



Nonlinear relationships between parameters of simplified diffusion-based model for service life design of concrete structures exposed to chlorides

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ABSTRACT

This paper presents an approach toward performance-based durability design of concrete structures based on a simplified diffusion-based model, which overcomes some of the limitations of the current prescriptive rules of existing design standards. Numerical nonlinear relationships and relative sensitivities between the four parameters that govern the time to corrosion initiation of reinforced concrete structures are determined, including chloride diffusion coefficient, chloride threshold value of reinforcement, concrete cover and surface chloride exposure condition. To achieve a specified design life, it is found that in aggressive chloride-laden environments: (i) increasing the minimum concrete cover is more effective than selecting a corrosion-resistant steel; (ii) it is necessary to use both high-performance concrete and corrosion-resistant steel; (iii) a relative decrease in the concrete cover has to be compensated by a much greater increase in the corrosion resistance of steel; and (iv) the variations in chloride diffusion coefficient have a large impact on required concrete cover and chloride threshold value, especially in increasingly aggressive environments. These relationships can be used to identify the most relevant parameters and serve as a guide for service life-based design for durable concrete structures.

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1. Introduction

Chloride-induced corrosion of steel reinforcement has been identified as one of the main causes of deterioration of concrete structures, such as highway bridges, parking structures, and marine structures. It has been extensively studied for decades [1–5], particularly, as a result of the high costs of highway bridge repair in North America and Europe from the effects of de-icing salts used during winter or from seawater exposure of coastal structures. From a durability-design point of view, the development of a performance-based approach is critical to ensure adequate safety and serviceability throughout the design life of concrete structures such as those in chloride-laden environments, as well as to help identify appropriate maintenance strategies to extend their service life and minimize their risk of failure.

The current practice of design of durable concrete structures is mainly based on prescriptive rules, such as specifying minimum concrete cover, maximum water/cement ratio, use of corrosion-resistant reinforcing steels, limitation of crack width, etc. These rules are deemed to satisfy the design life requirements of safety and serviceability without providing an explicit relationship between performance and service life (RILEM Report 14 [6]). However, the observed extensive deterioration and failure of a large

number of reinforced and prestressed concrete structures (e.g., bridge decks and girders) in North America well before the end of their design life have illustrated the serious shortcomings of the current “deem-to-satisfy” prescriptive rules. Most of the studies found in the literature have focused on the investigation of the mechanisms of chloride transport in concrete and reinforcing steel corrosion, as well as development of corrosion protection, repair strategies and detection techniques; however, there are few studies on the development of reliable and practical design approaches for durability design and corrosion protection of concrete structures. In order to achieve a specified design life for concrete structures, it is imperative to develop mechanistic and practical models—deterministic or probabilistic [6–8]—that can be used for the design of durable concrete structures, as well as for predicting the deterioration and maintenance optimization of existing concrete structures.

In this paper, the emphasis is on the durability design of new concrete structures that are built in chloride-laden environments by using practical mechanistic models. These models represent quantitatively an adequate understanding of the physico-chemical processes that induce deterioration in concrete structures, namely: (i) simplified diffusion-based model for chloride ingress into concrete; and (ii) concept of chloride threshold value as a corrosion resistance indicator of the reinforcing steel. Using these simplified models, the quantitative relationships between the critical design parameters that govern the service life of concrete structures are

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determined and can be used as guidance to designers on how to select the appropriate values of design parameters to achieve a specified design life.

2. Simplified mechanistic models for durability design

2.1. Fickian diffusion model for chloride ingress into concrete

The corrosion of concrete structures can be described as a two-stage process of [3]: (i) corrosion initiation; and (ii) corrosion propagation. The corrosion initiation stage corresponds to the process of chloride ions (chlorides) ingress into concrete (and/or carbonation, which is outside the scope of this paper) while the steel remains passivated. The corrosion propagation stage starts after the initiation of active corrosion, and it often occurs after the chloride concentration on the steel surface reaches a certain threshold value and steel is depassivated. Therefore, the duration of the initial stage, which is often used as a quantitative indicator of service life or durability of concrete structures, depends on the rate of chloride penetration in concrete. The needs to predict the service life of existing structures and to design more durable structures, as well as to extend their service life through effective maintenance strategies, are the objectives of the extensive efforts for modeling chloride ingress, corrosion initiation, and corrosion propagation in concrete structures in the last three decades [1–6].

Aggressive agents such as chlorides, water, and oxygen penetrate into concrete through the pore spaces and micro-cracks in the cement paste matrix as well as at the paste/aggregate interface. The rate of penetration is dependent primarily on the quality of concrete and more particularly on the water–cement ratio of the concrete mix, the presence of supplementary cementing materials (e.g., silica fume, fly ash, or slag), and/or protective systems that delay or slow down chloride ingress. In porous solids, such as concrete, chlorides can penetrate concrete via different physical mechanisms, such as diffusion, capillary absorption, electrical migration, and permeation due to hydraulic pressure heads [27]. Furthermore, the penetration process is also influenced by chemical reactions such as chloride binding.

Among the multiple transport mechanisms, diffusion, and absorption have been extensively studied. Absorption mostly occurs in concrete surface layers that are subjected to wetting and drying cycles, and it only affects the exposed concrete surface down to 10–20 mm [9,10]. Beyond this absorption zone, the diffusion process will dominate [10]. Chloride ingress driven by pressure gradient, which has to be maintained by a hydraulic head, is rarely found in highway concrete structures and its influence can be minimal [11,12]. Recently, a more general ion transport process for predicting chloride ingress into concrete structures described by the Nernst–Planck Equation was used (i.e. [13]). This model, which is well established in traditional electrochemical science [14], describes the ion transport as a combination of three mechanisms, namely: (i) diffusion; (ii) convection (carried along by moisture movement, i.e. by absorption or hydraulic pressure); and (iii) electrical migration. Although the mechanism of electrical migration under an imposed electrical potential has been used to develop an accelerated method (Rapid Chloride Permeability Test) to determine the diffusion coefficient of concrete [15], the study of its contribution to chloride ingress under normal service condition is only recent [13].

