



REVIEW

On factors affecting probabilistic service life modeling of concrete structures under marine environments

Fatores que afetam a modelagem probabilística da vida útil de estruturas de concreto armado expostas a ambientes marinhos

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Abstract: Concrete durability design has received increasing importance recently, with specifications moving from prescriptive to performance based. In performance-based approaches, it is essential to evaluate and calibrate service life models capable of reliably representing the phenomenon that triggers the degradation process. This paper aims to discuss the main concepts related to the probabilistic service life modeling of reinforced concrete structures under chloride environments, considering the application of different prediction models. Through numerical analysis, parametric differences among chloride penetration models are evidenced, and the results, their variability, and the admitted failure conditions are analyzed. An overview of the current scenario of the durability design of concrete structures is presented. Aspects associated with characteristic service life, the definitions of durability limit states, and their respective target failure probabilities are discussed.

Keywords: service life modeling, performance-based approach, durability limit states, probability-based design.

Resumo: O projeto de durabilidade de estruturas de concreto tem ganhado crescente importância nos últimos anos, havendo a transição das especificações prescritivas às baseadas em desempenho. No contexto das abordagens de desempenho, é essencial avaliar e calibrar modelos de previsão de vida útil capazes de representar o fenômeno que desencadeia o processo de degradação. Este artigo visa discutir os principais conceitos relacionados à modelagem da vida útil de estruturas de concreto armado expostas a ambientes ricos em cloretos, considerando a aplicação de diferentes modelos de estimativa da penetração de cloretos no concreto. Através de uma análise numérica, diferenças paramétricas entre os modelos são evidenciadas, bem como são analisados os resultados, suas variabilidades e as condições de falha admitidas. Uma visão geral do atual cenário dos projetos de durabilidade de estruturas de concreto é apresentada. Aspectos associados ao conceito de vida útil característica, às definições de estados limites de durabilidade e suas respectivas probabilidades de falha admissíveis são discutidos.

Palavras-chave: modelagem de vida útil, abordagem baseada em desempenho, estados limite de durabilidade, análise probabilística.

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1 INTRODUCTION

The sustainability of cement-based materials is a hot topic nowadays, primarily due to the carbon footprint of cement industries [1]–[3]. However, in the case of reinforced concrete structures, sustainability does not only comprise the production of concrete and its constituent materials but also involves strength, durability, performance, service life, and the life cycle of the structural elements [4]. Ensuring durability, therefore, is a crucial point for concrete sustainability [5].

Regarding reinforced concrete structures exposed to marine environments, many structural elements have had a service life shorter than the designed service life due to the various attacks from seawater, mainly due to chloride-induced reinforcement corrosion [6]. In several cases, in addition to the environmental aggressiveness, the concrete quality does not meet minimum parameters, reducing the service life of the structures. For these reasons, concrete durability specifications are moving from prescriptive to performance-based approaches [7].

In a performance-based approach, at least one parameter directly related to the concrete durability must be assessed – for example, the chloride diffusivity in the case of structures in marine environments. Additionally, the service life of the structure must be modeled considering the characteristics of the placed concrete, the environmental aggressiveness, and the durability limit states (DLS) [8], [9]. The definition of the service life model to be adopted, as well as the considered failure criterion, however, are quite complex. Incorrect selection of these factors can lead to significantly different service life predictions. Added to these facts is the difficulty of establishing long-term verification processes for the estimates made by each model, contributing to the increase in uncertainties related to the estimated service life of reinforced concrete structures.

This paper presents an overview of the current scenario of the durability design of concrete structures. The main parameters related to concrete service life prediction are analyzed through a probabilistic assessment. Three well-known chloride penetration models are addressed – namely, the Duracon model [10], the *fib* model [11], and the *Life-365* model [12]. Emphasis is placed on the influence of uncertainties in concrete durability design, especially related to concrete properties, cover depth, and environmental characteristics.

1.1 Factors affecting chloride penetration in concrete – a brief review

Chloride penetration in concrete is a complex process, affected by several factors. The durability of reinforced concrete structures against chloride penetration depends fundamentally on the characteristics of the reinforcement cover, both on the concrete properties and the cover depth. Additionally, the proper construction procedure of the structure and the adequate consideration of environmental aggressiveness are fundamental to guarantee concrete durability.

