 +60%GGBS 0.45") and 0.45 g/l (concrete "C2 0.40"), depending on the chloride binding capacity of the binder in concrete. When the concentration ratio $[Cl^-]/[OH^-]$ = 0.02 was taken as criterion, the value of c_cr varied between 0.21 g/l (concrete "C2 +60% GGBS 0.45") and 0.69 g/l (concrete "C3 0.45"), depending on the initial alkalinity of concrete. In the ClinConc modelling the value n is less than 0.2, t_6m = 0.5 years, t_ex = 0.04 years, t = 100 years, thus t_6m/t = 0.005 and t_ex/t = 0.0004. Under the assumption of cover thickness of 100 mm for marine conditions and 75 mm for de-icing road conditions, the value of z varied between 1.51/1.58 (concrete "C2 0.40") and 1.85/2.07 (concrete "C2 +60% GGBS 0.45") for marine/road conditions. When z = 1.75, the influences of n and c/c_s on the sensitivity of n can be illustrated in Figure 14.

It can be seen from Figures 13 and 14 that the sensitivity of input parameters is strongly dependent on the ratio c/c_s, or more meaningfully, $c_\mathrm{cr}/c_\mathrm{s}$, in concern of corrosion initiation. The influence of parameters D_6m and ξ_D on chloride concentration is different from that of the parameter n. Any increase in the former leads to an increase in concentration whilst any increase in the latter leads to a decrease of concentration. Depending on the individual type of concrete and assumption of chloride threshold, the ratio $c_\mathrm{cr}/c_\mathrm{s}$ varies. As a consequence, the sensitivity of input parameters also varies, as shown in Figures 15 (for parameters D_6m and ξ_D) and 16 (for parameter n). It can be seen that the sensitivities of input parameters for concrete with mineral additions are slightly higher than those with plain Portland cement when chloride threshold C_cr in % by mass of binder was used as criterion but in opposite when $[Cl^-]/[OH^-]$ was used as criterion. The main reason is because of the higher chloride binding capacity and lower alkalinity of concrete with mineral additions, both of which lead to a lower free chloride concentration, or lower value of c/c_s. The significantly lower sensitivity when $[Cl^-]/[OH^-]$ was used as criterion for the road environment is due to the lower value of c_s (1.5 g/l compared with 14 g/l in the seawater), which leads to a higher value of c/c_s.

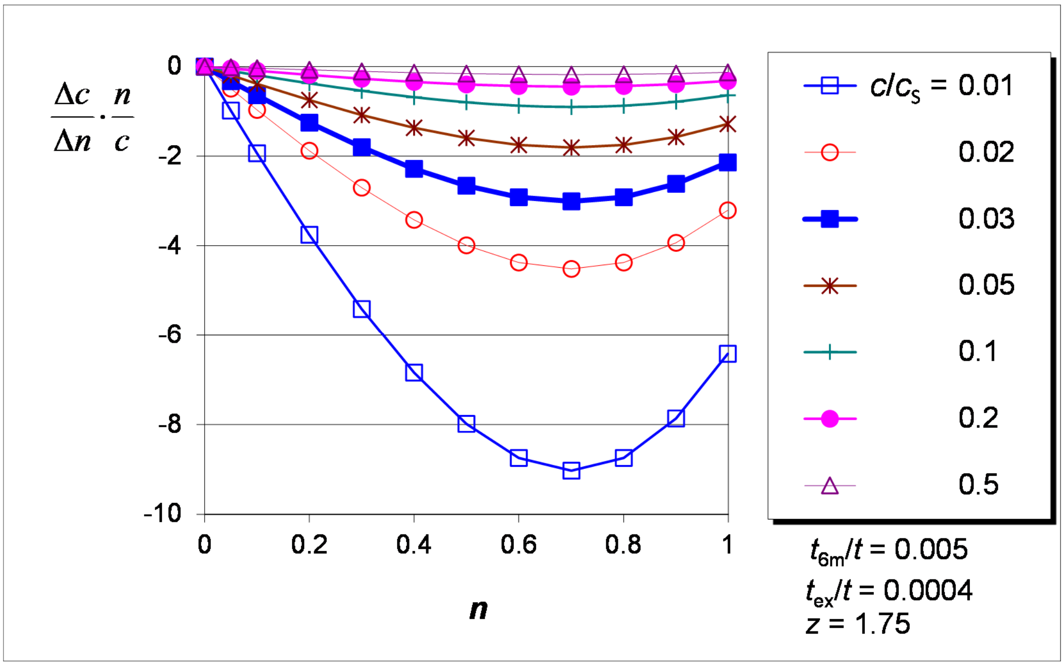

Figure 14. *Sensitivity of parameter n.*

As an example, for concrete "C2 0.45" under the marine condition, the sensitivity of D_{6m} or ξ_D is 2.9 and that of n is -2.1. This means that any increase in D_{6m} or ξ_D by1% will leads to an increase in c at the cover depth by 2.9% after 100 years, whilst any increase in n by 1% will leads to a decrease in c at the cover depth by 2.1% after 100 years.