Depending on the exposure condition as well as on the moisture content of the concrete element, the ingress of chlorides into concrete is through coupled multiple transport mechanisms [27]; however, most models for chloride ingress in concrete are still based on the assumption of a Fickian process of diffusion as the main transport mechanism. Moreover, diffusion-based models

have been found acceptable for long-term monitoring of chloride ingress in concrete structures [16,17]. The main differences between different diffusion-based models were summarized by Andrade et al. [18] and possible approaches for improving the prediction of chloride ingress were suggested by Nilsson [19], which include: (i) use of more appropriate boundary conditions; (ii) adoption of an age factor for parameters defining concrete properties such as diffusion coefficient; and (iii) consideration of more than one transport mechanism.

It should be pointed that the uncertainty of this Fickian model has been recognized for a long time, since it has been used with the so-called “apparent” values of the diffusion coefficient and surface chloride concentration, which are obtained by calibrating the Fickian model with available field data. Large amount of experimental data have shown that the diffusion coefficient depends on both concrete mix design, such as water/cement ratio, quantity of mineral admixtures, aggregate fraction, etc., and other parameters such as temperature and age of concrete [20,21]. Few standards, e.g., *Salt Ponding Test* and *Rapid Chloride Permeability Test*, have been developed over time to measure it as a material parameter or property for durability of concrete structures exposed to chlorides.

There is a need to make a distinction between the mechanistic models and data needed for the design of new structures and those needed for predicting the service life of existing structures. For the evaluation of the safety and serviceability of existing structures, it is critical to use the most rigorous models and data (obtained from measurements on the existing structures) to predict the service life and to compare it to the specified design life. This is because the consequences of erroneous predictions can be catastrophic, e.g., impaired structural safety and potential collapse. For the design of new structures, a useful model should have parameters that either are design parameters or have (empirical) correlations with design parameters, e.g., chloride diffusion coefficient and concrete mix design. A reliable yet practical model such as the diffusion-based model can be therefore assumed acceptable, as long as the uncertainties in the model and design parameters are taken into account. This ensures acceptable levels of performance and probabilities of failure in terms of safety and serviceability within the design life of the structures [6,7].

In this paper, the focus is on the use of the diffusion-based model as a quantitative approach to service life design of concrete structures that overcomes some of the limitations of the current prescriptive rules-based approach, as well as the complexities and lack of data associated with the use of models based on multi-transport mechanisms. Multi-mechanism models will be useful for predicting the service life of existing structures as well as for design purposes, when large number of experimental data become available in the future to establish an empirical correlation of model parameters with design parameters, as was done for correlating the chloride diffusion coefficient with concrete mix design parameters over the last two decades.

2.2. Simplified diffusion-based service life design model

The onset of corrosion of the reinforcing steel is assumed to occur when the concentration of chlorides at the level of the reinforcement has reached the so-called “chloride threshold value”. Therefore, the duration of the initiation stage is from time zero to the onset of corrosion. Based on the concept of chloride threshold value for corrosion initiation and the assumption of a Fickian process of chloride diffusion as discussed above, a deterministic model of the time to corrosion initiation can be expressed as a function of four key parameters [3,8] as follows:

$$T_i = f(C_s, C_{th}, D, d_c) = \frac{d_c^2}{4D \left[\operatorname{erf}^{-1} \left(1 - \frac{C_{th}}{C_s} \right) \right]^2} \quad (1)$$

where T_i is the time to onset of corrosion, C_s is the surface chloride concentration, C_{th} is the chloride threshold value, D is the chloride diffusion coefficient, d_c is the depth of concrete cover over the reinforcing steel, and erf is the error function. The model is a transformation of Crank's solution to Fick's second law of diffusion for one-dimensional flow over a semi-infinite medium [22] with the mathematical interpretation of Tuutti's corrosion initiation model [8], under the assumptions of a constant diffusion coefficient, constant surface chloride content as the boundary condition, and the initial chloride concentration condition specified as zero.

In this deterministic model (Eq. (1)), the four parameters are assumed to be four independent design parameters that are critical for corrosion protection: (i) structural parameter: concrete cover depth d_c ; (ii) material parameters: chloride diffusion coefficient D , which is an indicator of the rate of chloride penetration into concrete, and chloride threshold value C_{th} , which is an indicator of the corrosion resistance of reinforcing steel; and (iii) environmental parameter: surface chloride concentration C_s , which is a measure of the corrosion load or exposure risk on concrete structures.

This model was used to study the sensitivity of the time to corrosion initiation to each of the four parameters [8]. This paper will explore the quantitative relationships between pairs of the governing parameters, when the time to corrosion initiation, which is defined as the design life or service life, is specified. For example, the relationship between concrete cover depth and chloride threshold value of steel can be solved through Eq. (1), which can be a guide for choosing appropriate concrete cover depth and type of steel that offers the required chloride threshold value. In this paper, the emphasis is on the physical performance, e.g., durability in terms of the length of corrosion initiation stage, and no consideration is given to cost implications. It should be pointed out that for a truly effective design of concrete structures, the cost implications of changing the different design parameters should be explicitly considered in order to determine the optimal combination of values that satisfy the design life and minimize the life cycle costs of concrete structures, including the costs of design, construction, maintenance, rehabilitation, etc. [26].

In order to evaluate the relationships between these four parameters for durability design, it is easier to examine the relationships by re-writing Eq. (1) as follows:

$$1 - \frac{C_{th}}{C_s} = \operatorname{erf} \left(\frac{d_c}{2\sqrt{DT_i}} \right) \quad (2)$$

Eq. (2) indicates that a relationship between two variables on the same side of the equation will be easily obtained for a specified design life (T_i), such as C_{th} and C_s or d_c and D , for which an analytical solution can be obtained. Eq. (2) shows that C_s and C_{th} are linearly related, which means that the corrosion resistance of embedded steel has to be chosen proportional to the severity of the surface exposure condition in order to achieve a specified design life. Two variables on either side of the equation are related through the error function and their relationships are highly nonlinear. Given the complexity of the error function and its inverse, a numerical solution of the equation $y = \operatorname{erf}(x)$, where $x = \frac{d_c}{2\sqrt{DT_i}}$ and $y = 1 - \frac{C_{th}}{C_s}$, will be used to determine these relationships (e.g., the numerical relationship between $\frac{4DT_i}{d_c^2}$, equivalent to $1/x^2$ vs. $\frac{C_{th}}{C_s}$, equivalent to $1 - y$, is obtained by using math tables or common computer spreadsheets).