For example, the solution of Fick's 2nd Law of Diffusion [13], widely adopted in estimating chloride penetration in concrete, synthesizes the environmental aggressiveness in a parameter called surface chloride content (C_s) and summarizes the structural characteristics in chloride diffusion coefficient (D) and cover depth (x_c).

In general terms, the durability potential of a reinforced concrete structure is, therefore, a relationship between the environmental aggressiveness and the resistance presented by the structure, as illustrated in Figure 1. It is necessary to realize, however, that these parameters can be influenced by several other factors, such as, for example, characteristics of the constituent materials of concrete, temperature, relative humidity, and the structure construction process. Several chloride penetration models seek to introduce the influence of these parameters in estimating service life. However, there is great difficulty in accurately measuring how each factor affects the process of chloride penetration. Such effects are discussed below.

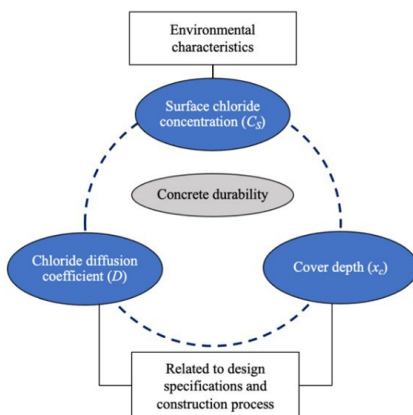


Figure 1. Leading factors affecting chloride penetration in concrete.

1.1.1 Chloride diffusion coefficient

In the case of durability analyses, the main concrete characteristic to be addressed is its penetrability. Concrete penetrability is how the concrete allows ions or fluids to move through its pore structure. Therefore, penetrability comprises transport mechanisms such as diffusion, permeability, capillary absorption, and migration. Despite this, most models that aim to describe chloride penetration in concrete adopt the diffusion process as the main responsible for the ingress of these ions through the pore structure of concrete. Thus, the chloride diffusion coefficient of concrete is considered the primary indicator of concrete performance against the action of these ions.

It is known that several factors affect concrete diffusivity, and these can be related to the concrete mix design, such as the water/binder ratio [14], [15], cement type and content [16], [17], and the use of mineral admixtures [18]–[20]. Other factors are related to concrete curing [21]–[24]. Some are related to characteristics of the environment in which the structure is inserted, such as temperature [25], [26], relative humidity [27], [28], and concrete saturation degree [29], [30]. These influences, however, seem to be clear and well established in the concrete production chain.

Special attention is given to test methods to determine the chloride penetrability in concrete. When chloride penetration into concrete is evaluated using diffusion-based test methods, long periods are required. From a technical perspective, however, test methods linked to durability-related properties must be easy and quick to perform, facilitating quality control of the placed material and decision-making in cases of non-compliance. Therefore, several migration-based test methods have been proposed and used in concrete specification and quality control, as discussed by Bjegović et al. [31], Nanukuttan et al. [32], and Milla et al. [33].

Using different test methods to determine the chloride penetrability in the concrete, however, lead to different results for the same concrete. This variation in the results was observed, for example, by Castellote and Andrade [34] and Sell Junior et al. [35]. Thus, it is essential that when specifying a specific target value for the chloride penetrability in a durability design, there is a clear indication of which test method should be adopted and the definition of which coefficient should be considered - whether effective or apparent, for example. It should also be noted that, regardless of the test method adopted, it is necessary to understand how the results obtained relate to the concrete performance *in situ*.

In addition to determining the diffusion coefficient, it is essential to consider the time dependence of the diffusion coefficient when estimating chloride penetration in reinforced concrete structures. This reduction in diffusivity is mainly due to the refinement of the concrete pore structure during the cement hydration process and possible pozzolanic reactions. It depends, among other factors, on the w/b ratio, the cement type and content, and the mineral admixtures type and content [36], [37]. This topic has been the subject of several discussions [38]–[40] and has contributed to the increase in uncertainties about estimating the service life of reinforced concrete elements. The calibration of the concrete diffusivity reduction level requires long-term analyses, which are more complex to perform. However, it should be noted that using the diffusion coefficient measured in the first ages as an input parameter in service life prediction models is a factor that favors safety. Many phenomena that occur in the first years after the structure's construction tend to reduce the chloride diffusivity.