Because the effects of chloride binding, temperature, hydroxide, and so on have already been involved in the parameter ξ_D, no further analysis was made for their individual effect.

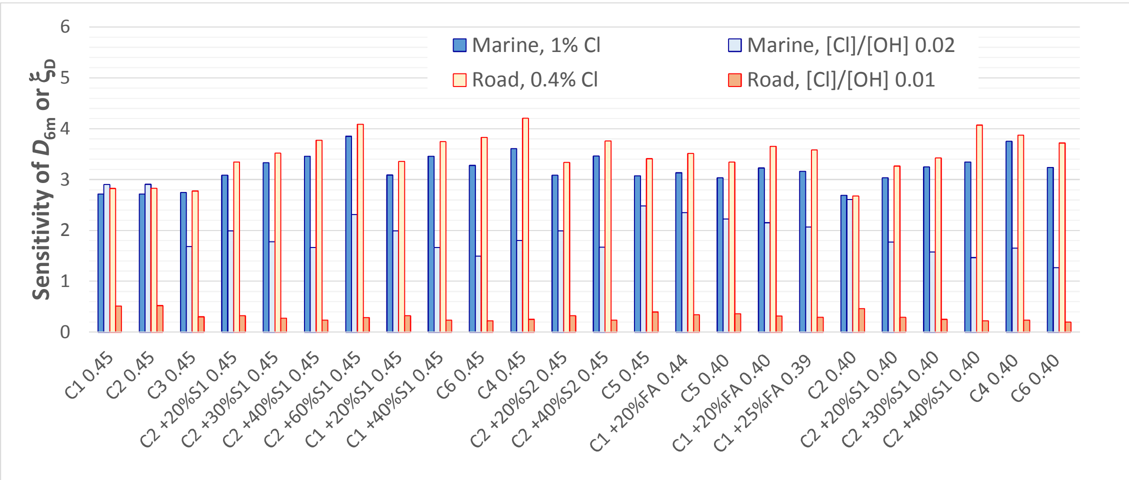

Figure 15. *Sensitivity of parameter D_{6m} and ξ_D for various types of concrete (Marine = Swedish west coast).*

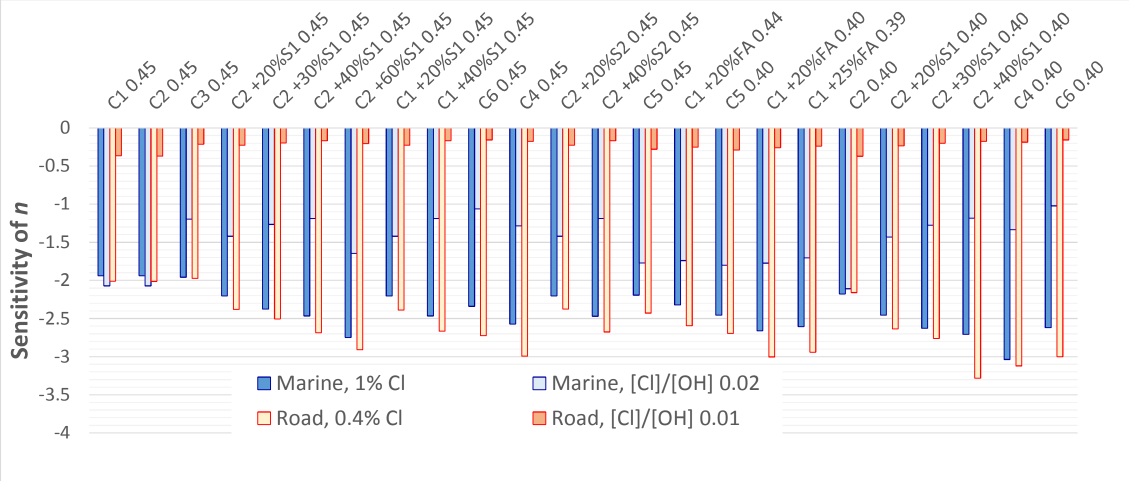

Figure 16. *Sensitivity of parameter n for various types of concrete (Marine = Swedish west coast).*