In addition to the relationships derived from Eq. (2), the sensitivity between parameters is also explored to take into account the considerable level of uncertainty and variability associated with

each of the four governing parameters [8]. This parameter uncertainty is due to: (i) heterogeneity and aging of concrete with temporal and spatial variability of its chloride diffusivity; (ii) variability of concrete cover depth, which depends on quality control, workmanship, and size of structure; (iii) variability of surface chloride concentration, which depends on the severity of the environmental exposure; and (iv) uncertainty in chloride threshold value that depends on the type of reinforcing steel, type of cementing materials, test methods, etc. [16]. The study of sensitivity enables to illustrate the impact of variation (or uncertainty) of one parameter on another, and how the value of one parameter needs to be adjusted to compensate for the variation in another parameter, especially for the case of negative impacts on service life. This is done through a numerical evaluation of the relationships of pairs of parameters by determining their respective relative changes when other parameters are kept constant [8].

3. Relationships between governing parameters

3.1. Range of values and variations of parameters

Realistic ranges of values of the four governing parameters, C_s , C_{th} , D , and d_c , are summarized below, with emphasis on highway bridge deck applications [8]. A value of 50 years for T_i is assumed as the specific design life throughout the numerical illustrations in this paper.

The chloride threshold C_{th} is defined as the chloride concentration at which the passivation of the steel in concrete will be destroyed. It depends on the type of steel, electrochemical environment in concrete, and testing method and consequently, a considerable variation is expected for this parameter. Different types of steel have different C_{th} values, and even one type of steel (e.g., carbon steel) has a wide range of reported values of C_{th} [16]. In this paper, the values obtained from the same research study are used in order to minimize the variation induced by testing methods. The values of C_{th} are taken from the reported values from Trejo and Pillai [23,24], with the mean values of C_{th} for carbon steel, micro-composite steel, stainless steel SS304 and SS316LN, being 0.52 kg/m³, 4.6 kg/m³, 5.0 kg/m³, and 10.8 kg/m³, respectively.

The concrete cover depth is a design parameter that varies with the severity of environmental exposure. The variability of concrete cover depends on the quality of workmanship and control during construction. A constant value of d_c is chosen to be 50 mm when studying the relationships between other parameters, and the associated variation of d_c is between 20 mm and 80 mm, which is $\pm 60\%$, representing a range of cast-in-place bridge decks built in the sixties and seventies [7].

The chloride diffusion coefficient D of concrete is a material property that is a measure of the rate of chloride penetration into concrete. The value of D depends on both concrete mix design, such as water/cement ratio, quantity of mineral admixtures, aggregate fraction, etc., and other parameters such as temperature and age of concrete [16,20,21]. The typical values of D for normal concrete (e.g., $w/c = 0.4$) were reported between 10^{-12} m²/s and 10^{-11} m²/s [21,25], e.g., $D = 4 \times 10^{-12}$ m²/s for a normal concrete with $w/c = 0.4$ and $D = 10^{-11}$ m²/s for $w/c = 0.55$ [25]. The incorporation of mineral admixtures (e.g., fly ash, slag, silica fume), however, can lead to orders of magnitude reduction of D in the long term [21]. In this study, D of normal concrete is considered to vary from 10^{-12} m²/s to 10^{-11} m²/s. Lower values of D from 10^{-13} m²/s to 10^{-12} m²/s are referred as high-performance concrete (HPC) only in terms of resistance to chloride penetration in this paper. It should be noted that the possible mechanism of chloride transport through potential cracking in concrete (e.g., caused by early-age

autogeneous shrinkage in HPC or due to service load on structures) is not considered in the model.

Weyers et al. [9] classified the corrosive environments surrounding bridge decks into four categories in terms of surface chloride concentration: *Light, Moderate, Heavy, and Severe Exposures*. In this paper, the mean value for each exposure condition is used when a constant C_s value is needed to study, for example, the relationships of C_{th} vs. d_c under these exposure conditions. The exposure condition is deemed as a corrosion load for concrete structures with a constant value for each of the exposure conditions. The constant values C_s are 1.8 kg/m^3 , 3.5 kg/m^3 , 5.3 kg/m^3 , and 7.4 kg/m^3 , respectively, for the four exposure conditions.

3.2. Relationship between design parameters

The selection of an appropriate concrete cover depth (d_c) and type of steel with sufficient corrosion resistance can be determined by the relationship between d_c and C_{th} . Their relationship will be

affected by chloride diffusion coefficient and exposure condition. Fig. 1 illustrates this relationship under four exposure conditions when $D = 10^{-12} \text{ m}^2/\text{s}$, which implies a low chloride diffusion coefficient of either a high-quality normal concrete or an HPC. As expected, C_{th} increases as d_c decreases. The slopes of these curves indicate that increasing d_c is more effective under heavier exposure to chlorides, while increasing C_{th} is more effective under lighter exposure conditions. For example, under “*Light Exposure*”, the concrete cover has to be 60 mm for carbon steel with $C_{th} = 0.52 \text{ kg/m}^3$ in order to achieve a design life of 50 years, but only 30 mm if using a steel with $C_{th} = 1.0 \text{ kg/m}^3$. On the other hand, under “*Severe Exposure*”, the chloride threshold needs to be increased from about 2.0 kg/m^3 to 4.6 kg/m^3 (e.g., that of a micro-composite steel) if the cover depth is reduced from 60 mm to 30 mm.

Fig. 2 illustrates the relationship between d_c and C_{th} under “*Heavy Exposure*” condition for different D values, which ranges from those of normal concrete to HPC. The slopes of the curves indicate that increasing d_c is more effective in HPC than in normal concrete.