Although the decisions made during the structure design are of great importance for ensuring concrete durability, the processes of concrete production and construction also strongly influence the durability potential of a reinforced concrete structure. For example, according to Helene and Terzian [41], the influence of labor on concrete properties, including variability in time and mixing procedure, is on the order of 30%. Depending on several factors related to concrete production and placing, the quality achieved by the placed concrete can present an even more significant variability.

Magalhães et al. [42], in turn, highlight that gross errors that affect the properties of concrete in a generalized way, such as overdoses of additives, excess water, or failures in cement weighing, are more easily detected. However, systematic variations in the production process tend to be more difficult to identify and correct. Also, the effects of slump corrections carried out without criteria, which strongly influence the porosity and, consequently, the diffusivity of the concrete, cannot be neglected. In the Brazilian context, NBR 7212 [43] indicates that a strict system must be established to control and record the amount of water added to the concrete at the plant and the complementation to be carried out at the construction site to avoid excess water. In some Brazilian constructions, however, the requirement of additional water to facilitate the concrete placing persists by those responsible for the placing process.

1.1.2 Cover depth

Reinforcement cover depth constitutes, together with the characteristics of concrete expressed by the chloride diffusion coefficient, the resistant capacity of the concrete against the chloride penetration. The stipulation of minimum

x_C values ($x_{C,min}$) is a common practice in prescriptive durability specifications of many current codes. The Brazilian NBR 6118 [44], for example, indicates $x_{C,min}$ between 35 and 50 mm for structures exposed to marine environments, depending on exposure class and structural element type.

Naturally, the increase in the cover depth makes it difficult to start the chloride-induced corrosion since the path to be followed by the aggressive ions increases. In this sense, the Eurocode 2 [45] suggests that an increase of 10 mm can increase the structural service life from 50 to 100 years. However, it should be noted that excessive increases in the cover depth may not be the most appropriate measure since there is a greater risk of concrete cracking. Within the acceptable limits of cover depth, the best alternative is to ensure that the concrete has low penetrability.

It is also necessary to pay attention to the fact that the cover depths, although measured and considered adequate before the concrete placing, can undergo alterations during this process, resulting, in some instances, in inadequate reinforcement covers in the placed structure. The guarantee that the structure will present adequate cover depth to reach the required service life is, therefore, a function of the specification of the appropriate nominal cover depth for a certain exposure class and of rigorous quality control at the construction time.

1.1.3 Environmental characteristics

Surface chloride content is a critical parameter in concrete durability design, being a quantitative measure of the environmental aggressiveness against the structure [46]. Previous studies show that C_S is strongly affected by various factors, especially exposure duration and environmental conditions, such as wind directions and speed, chloride concentration of seawater, distance from seawater, and rain fallout [47]–[49].

Due to many environmental parameters that affect the surface chloride content and its high variability, it is not easy to make an accurate estimate of C_S values. Additionally, several of these factors have characteristics or influences that vary over time, making C_S also time-dependent ($C_{S(t)}$). Despite this, many service life prediction models adopt constant C_S values – as is the case in the models addressed in this paper. It should be noted that the constant value adopted is usually higher than the effective value of C_S during the first ages of the structure; on the other hand, although C_S can become elevated at advanced ages, it tends to stabilize. Thus, an average value of C_S is usually adopted in service life prediction models and tends not to generate significant distortions in the analyzes performed.

In reporting the Norwegian experience on concrete durability specifications, GjØrv [10] also highlights the high variability of C_S , pointing to the need for C_S values to be appropriately estimated and selected and be as representative as possible, especially regarding the most exposed elements of the structure. In some instances, it is appropriate to consider different surface chloride contents for different parts of the same structure, as highlighted by Saassouh and Lounis [50] and proposed by Beushausen et al. [51] for the upcoming *fib* Model Code 2020.