3.3.2 Uncertainties in the estimation of minimum cover thickness

It is difficult to find analytical solution of sensitivity of various parameters from question (14). From the previous section it is known that the ratio c/c_s has strong influence on the sensitivity of the input parameters. Therefore, the threshold level c_{cr} is one of the parameter influencing the estimation of minimum cover thickness. Other influencing parameters include D_{6m}, c_s and temperature T. In this study only the road environment, that is, C_{cr} = 0.4% Cl by mass of binder, c_s = 1.5 g/l, and T = 10 °C, was considered for evaluating the uncertainties. The value of D_{6m} is as listed in Table 2. For each parameter a variation of ±10% of its value was set. The standard deviation of the three values of output from each mix was divided by its mean value to obtain the coefficient of variation (CoV). The results are shown in Figure 17. It can be seen that a 10% variation of each input parameter may result in a 3-5% variation of the determined minimum cover thickness, independent of type of concrete. It is interesting to see that the variation of threshold value, C_{cr}, has the least whilst the variation of diffusivity has the largest influence among the four parameters. This is, of course, in agreement with the sensitivity analysis as presented in 3.3.1. Moreover, the threshold value of 1.0% Cl by mass of binder for the marine condition was based on the reinforced concrete with a cover of 15-25 mm. For large concrete covers the effects of more stable internal climate and less oxygen availability may have positive impact on the chloride threshold values and corrosion rate.

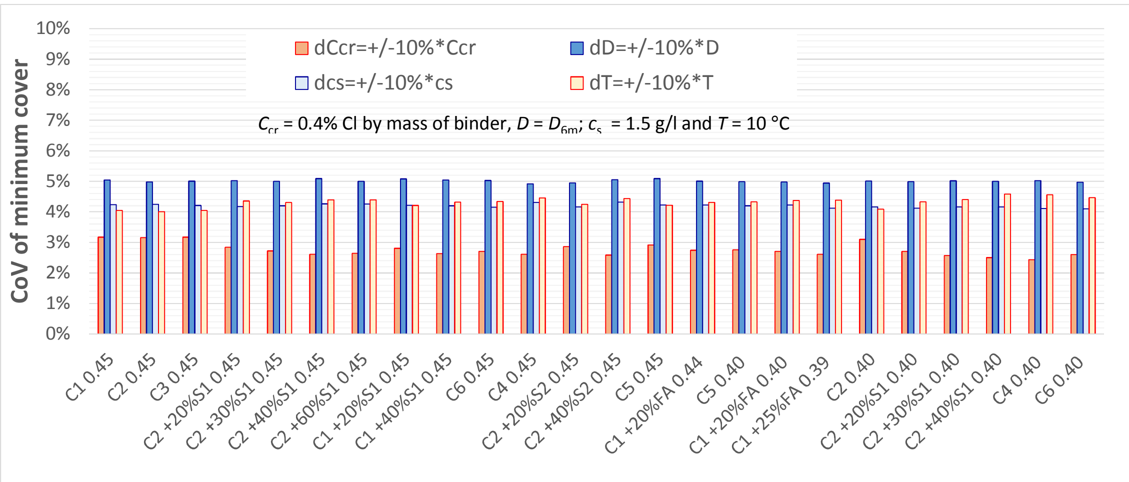

Figure 17. *The coefficient of variation (CoV) of minimum cover of various types of concrete (Marine = Swedish west coast).*

3.3.3 Allowable low limit of chloride threshold

So far it is still lack of actual chloride threshold value for concrete especially for the concrete with mineral additions due to the lack of standard test method for the threshold value. With the help of the ClinConc model, the allowable low limit of chloride threshold in concrete with mineral additions can be estimated with the same service life and cover thickness as the reference concrete with plain sulphate resistance Portland cement C1 and C2.

Under the assumption of the chloride threshold value of 1.0% and 0.4% by mass of binder for the marine and road environment, respectively, the service lives of reference concretes with cement C1 and C2 were calculated using the ClinConc model with the cover specified in EKS 10. The calculated results are listed in Table 5. It is obvious that the service life is far shorter from the expected 100 years, especially for the concrete exposed to XS2 and XS3. Therefore, in this study the cover thickness of 100 and 70 mm and chloride threshold value of 1.0% and 0.4% by mass of binder for the marine and road environment, respectively, were assumed for the calculation of service life of reference concretes with cement C1 and C2 using the ClinConc model. For cement type “C1”, the service life of concrete with *w/c* 0.40 was estimated based on the value with *w/c* 0.45 and the ratio of service life of the “C2” reference concrete with water-binder ratio 0.40 to 0.45. These service lives were then used for finding out the allowable low limits of chloride threshold using MS Excel Solver. The results are illustrated in Figure 18 and listed in Table 6. It can be seen that, the allowable low limit of threshold for all the types of concrete with mineral additions is quite lower than the reference threshold 1.0% and 0.4% for marine and road environment, respectively. The most of concretes with mineral addition even reveal a value 0.1% (the initial total chloride content) in Figure 18, implying that no external chloride has reached the cover depth 100 mm for marine or 70 mm road environment. This value of 0.1% is also the maximum allowable chloride content for pre-stressed reinforced concrete according to SS-EN 206 (Table 15) and SS 137003 (Table 7). This means that even the chloride threshold value is reduced to 0.1% by mass of binder, the concrete with mineral additions would achieve the same service life as that with plain sulphate resistance Portland cement. Thus the concrete with mineral additions would have sufficient margin to protect reinforcement steel from corrosion. The concrete with cement type C3 also shows a lower allowable threshold value. This is due to its lower chloride migration coefficient as shown in Table 4 and possibly higher chloride binding capacity with its higher content of C_3A, which can strongly bind chloride to form Friedel’s salt.