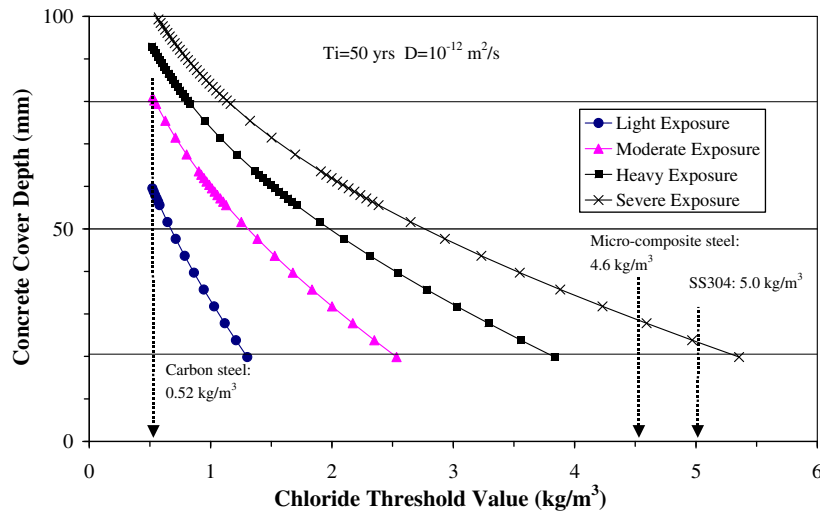


Fig. 1. Concrete cover vs. chloride threshold value for different environments.

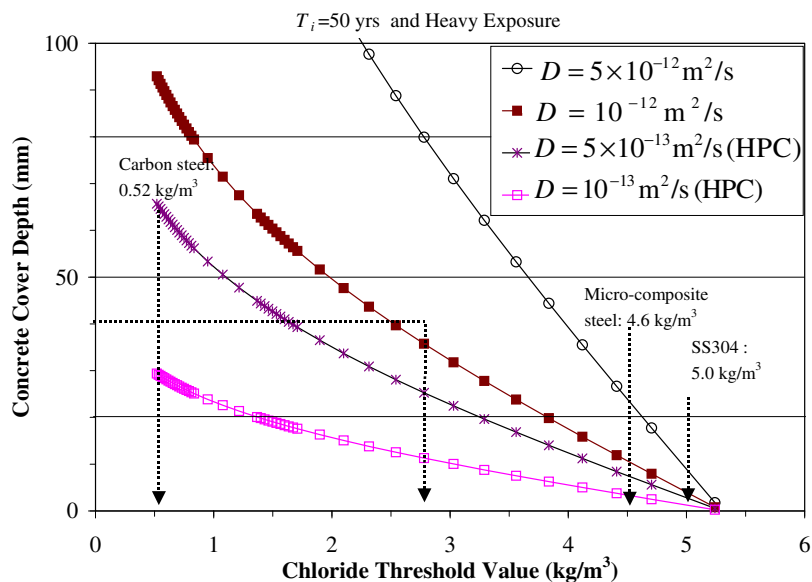


Fig. 2. Concrete cover vs. chloride threshold value for different concretes.

For a concrete with $D = 5 \times 10^{-13} \text{ m}^2/\text{s}$ (e.g., HPC) an increase of d_c to about 65 mm can bring down the requirement on C_{th} to 0.52 kg/m^3 (that of carbon steel) even under “Heavy Exposure” condition. The d_c – C_{th} relationship also provides other useful information for design: Fig. 2 shows that if the design value of d_c is less than 80 mm for “Heavy Exposure” condition and carbon steel is to be used, the use of a concrete with $D < 10^{-12} \text{ m}^2/\text{s}$ (e.g., that of HPC) should be considered.

Fig. 3 presents the relationship between D and d_c by determining the curves for different combinations of steel type (characterized by C_{th}) and exposure conditions. For corrosion resistant steels under *Severe Exposures*, the required concrete cover increases at a relatively fast rate as D increases. For example, an increase in D from $10^{-12} \text{ m}^2/\text{s}$ (HPC) to $10^{-11} \text{ m}^2/\text{s}$ (normal concrete) requires an increase in the cover depth from 30 mm to about 90 mm for Micro-composite steel under “Severe Exposure”. On the

other hand, for corrosion-resistant steels under *Heavy Exposure*, the required concrete cover increases at a much slower rate as D increases. Fig. 3 also shows that curves between D and d_c can be generated in different ways to meet specific design purposes and serve as guidelines for choosing different combinations of values for the design parameters governing service life.

The relationship between D and C_{th} , i.e. between concrete diffusion coefficient and corrosion resistance of steel is affected by design of concrete cover depth and exposure condition. Fig. 4 illustrates the relationship under “Heavy Exposure” for different d_c values. An increase of d_c from 20 mm to 80 mm can reduce the requirement on C_{th} from about 3.6 kg/m^3 to 0.52 kg/m^3 (that of carbon steel) in HPC, as indicated by line A, for a value of D slightly lower than $10^{-12} \text{ m}^2/\text{s}$; however, in normal concrete it will reduce the requirement on C_{th} slightly from about 4.8 kg/m^3 (about that of Micro-composite steel or SS 304) only to 3.3 kg/m^3 , as indicated by

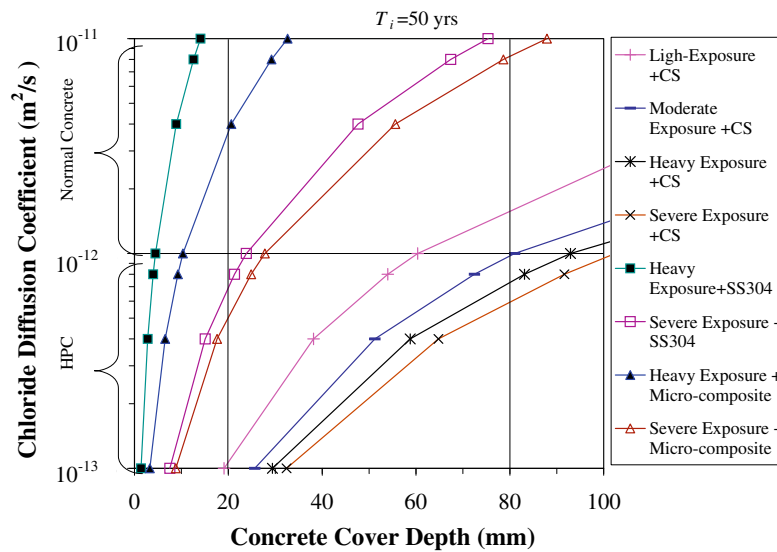


Fig. 3. Concrete cover vs. concrete diffusion coefficient for different environments and types of steel.