Therefore, correctly estimating environmental aggressiveness tends not to be an easy task for designing a new structure. Although data obtained from structures exposed to similar environments can be taken as a basis for designing new structures, these also tend to present great variations. For example, Helene [52] guides de adoption of $C_S = 0,9\%$. Nunes et al. [53] evaluated the chloride content in the outer layers of structures located in the city of Rio Grande, southern Brazil, and obtained surface chloride contents equal to 3.1%, 1.1% and 0.6% at distances of 0 m, 160 m, and 630 m from the coastline. In turn, Balestra et al. [54] evaluated the C_S at three different points of the same structure, with values of 3.14% in the tidal zone, 2.32% in the splash zone, and 0.65% in the airborne zone.

1.2 Concrete service life modeling

Estimating the time during which a specific reinforced concrete structure can perform its functions without significant deterioration is of great technical and economic importance. Service life modeling, in turn, is of fundamental importance in ensuring the performance of reinforced concrete structures under chloride environments. However, concrete service life modeling requires some parameters to be known during the structural design, e.g., surface chloride content. In many cases, however, estimating these parameters can pose difficulties.

At the conceptual level, the model presented by Tuutti [55] (Figure 2) is commonly adopted to describe the degradation process of reinforced concrete structures due to reinforcement corrosion. The initiation period comprises the penetration of harmful agents through the cover layer until they reach the rebar. This process is directly influenced by the concrete characteristics, environmental exposure conditions, reinforcement cover depth, and the nature of the ingress agent. The corrosion process is established during the propagation phase, causing the progressive degradation of the concrete and the structure. Due to the significant damage caused to the structure during the propagation period and the fact that it is considerably shorter than the initiation period, many models focus their analysis on the initiation period.

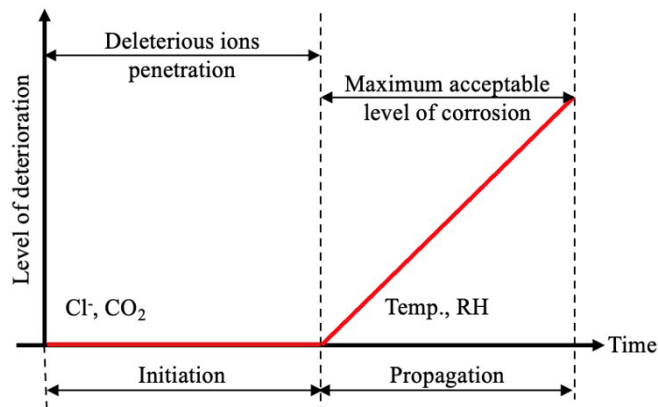


Figure 2. Tuutti [55] two-phase model for concrete service life.

In the case of chloride penetration, the corrosion begins when the chloride content at the reinforcement depth ($C_{(x,t)}$) reaches levels above the chloride threshold (C_{CR}), causing the rebar depassivation. The chloride threshold, however, is a parameter that depends on several factors, such as, among others, characteristics of the constituent materials, water/binder ratio, and concrete saturation degree. Even so, in many cases, the value of 0.4% by mass of cement is adopted as C_{CR} [56]–[59].

Adopting depassivation as the durability limit state aims to prevent reinforcement corrosion. According to Andrade [60], however, depassivation does not comprise the classic definition of the serviceability limit state presented in ISO 16204 [61] and ISO 2394 [62] codes. This is because, at the time of depassivation, there is only the triggering of the corrosive process, without any negative effect on the structural behavior. According to the author [60], an adequate definition is that depassivation indicates a limit state of initiation of deterioration, as presented in ISO 13823 [63]. Additionally, *in situ* identification of the exact moment of depassivation is only possible using electrochemical measurements. For these reasons, the adoption of the moment of the appearance of rusts or spots or the beginning of concrete cracking as DLS is discussed. However, it is necessary to pay attention to the fact that, from a technical perspective, reversing the corrosion process after depassivation can be quite difficult. Thus, although adopting a post-depassivation limit state has an important role in evaluating existing structures, taking depassivation as a DLS is a conservative measure, which may be interesting in the design phase and the definition of the maintenance plan of the structure.