Table 5 Modelled service life of reference concrete with the cover specified in EKS 10.

Cement	*C1*	*C2*	*C2*
w/c	0.45		0.40
Cover (mm) for L 100 acc. to EKS 10	50		45
Exposure class	*XS2, XS3*		*XS3, XD3*
Modelled service life (year), west coast	11	10	12
east coast	34	32	34
road	-	-	45

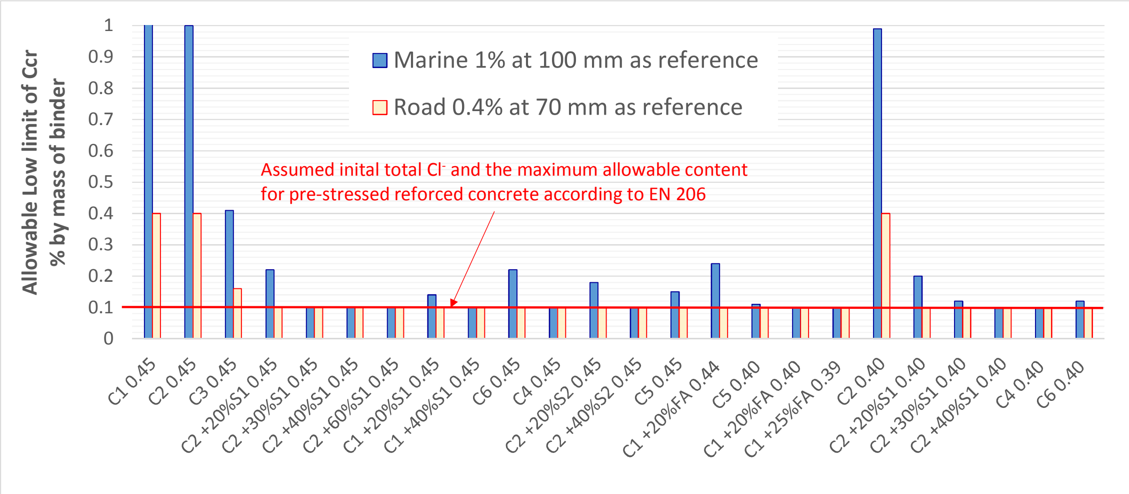

Figure 18. *Allowable low limit of threshold for concrete with mineral additions (Marine = Swedish west coast; the cover thickness of 100 and 70 mm were assumed in marine and road environment, respectively).*

Table 6 Modelled service life and allowable low limit of chloride threshold.

Mix ID	*Binder type & w/b*	*Marine environment XS2 Cover 100 mm*		*Road environment XD3 Cover 60 mm*	
		t_L *year*	C_{cr} % mass of binder	t_L *year*	C_{cr} % mass of binder
Ref 1	**C1 0.45**	**36**	**1**	**75**	**0.4**
Ref 2	**C2 0.45**	**34**	**1**	**71**	**0.4**
Ref 3	C3 0.45	34	0.41	71	0.16
#1	C2 +20%S1 0.45	34	0.22	71	*0.1*
#2	C2 +30%S1 0.45	34	*0.1*	71	*0.1*
#3	C2 +40%S1 0.45	34	*0.1*	71	*0.1*
#4	C2 +60%S1 0.45	34	*0.1*	71	*0.1*
#5	C1 +20%S1 0.45	36	0.14	75	*0.1*
#6	C1 +40%S1 0.45	36	*0.1*	75	*0.1*
#7	C6 0.45	34	0.22	71	*0.1*
#8	C4 0.45	34	*0.1*	71	*0.1*
#9	C2 +20%S2 0.45	34	0.18	71	*0.1*
#10	C2 +40%S2 0.45	34	*0.1*	71	*0.1*
#11	C5 0.45	36	0.15	75	*0.1*
#12	C1 +20%FA 0.45	36	0.24	75	*0.1*
#13	C5 0.40	49 1)	0.11	103 2)	*0.1*
#14	C1 +20%FA 0.40	49 1)	*0.1*	103 2)	*0.1*
#15	C1 +25%FA 0.40	49 1)	0.10	103 2)	*0.1*
#16	**C2 0.40**	**46**	**1**	**98**	**0.4**
#17	C2 +20%S1 0.40	46	0.20	98	*0.1*
#18	C2 +30%S1 0.40	46	0.12	98	*0.1*
#19	C2 +40%S1 0.40	46	*0.1*	98	*0.1*
#20	C4 0.40	46	*0.1*	98	*0.1*
#21	C6 0.40	46	0.12	98	*0.1*

1) Calculated by 36×46/34 (service life of “C1 0.45” × “C2 0.40”/ “C2 0.45” under XS2).
2) Calculated by 75×98/71 (service life of “C1 0.45” × “C2 0.40”/ “C2 0.45” under XD3).