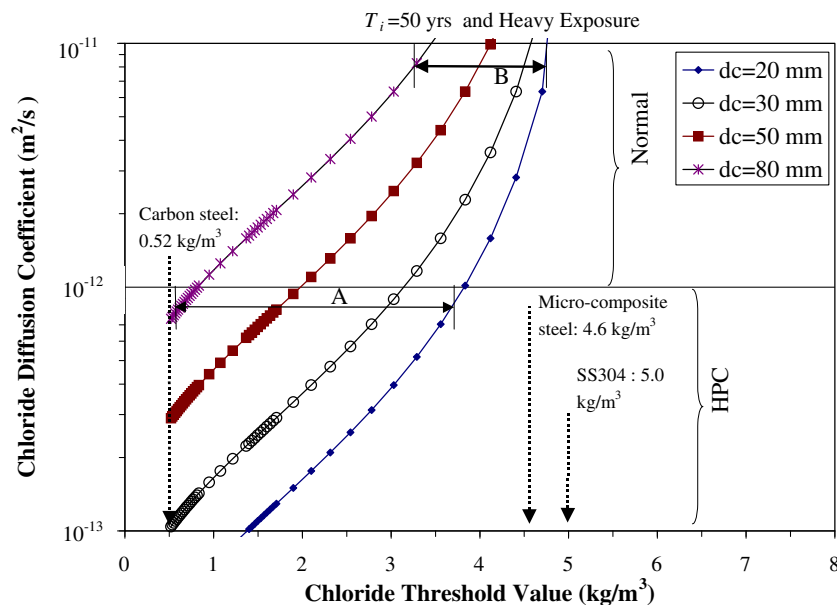


Fig. 4. Chloride threshold vs. concrete diffusion coefficient for different concrete covers.

line B, for a value of D slightly lower than $10^{-11} \text{ m}^2/\text{s}$. This means that a larger D value of normal concrete results in a limited choice of types of steel that can be used to achieve the design life. Their relationship also suggests that the use of a steel with C_{th} around 5.0 kg/m^3 (e.g., that of stainless steel 304) under “Heavy Exposure” will help achieve $T_i = 50$ years, even for a low cover ($d_c = 20 \text{ mm}$) and normal concrete ($D = 10^{-11} \text{ m}^2/\text{s}$).

Fig. 5 illustrates the relationship of d_c and C_s for concrete with $D = 10^{-12} \text{ m}^2/\text{s}$, but for different types of steel. It clearly shows the necessity to use both HPC and high-performance steel for both “Heavy Exposure” and “Severe Exposure” conditions; e.g., the micro-composite and SS304 steels can be used under “Severe Exposure” without increasing concrete cover above 50 mm . This relationship can also be used to identify a threshold value of C_{th} of steel that can be used to provide the required resistance under a specified exposure condition, and Fig. 5 shows that a value of about 3.4 kg/m^3 for

C_{th} is needed under exposure conditions if the concrete cover is less than 50 mm . Therefore, the requirement on the corrosion resistance of steel can be conveniently estimated for design by using the developed curves. This is further illustrated in the relationship of D and C_s in Fig. 6. It shows that for a concrete cover of 50 mm , C_{th} needs to be greater than about 3.6 kg/m^3 to achieve a design life of 50 years under “Heavy Exposure” condition for a normal concrete; otherwise, a concrete (HPC) with $D < 2 \times 10^{-13} \text{ m}^2/\text{s}$ should be specified if carbon steel is to be used.

3.3. Relative sensitivities of service life parameters

Consider the cover depth (d_c) of a concrete structure with a mean value of 50 mm and a range variation from 20 mm (–60%) to 80 mm (60%); the sensitivity of C_{th} to the variation in d_c is presented in Fig. 7. It shows the necessary relative change in C_{th} (by

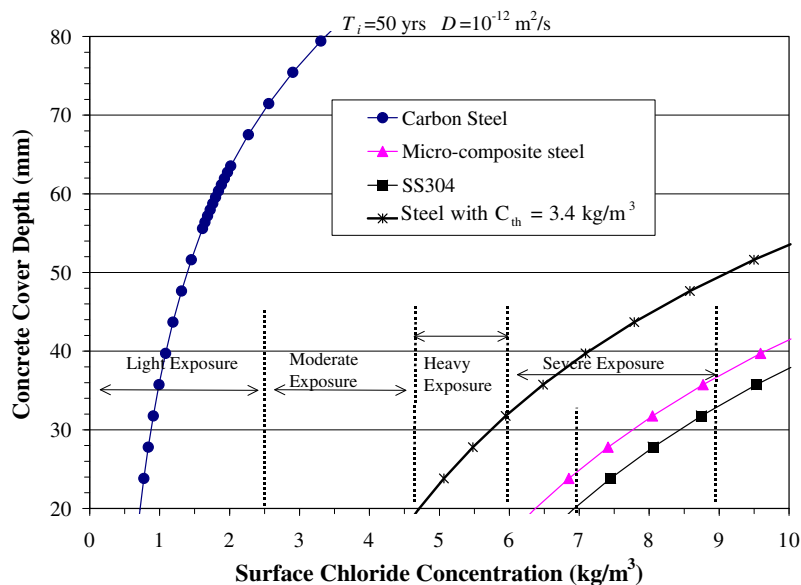


Fig. 5. Concrete cover vs. surface chloride exposure for different types of steel.