Andrade [64] proposed considering four different levels to estimate the service life of reinforced concrete structural elements: deemed to satisfy, hybrid approach, deterministic performance-based, and probabilistic performance-based. This methodology is like that presented by ISO 13823 [63] and has been discussed in some studies (e.g., [7], [65]–[69]).

The deemed to satisfy, also called the prescriptive approach, is adopted by important current codes ([44], [70], [71]). It is limited to stipulating limit values for parameters such as maximum water/binder ratio, minimum concrete compressive strength, minimum cement content, and minimum cover depth. In a hybrid approach, the evaluation of concrete properties directly linked to its durability is included, especially using accelerated test methods - called durability indicators. However, it should be noted that in a hybrid approach, prescriptive specifications are still heavily demanded, with durability indicators usually adopted as a complementary test method. Additionally, in this approach, service life numerical modeling is not required.

Service life prediction models are used when deterministic or probabilistic performance-based approaches are adopted. Among the deterministic service life models against chloride penetration, the most used expression is the solution of Fick's 2nd Law of Diffusion, which allows estimating the $C_{(x,t)}$ value. When a purely deterministic model is used, only the average values of each variable involved in the process are considered.

In practice, however, the concrete resistance to chloride penetration and the environmental aggressiveness are variable parameters of a random nature. Because of such randomness of the parameters involved in the chloride penetration in concrete, strictly deterministic models tend to present flaws in the representation of the phenomenon and, consequently, in the estimate performed. Thus, the use of probabilistic models constitutes a possibility for a more realistic assessment of the mechanisms, variables, and processes that cause the deterioration of reinforced concrete structures [72], [73].

1.2.1 Probabilistic modeling

Most probabilistic analyses of concrete durability adopt deterministic models and introduce probabilistic parameters of the variables involved in the process. In these cases, simulations are carried out based on a model considered adequate for representing the chloride penetration in concrete (e.g., Fick's 2nd Law of Diffusion), considering the average values and the variability allowed for each parameter involved.

In a general and simplified way, the achievement of a specific limit state can be evaluated based on the limit state function presented in Equation 1.

$$g = R - S \tag{1}$$

In Equation 1, g indicates the limit state function, R refers to the resistant capacity of the structure under the evaluated situation, and S concerns the demand or loading that can lead the structure to reach the limit state in question.

In cases of service life analysis of concrete structures under chloride penetration, taking depassivation as the durability limit state, Equation 1 can be rewritten as a function of $C_{(x,t)}$ and C_{CR} (Equation 2).

$$g_{(x,t)} = C_{CR} - C_{(x,t)} \tag{2}$$

By adopting a probability-based approach, analyzes are performed by evaluating the probability of failure (P_f). Therefore, the aim is to determine the probability of reaching the limit state in question. From the limit state function established in Equation 2, it is possible to calculate P_f based on Equation 3.

$$P_f = P(C_{(x,t)} \geq C_{CR}) \tag{3}$$

Since the performance of a structure against chloride penetration is a function of several random variables, its service life will also be a random variable. Thus, probability-based service life estimates must also be analyzed from the perspective of probability.

The consideration of characteristic service life (SL_k) is analogous to the already familiar concept of concrete characteristic compressive strength (f_{ck}). Such characteristic resistance refers to a value with a predefined probability of not being reached. Thus, the characteristic service life of a structure can be defined as the age from which a probability of failure is greater than the admitted probability of failure ($P_{f,lim}$), as shown in Figure 3.

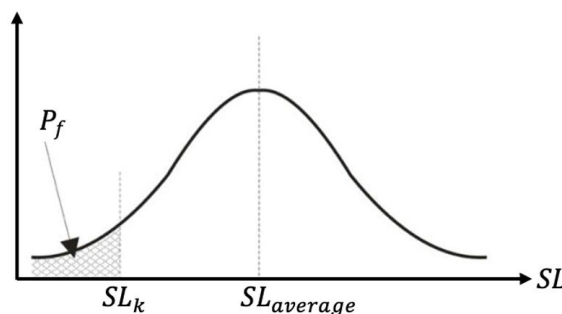


Figure 3. Characteristic service life concept illustration (adapted from [74]).