3.3.4 Benefit of mineral addition to concrete durability

The results from the above modelling together with the limited field data after exposure in the Träslövsläge harbor for 19 years have given a certain evidence showing the positive contribution of mineral addition to the resistance of concrete to chloride ingress. For the addition of GGBS up to 60% in this study (75% for the field exposure), the chloride resistance increases with the addition level. In consideration of both chloride resistance and alkalinity, the concrete with mineral additions still reveals sufficient margin to allow a lower chloride threshold for initiation of corrosion of reinforcement steel in concrete, although a reliable test method is needed to quantify chloride threshold value.

The above findings are also in agreement with the finding in the literature. Mangat and Molloy (1991) studied the chloride induced corrosion of concrete with various mineral additions and concluded that the addition of 60% GGBS, significantly reduced corrosion rate of reinforcement, similar to the addition of 10-15% silica fume. Al-Haj Hussein (2003) also studied corrosion behavior of reinforcement in concrete with 30% FA and 70% GGBS, and observed less corrosion in concrete with 30% fly ash and 70% GGBS. Bouteiller et al. (2012) examined the corrosion resistance of concrete with 70% GGBS (CEM III/B 42.5 N) and also found better corrosion resistance than the reference concrete. Gruyaert (2011) studied concrete with 50%, 70% and 85% GGBS and found that the chloride migration coefficient using the test method NT BUILD 492 decreased with the increased addition of GGBS. Olsson et al. (2013) tested also chloride migration coefficient of concrete with 40% and 70% GGBS using NT BUILD 492 and found again a markedly lower diffusivity of concrete with the addition of GGBS than that of concrete with plain Portland cement. An overview of the corrosion resistance of concrete with GGBS has been published by Song and Saraswathy (2006). According their review, the conclusions related to the durability of concrete in chloride-induced corrosive environment include

- Replacement of cement by 40% GGBS has no significant influence on corrosion rates of rebar in concrete. At the replacement level of 60% the corrosion rate is significantly reduced.
- The reduction of pH value due to the addition of GGBS seems no adverse effect on the corrosion resistance of concrete.
- An increased addition of GGBS leads to a decreased rate of corrosion of reinforcement in concrete.

In a recent report Sharp et al. (2014) demonstrated again that the concrete with 40% GGBS revealed the least amount of corrosion among all the concretes with 6 types of binder after prolonged exposure to saltwater.

So far very limited data from long-term field exposure are available. Mohammed et al. (2003) reported some data from concrete with Japanese slag cement type B at water-binder ratio of 0.52-0.55 after exposure in the seawater for 30 years. The results showed that the chloride ingress in concrete with slag cement was significantly lower than that with Portland cement.

3.3.5 Minimum cover thickness

As presented in section 3.2.2, concrete with different types of binder has different resistance to chloride resistance. To achieve the designed service life the minimum cover thickness should be different depending on the type of binder used in concrete. Based on the results from this study, some values of minimum cover thickness are suggested in Table 7, where the values of CEM I were based on the modelled results from concrete mixes Ref 1, Ref 2 and #16 (C1 and C2); CEM II/A-S based on mixes #1, #5, #9 and #17 (C2 +20%S1 or S2); CEM II/A-V based on mixes #11 to #14 (C5 with 13%FA and C1 +20%FA); CEM II/B-V based on mix #15 (C1 +25%FA); CEM II/B-S based on mixes #2, #7, #18 and #21 (C2 +30%S1 and C6 with 33% GGBS); and CEM III/A based on mixes #3, #4, #6, #8, #10, #19 and #20 (C2 +40%S1, +60%S1 and C4 with 49% GGBS).

Table 7. Suggested values of minimum cover (mm) of concrete with different types of binder for a service life of 100 years in Swedish exposure conditions.

Exposure class	*XS2*				*XD3*	
Environment	*West coast*		*East coast*		*Road environment*	
Binder type	*w/b*	*Cover*	*w/b*	*Cover*	*w/b*	*Cover*
CEM I	0.40	140	0.45 0.40	90 80	0.45 0.40	80 70
CEM II/A-S[1)]	0.45 0.40	120 100	0.45 0.40	60 50	0.45 0.40	55 45
CEM II/A-V[1)] or CEM II/B-V	0.45 0.40	100 80	0.45 0.40	60 45	0.45 0.40	50 45
CEM II/B-S	0.45 0.40	110 90	0.45 0.40	60 50	0.45 0.40	50 45
CEM III/A	0.45 0.40	90 80	0.45 0.40	50 45	0.45 0.40	50 45

[1)] Assuming about 15% GGBS or FA as the minimum addition.

It should be noticed that the water-binder ratio *w/b*, i.e. no efficiency factor is used, was given in Table 7 instead of equivalent water-cement ratio in EKS 10 (Table D-1). This is more conservative because in practice the equivalent water-cement ratio will anyhow be used in order to fulfill the requirements for strength.