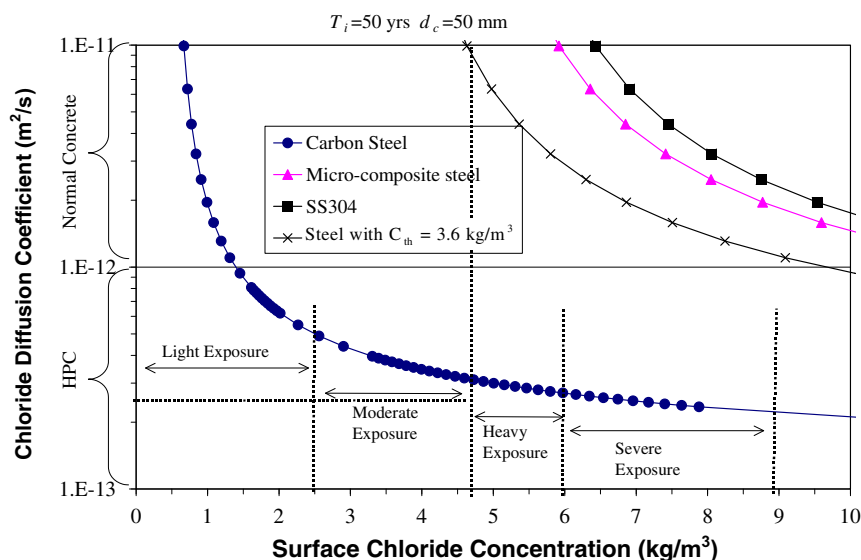


Fig. 6. Concrete diffusion coefficient vs. surface chloride exposure for different types of steel.

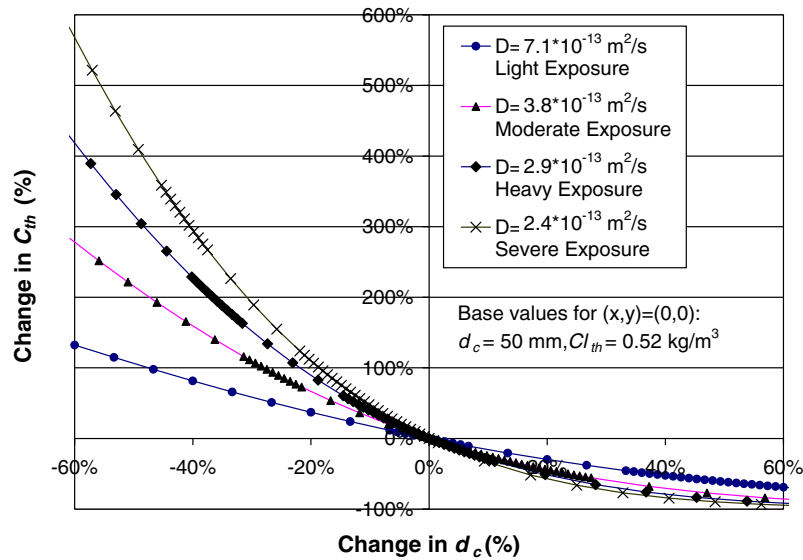


Fig. 7. Sensitivity of chloride threshold value to variation in concrete cover depth.

selecting the appropriate type of steel) to satisfy the specified design life of $T_i = 50$ years under four different exposure conditions. The reference values for the curves are $T_i = 50$ years, $C_{th} = 0.52$ kg/m³ (carbon steel), and $d_c = 50$ mm, and the values of D are 7.1×10^{-13} m²/s, 3.8×10^{-13} m²/s, 2.9×10^{-13} m²/s, 2.4×10^{-13} m²/s, respectively, for the four exposure conditions from “Light Exposure” to “Severe Exposure”. The relative change in C_{th} means by how much its value has to be adjusted from the base value of a carbon steel, in order to compensate for the effect of change in d_c , especially in the case of a negative change in d_c , and satisfy the design life of 50 years. It can be seen that the variation in d_c has a considerable impact on C_{th} . A value of $d_c = 20$ mm (i.e. -60% of the base value of 50 mm) will require the use of a steel with a corrosion resistance about five to six times higher than that of carbon steel. Furthermore, the four curves show that C_{th} is more sensitive to the variation in d_c under more severe

exposure conditions, which means that for a decrease in concrete cover depth, the corrosion resistance of steel has to be increased more for severe exposure conditions. If d_c is reduced by 60%, it requires an increase in C_{th} by about 5.8 times under “Severe Exposure” and an increase by about 1.3 times under “Light Exposure.”

On the other hand, the sensitivity of d_c to C_{th} is not as much. Fig. 8 was plotted from the same data as those in Fig. 8 but with reversed x and y axes to focus on the range of variation in C_{th} , which is $\pm 40\%$ from the base value for carbon steel. It shows that the variation in C_{th} causes the necessary change in d_c by about 30% to -20% for “Light Exposure”, which means an increase in d_c from 50 mm to 65 mm will suffice to compensate the adverse effect of a reduction in C_{th} by 40%. Also in reverse, d_c is more sensitive to variation in C_{th} under lighter exposure conditions. For a relative decrease in chloride threshold value, the relative increase in concrete cover is much higher for lighter conditions. If C_{th} is reduced

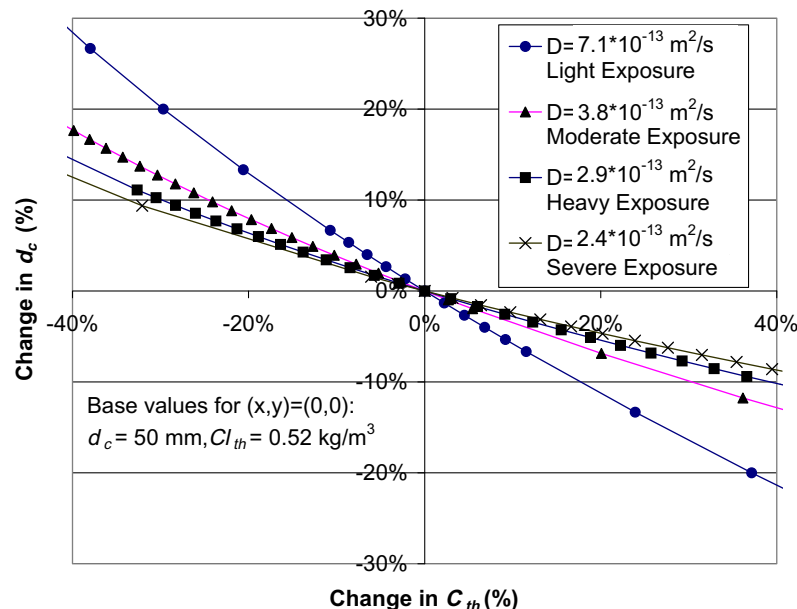


Fig. 8. Sensitivity of concrete cover depth to variation in chloride threshold value.