From the above table it can also be seen that concrete with plain Portland cement needs a thick cover. Mineral additions can effectively reduce the cover to a size more beneficial to not only material saving but also structural reinforcement.

In addition to the specification of minimum cover and maximum water-binder ratio, concrete should be tested using e.g. the rapid chloride migration test or similar standardized test in order to validate the performance of the concrete mix with regard to chloride resistance. Models validated with the field data can be used to examine the actual cover thickness for the designed service life.

4 Summary and conclusions

This study briefly reviewed some models for chloride ingress and factors influencing chloride threshold for reinforcement corrosion. The ClinConc model was used to model chloride ingress in concrete with various mineral additions with the measured chloride migration coefficient as the key input parameter. The uncertainties involved in the modelling were assessed through the analysis of sensitivity of input parameters. Some limited field data measured from the concrete exposed in the Träslövsläge seawater for about 20 years were used for validation of the modelled results. From both the literature review and the experimental and modelling results it can be concluded that, for the mineral additions,

- The chloride resistance of concrete increases with mineral addition. For GGBS, the higher the addition level (up to 60% GGBS in this study), the better the resistance is, whilst for FA, the addition level in the range of 13% and 25% reveals similar resistance.
- The alkalinity of concrete with GGBS may not necessarily be low because both the alkaline components in GGBS and the reduced porosity contribute to a high concentration of hydroxide ions in the pore solution.
- The alkalinity of concrete with FA is proportionally reduced with the addition of FA, but the reduction is limited if the addition of FA is not more than 25%.
- In consideration of both chloride resistance and alkalinity, the concrete with mineral additions still reveals sufficient margin to allow a lower chloride threshold for initiation of corrosion of reinforcement steel in concrete.

For the prediction models and minimum cover,

- The sensitivity of input parameters in the ClinConc model is strongly dependent on the ratio of free chloride threshold concentration to the surface concentration c_{cr}/c. The sensitivity of input parameters for concrete with mineral additions varies depending on the selection of chloride threshold, which influences the ratio c_{cr}/c.
- A 10% variation of each input parameter leads to a 3-5% variation of the modelled minimum cover thickness, independent of mineral additions. Among four parameters (chloride threshold C_{cr}, migration coefficient D_{6m}, surface chloride concentration c_s, and exposure temperature T) coefficient D_{6m} caused the largest variation by 5% whilst chloride threshold C_{cr} became the least variation by 3%.
- The values of minimum cover specified in the current standard EKS 10 (Table D-1) should be revised based on the type of binder used in concrete. Some suggested values are given in Table 7. To validate the performance of the concrete mix, the resistance to chloride ingress should be tested using e.g. the rapid chloride migration test or similar standardized test.
- To assure sufficient margin for external chlorides, the maximum allowable initial total chloride content should be limited to 0.1% by mass of binder for both marine and road exposure environments.

Finally, it can be pointed out that the overall effect of mineral additions in concrete is significant in terms of resistance to chloride ingress with a marginal influence on the chloride threshold value. Therefore, the use of mineral additions in concrete should have a clear, great advantage from viewpoint of sustainability in terms of technical performance, cost-effectiveness and ecological benefit.

References

Al-Haj Hussein, M.T. (2003), The effect of cement type on the potential and corrosion behaviour of steel reinforcement. PhD thesis, School of Engineering, Civile Engineering, University of Surrey, September 2003, U.K.

Alonso M.C. and Sanchez M. (2009), Analysis of the variability of chloride threshold values in the literature, Materials and Corrosion. 60, 631-637.

Andrade C., Rebolledo N., Castillo A., et al., (2014), Use of resistivity and chloride resistance measurements to assess concrete durability of new panama canal, in: D. Bjegović, H. Beushausen and M. Serdar, ed., Performance-based Specification and Control of Concrete Durability, Zagreb 11-13 June, RILEM PRO 089, 411-418.

Angst U., Elsener B., Larsen C.K. and Vennesland C.K., Ø. (2009), Critical chloride content in reinforced concrete – A review, Cement and Concrete Research, 39, 1122-1138.

Bamforth P.B. (2004), Enhancing reinforced concrete durability: Guidance on selecting measures for minimising the risk of corrosion of reinforcement in concrete. Technical Report No. 61. Camberley: The Concrete Society.

Bentur A., Diamond S. and Berke N.S. (1997), Steel corrosion in concrete – fundamentals and civil engineering practice, Taylor & Francis, London.

Bertolini L., Elsener B., Pedeferri P. and Polder R. (2004), Corrosion of steel in Concrete – Prevention, Diagnosis, Repair, Wiley-VCH, Weinheim.

Boubitsas D., Tang L. and Utgenannt P. (2014), Chloride Ingress in Concrete Exposed to Marine Environment - Field data up to 20 years' exposure. CBI Report to SBUF Project 12684, Swedish Cement and Concrete Research Institute, Stockholm, Sweden.

Bouteiller V., Cremona C., Baroghel-Bouny V. and Maloula A. (2012), Corrosion initiation of reinforced concretes based on Portland or GGBS cements: Chloride contents and electrochemical characterizations versus time. Cement and Concrete Research, 42, 1456-1467.

Boverket (2016), The Board's Building Regulations, EKS 10 – Section D – EN 1992 – Concrete Structures: Chapter 2.1.1 – Application of SS-EN 1992-1-1 – General Rules (in Swedish). Swedish National Board of Housing, Building and Planning, ISBN pdf: 978-91-7563-328-2, Karlskrona.

Collepardi M., Marcialis A. and Turriziani, R. (1972), Penetration of chloride ions into cement pastes and concrete. Journal of American Ceramic Society, 55, 534-535.

Diamond S. (1986), Chloride concentrations in concrete pore solutions resulting from calcium and sodium chloride admixture, Cement Concrete and Aggregates, 8, 97-102.

Engelund S., Edvardsen, C. and Mohr, L. (2000), General Guidelines for Durability Design and Redesign, DURACRETE: Probabilistic Performance based Durability Design of Concrete Structures. EU-Project (Brite EuRam III) No. BE95-1347, Report R15.

fib (2006), Model code for service-life design. *fib* bulletin 34, Federation International du Beton, 1st ed. Lausanne, Switzerland.

Frederiksen J.M. (2000), Chloride threshold values for service life design. Proceedings of the 2rd International RILEM Workshop on Testing and Modelling the Chloride Ingress into Concrete, Paris, September 2000, pp. 397-414.

Frederiksen J.M. and Geiker M. (2000), On an empirical model for estimation of chloride ingress into concrete., in: C. Andrade and J. Kropp, ed. Testing and Modelling the Chloride Ingress into Concrete, Paris Sept. 2000, RILEM PRO 19, 355-371.

Frederiksen J.M. and Geiker M., (2008), Chloride ingress prediction - Part 1 & Part 2, in: E. Schlangen and G. De Schutter, ed. Concrete Modelling - ConMod '08, Delft April 2008, RILEM PRO 58, 275-290.

Frederiksen J.M., Nilsson, L.-O., Poulsen, E., et al. (1996), HETEK - A system for estimation of chloride ingress into concrete: Theoretical background. Report No. 83, Danish Road Directorate, Copenhagen, Denmark.

Garcia V.François R., Carcasses M. and Gegout P. (2014), Potential measurement to determine the chloride threshold concentration that initiates corrosion of reinforcing steel bar in slag concretes. Materials and Structures, 47(9), 1483-1499

Gruyaert E. (2011), Effect of blast-furnace slag as cement replacement on hydration, microstructure, strength and durability of concrete, PhD thesis, Magnel Laboratory for Concrete Research, Department of Structural Engineering, Faculty of Engineering & Architecture Ghent University, Belgium.

Hausman D.A. (1967), Steel corrosion in concrete, Materials Protection. 6(11) 19-23.

Jin W.-L. and Xu C. (2010), Service life prediction of concrete marine structures, in: K. van Breugel, Guang Ye and Yong Yuan, ed., Service Life Design for Infrastructures, Delft 4-6 October, RILEM PRO 070, 339-350.

Johannesson B. (2000), Transport and sorption phenomena in concrete and other porous media, PhD thesis, Report TVBM-1019, Lund Institute of Technology, Lund, Sweden.

Lindvall A. (2007), Chloride ingress data from field and laboratory exposure – Influence of salinity and temperature. Cement and Concrete Composites, 29, 88–93.

Locher F.W. (2006), Cement: Principles of production and use, Düsseldorf, Germany: Verlag Bau + Technik GmbH, 2006, ISBN 3-7640-0420-7.

Majlbro L. (1996), The complete solution of Fick's second law of diffusion with time-dependent diffusion coefficient and surface concentration', in Durability of Concrete in Saline Environment, CEMENTA AB, Danderyd, Sweden, 127-158.

Mangat and Molloy (1991), Influence of PFA, slag and microsilica on chloride induced corrosion of reinforcement in concrete, Cement and Concrete Research, 21, 819-834.

Meijers S.J.H. (2003), Computational modelling of chloride ingress in concrete, PhD thesis, Delft University, Delft, Netherlands.

Nguyen, T.Q., Baroghel-Bouny, V., Dangla, P., et al. (2006), Multi-level modelling of chloride ingress into saturated concrete, in: V. Baroghel-Bouny, C. Andrade, R. Torrent and K. Scrivener, ed. Performance Based Evaluation and Indicators for Concrete Durability, Madrid 19-21 March, RILEM PRO 047, 145-155.