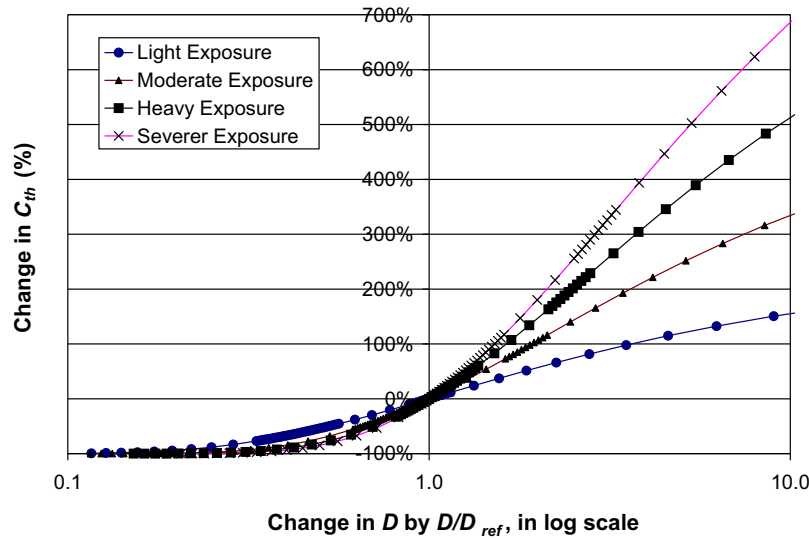


Fig. 9. Sensitivity of chloride threshold value to variation in chloride diffusion coefficient.

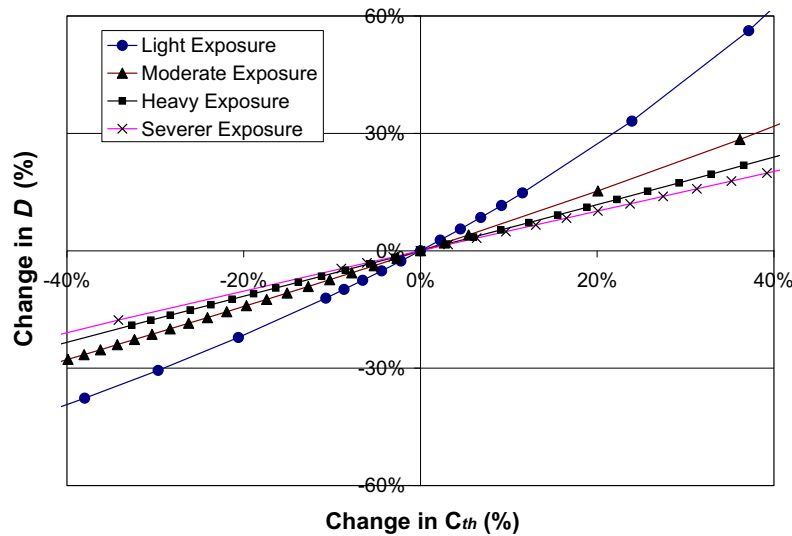


Fig. 10. Sensitivity of chloride diffusion coefficient to variation in chloride threshold.

by 40%, it requires an increase in d_c by about 18% under “Severe Exposure” (low D) and an increase by about 30% under “Light Exposure” (high D). Note for Figs. 7 and 8, T_i is kept a constant 50 years for different exposure conditions, which is the main interest of the study in this paper, instead of keeping D constant.

Fig. 9 shows the sensitivity of C_{th} to the variation in D . The reference values are $T_i = 50$ years, $d_c = 50$ mm, and $C_{th} = 0.52$ kg/m³ (carbon steel) and the reference values of D (noted in Fig. 9 as D_{ref}) are, 7.1×10^{-13} m²/s, 3.8×10^{-13} m²/s, 2.9×10^{-13} m²/s, 2.4×10^{-13} m²/s, respectively, for the four exposure conditions from light to severe. In Fig. 10, instead of showing the change by percentage (which is $\Delta D/D = (D - D_{ref})/D_{ref}$), the variation is illustrated by D/D_{ref} . This is because the change of D can be significant on a linear scale and up to one order of magnitude. The ratios D/D_{ref} 0.1 and 10 represent the variation by one order of magnitude for decreasing and increasing, respectively. It is clear that D has a significant impact on C_{th} : an increase in D by one order of magnitude

(e.g., from HPC to normal concrete) requires an increase in C_{th} by about 6.9 times under “Severe Exposure”. Furthermore, C_{th} is more sensitive to the variation in D under more severe exposure conditions, which means for an increase in chloride diffusion coefficient, the relative increase in corrosion resistance of steel has to be higher for more severe exposure conditions. If D is increased by one order of magnitude (e.g., from 7.1×10^{-13} m²/s to 7.1×10^{-12} m²/s), it requires an increase in C_{th} by about 6.9 times under “Severe Exposure” and an increase by about 1.6 times under “Light Exposure.”