Nordtest (1999), NT BUILD 492 – Concrete, Mortar and Cement Based Repair Materials: Chloride Migration Coefficient from Non-steady State Migration Experiments, NORDTEST, Espoo, Finland.

Olsson N., Baroghel-Bouny V., Nilsson L.-O. and Thiery M. (2013), Non-saturated ion diffusion in concrete – A new approach to evaluate conductivity measurements. Cement and Concrete Composites, 40, 40–47.

Page C.L., Short N.R. & Holden W.R. (1986): The influence of different cements on chloride-induced corrosion of reinforcing steel. *Cement and Concrete Research*, 16, 79-86.

Petre-Lazar I., Abdou L., Franco C., et al. (2003), THI: a physical model for estimating the coupled transport of heat, moisture and chloride ions in concrete, in: D.J. Naus, ed. Life Prediction and Aging Management of Concrete Structures, Paris 5-6 May, RILEM PRO 029, 55-69.

Poulsen E. (1993), On a model of chloride ingress into concrete having time-dependent diffusion coefficient, in Chloride Penetration into Concrete Structures - Nordic Miniseminar, Division of Building Materials, Chalmers University of Technology, Publication P-93:1, Gothenburg, Sweden, 298-309.

Samson, E. and Marchand, J. (1999), Modeling the mechanisms of ion diffusion in porous media. International Journal for Numerical Methods in Engineering, 46, 2043-2060.

Sergi W, Yu S.W. and Page C.L. (1992), Diffusion of chloride and hydroxyl ions in cementitious materials exposed to a saline environment. Magazne of Concrete Research, 44(158), 63–9.

Sharp S.R., Ozyildirim C. and Mokarem D.W. (2014), Corrosion sensitivity of concrete mix designs. VCTIR 14-R19, Virginia Center for Transportation Innovation and Research, Charlottesville, Virginia, June 2014.

Shi X., Xie N., Fortune K., et al. (2012), Durability of steel reinforced concrete in chloride environments: An overview. Construction and Building Materials, 30, 125-138.

Song H.W. and Saraswathy V. (2006), Studies on the corrosion resistance of reinforced steel in concrete with ground granulated blast-furnace slag – An overview. Journal of Hazardous Materials, B138, 226-233.

Statens Vegvesen (2015), Bridge Projecting Handbook N400 (in Norwegian). Norwegian National Road Administration, ISBN: 978-82-7207-680-0, Oslo.

Tang L. (1996), Chloride transport in concrete - Measurement and prediction, Thesis (PhD), Chalmers Universities of Technology, Publication P-96:6, Gothenburg, Sweden.

Tang L. (2003), Chloride ingress in concrete exposed to marine environment – Field data up to 10 years exposure. SP Report 2003:16, SP Swedish National Testing and Research Institute, Borås.

Tang L. (2006), Service-life prediction based on the rapid migration test and the ClinConc model, in: V. Baroghel-Bouny, C. Andrade, R. Torrent and K. Scrivener, ed. Performance Based Evaluation and Indicators for Concrete Durability, RILEM PRO 047, 157-164.

Tang L. (2008), Engineering expression of the ClinConc model for prediction of free and total chloride ingress in submerged marine concrete. Cement and Concrete Research, 38, 1092-1097.

Tang L. and Nilsson L.-O. (1993), Chloride binding capacity and binding isotherms of OPC pastes and mortars. Cement and Concrete Research, 23, 347-353.

Tang L., Nilsson L.O. and Basheer P.A.M. (2011), Resistance of concrete to chloride ingress – Testing and modelling. London: Taylor & Francis Group.

Tang, L. and Nilsson, L.-O. (1994), A numerical method for prediction of chloride penetration into concrete structures, in: H. Jennings, J. Kropp and K. Scrivener, ed. The Modelling of Microstructure and its Potential for Studying Transport Properties and Durability, Dordrecht: Kluwer Academic Publisher, 539-552.

Tang L., Utgenannt P., Lindvall A. and Boubitsas D. (2012), "Validation of models and test methods for assessment of durability of concrete structures in the road environment", CBI Report 2:2012, Swedish Cement and Concrete Research Institute, Stockholm, Sweden.

Thomas M.D.A. and Bentz D. (2001), Life 365 – Computer program for predicting the service life and life-cycle costs of reinforced concrete structures exposed to chlorides, User Manual (version 1.0.0), presented at the Nordic Mini Seminar & *fib* TG 5.5 Meeting, Gothenburg 22-23 May.

Truc, O., Ollivier, J.P. and Nilsson, L.-O. (2000), Numerical simulation of multi-species diffusion. Materials and Structures, 33, 566-573.

Tuutti K. (1982), Corrosion of steel in concrete. CBI Report 4:82, Swedish Cement and Concrete Research Institute, Stockholm.

Vollpracht A., Lothenbach B., Snellings R. and Haufe J. (2015), The pore solution of blended cements: a review. Materials and Structures, published online 15 October 2015, DOI 10.1617/s11527-015-0724-1.