Fig. 10 was plotted from the same data as in Fig. 9 but with reversed x and y axes to focus on relative change in C_{th} (from the base value of carbon steel by $\pm 40\%$) vs. the relative change of D . The sensitivity of D to C_{th} is less significant than the sensitivity of C_{th} to D . It shows that the variation in C_{th} causes the necessary change in D by about 60% to -40% for “Light Exposure”, which means that a decrease in D from 7.1×10^{-13} m²/s to 4.3×10^{-13} m²/s will suffice to compensate the adverse effect of a reduction in C_{th} by 40%. On

the other hand, D is more sensitive to the variation in C_{th} under lighter exposure conditions, which means that for a reduction in chloride threshold value, the relative decrease of chloride diffusion coefficient has to be higher for decreasing severity of exposure environments. If C_{th} is reduced by 40%, it requires a decrease in D by about 20% under “Severe Exposure” and a decrease by about 40% under “Light Exposure”.

The sensitivity between d_c and D are more straightforward because they are not related by the error function (Eq. (1)), and the percentage change of $\Delta d_c/d_c$ vs. $\Delta D/D$ can be obtained analytically when other parameters are kept constant as follows:

$$\frac{\Delta D}{D} = 2 \frac{\Delta d_c}{d_c} + \left(\frac{\Delta d_c}{d_c} \right)^2 \quad (3)$$

Eq. (3) shows that for small variations of d_c (e.g., less than 10%), the relative change ($\Delta D/D$) is approximately twice, i.e. a 10% decrease in d_c will require decreasing D by 21%. The increase and decrease in D are asymmetrical with d_c ; an increase in d_c will induce a much greater increase in D than a decrease of d_c for the same percentage, e.g., variation in d_c from –60% to 60% will result in the change of D from –84% to 156%. Considering that variation in D can be up to one order of magnitude, an increase by one order of magnitude ($\Delta D/D = 9$, e.g., from 10^{-13} m²/s to 10^{-12} m²/s) will require an increase in d_c by 2.16 times (e.g., from 50 mm to 108 mm); a decrease by one order of magnitude ($\Delta D/D = -0.1$, e.g., from 10^{-12} m²/s to 10^{-13} m²/s) can reduce d_c by 68% (e.g., from 50 mm to 16 mm). It implies that the quality of concrete mix in terms of chloride permeability can have a significant impact on selecting the concrete cover depth to achieve the specified design life. By choosing HPC over normal concrete, assuming its D is reduced by one order of magnitude, the concrete cover depth can be reduced by 68%. On the other hand, choosing a normal concrete over an HPC, assuming its D is increased by one order of magnitude, the concrete cover depth must be more than doubled.

4. Summary and conclusions

This paper presented an approach toward performance-based durability design of concrete structures based on a simplified diffusion-based model, which overcomes some of the limitations of the current prescriptive rules of existing design standards. A quantitative evaluation of the nonlinear relationships and relative sensitivities between the four governing parameters for service life design and corrosion protection of concrete structures, including surface chloride concentration, chloride diffusion coefficient, chloride threshold value of steel, and concrete cover depth have been presented.

The relationships can be used to identify the parameters that are more effective in achieving a specific design life, which is defined as the time to corrosion initiation in this paper. It was found that increasing concrete cover depth is more effective under heavier chloride exposure conditions, while selecting a type of reinforcing steel with higher chloride threshold value is more effective under lighter exposure conditions. Under heavy chloride exposure conditions, increasing concrete cover depth is more effective in HPC (chloride diffusion coefficient from 10^{-13} m²/s to 10^{-12} m²/s) than in normal concrete (chloride diffusion coefficient from 10^{-12} m²/s to 10^{-11} m²/s).

The relationships can also be used as guidelines for a selection of an adequate value for one parameter or for choosing the most appropriate combination of values of the governing parameters to achieve a specified design life for concrete structures. The main findings can be summarized as follows: (i) The relationship between concrete cover depth and exposure condition (in terms of surface chloride concentration) reveals the necessity of using both

HPC with chloride diffusion coefficient less than 10^{-12} m²/s and high-performance steel under heavy chloride exposure conditions. This relationship can also be used to identify the appropriate type of steel with the required corrosion resistance that can withstand a specified exposure condition; and similarly (ii) the requirement on the concrete mix design in terms of chloride diffusion coefficient can be estimated by using the relationship between chloride diffusion coefficient and surface chloride concentration.

The study of relative sensitivities illustrated how the value of one parameter needs to be adjusted to compensate for the variation in another parameter to achieve the specified design life. It was found that chloride threshold value is more sensitive to concrete cover depth. It means that for a relative decrease in concrete cover depth, the relative increase in corrosion resistance of steel has to be higher by choosing an appropriate type of steel. Similarly, a variation in chloride diffusion coefficient, which depends on the design and quality control of concrete mix, has a significant impact on chloride threshold value as well, especially under severe chloride exposure conditions. If chloride diffusion coefficient is increased by one order of magnitude (e.g., from 7.1×10^{-13} m²/s to 7.1×10^{-12} m²/s), it requires an increase in chloride threshold value by about 6.9 times by choosing an appropriate type of steel under severe chloride exposure and an increase by about 1.6 times under light chloride exposure, in order to compensate for the adverse effect and satisfy the specified design life.

The sensitivity study also implies that the design and quality of concrete mix in terms of chloride permeability has a significant impact on the design of concrete cover depth. By choosing an HPC over a normal concrete, assuming its chloride diffusion coefficient is reduced by one order of magnitude, the concrete cover depth can be reduced by 68%. On the other hand, choosing a normal concrete over an HPC and assuming that its chloride diffusion coefficient is increased by one order of magnitude, the concrete cover depth must be doubled.

Note that the results presented in this paper are illustrated for the case of a reinforced concrete structure with a specified design life of 50 years and the quantitative correlations of the parameters for other specified design lives can be obtained by using the methodology presented in this paper. When more rigorous mechanistic models are used at the design stage, a similar approach can be adopted to study the nonlinear relationships and sensitivities between the model parameters that are correlated with design parameters, and the use of more rigorous mechanistic models will overcome the potential shortcomings of the simplified diffusion models. In addition, a study of the costs of changing the different design parameters and their impacts on the life cycle costs of concrete structures is needed to determine a truly optimal service life design of concrete structures that are subjected to chlorides.

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