It should be noted, however, that $P_{f,lim}$ value is another non-consensual aspect. Helland [75] reports that Norway has adopted $P_{f,lim} = 10\%$ for the calibration of standardized prescriptive parameters. Additionally, the author reports $P_{f,lim}$ values of 2%, 30%, and 50% in other European countries. In terms of service life, this $P_{f,lim}$ variations imply estimates between 50 and 109 years if considering the same structure exposed to the same environment.

2 PROBABILISTIC SERVICE LIFE ASSESSMENT

Three chloride penetration models were used to evaluate the service life of concrete structures – namely, the Duracon model [10], the *fib* Model [11], and the *Life-365* model [12]. These models were chosen because they result from well-structured research programs, are presented in normative or pre-normative texts, and have already been applied in the durability design of reinforced concrete structures exposed to marine environments. The three models are based on Fick's 2nd Law of Diffusion.

2.1 Duracon model

The Duracon model [10] was developed from improvements to the model proposed by the European project DuraCrete [76]. Since then, several organizations have adopted this model in normative codes that deal with reinforced concrete structures and probability-based service life design, especially in Nordic countries. The model is presented in Equations 4, 5, and 6.

$$C_{(x,t)} = C_S \left[1 - \operatorname{erf} \left(\frac{x_C}{2\sqrt{D(t)t}} \right) \right] \quad (4)$$

where $C_{(x,t)}$ is the chloride content at depth x_C after time t (%), C_S is the surface chloride content (%), erf is the Gauss error function, and $D(t)$ is the time-dependent chloride diffusion coefficient (Equation 5), adopted by the Duracon model based on the study presented by Tang and Gulikers [77].

$$D(t) = \frac{D_0}{1-\alpha} \left[\left(1 + \frac{t'}{t} \right)^{1-\alpha} - \left(\frac{t'}{t} \right)^{1-\alpha} \right] \left(\frac{t_0}{t} \right)^\alpha k_e \quad (5)$$

where D_0 is the diffusion coefficient at the reference time t_0 (m^2/s), t' is the age of concrete at the time of first chloride exposure (years), α represents the concrete aging factor, and k_e is a parameter that considers temperature's effect (Equation 6).

$$k_e = \exp \left[\frac{E_A}{R} \left(\frac{1}{293} - \frac{1}{273+T} \right) \right] \quad (6)$$

where \exp is the exponential function, E_A is the activation energy for chloride diffusion (kJ/mol), R is the gas constant ($\text{J}/(\text{mol} \times \text{K})$), and T is the temperature ($^\circ\text{C}$).

2.2 *fib* model

The *fib* model for chloride penetration was initially presented in the *fib* Model Code for Service Life Design [11]. Later, it was also included in the text of the *fib* Model Code 2010 [78] and ISO 16204 [61]. Like the Duracon model, the *fib* model also adopts the solution of Fick's 2nd Law of Diffusion and includes the consideration of a time-dependent diffusion coefficient. This model is presented in Equations 7 and 8.

$$C_{(x,t)} = C_S - (C_S - C_0) \left[\operatorname{erf} \left(\frac{x_C}{2\sqrt{D_{app}(t)t}} \right) \right] \quad (7)$$

where $C_{(x,t)}$ is the chloride content at depth x_C after time t (%), C_S is the surface chloride content (%), C_0 is the initial chloride content of concrete, erf is the Gauss error function, and $D_{app}(t)$ is the time-dependent chloride diffusion coefficient (Equation 8).

$$D_{app}(t) = D_{app}(t_0) \left(\frac{t_0}{t} \right)^\alpha \quad (8)$$

where $D_{app}(t)$ is the time-dependent chloride diffusion coefficient, $D_{app}(t_0)$ is the apparent diffusion coefficient measured at a reference time t_0 , and α is the concrete aging factor.

2.3 Life-365 model

The *Life-365* software [12] enables the analysis of corrosion initiation and propagation periods, the determination of the structure's maintenance plan, and the estimation of the structure's life cycle costs. In this paper, however, only the initiation period is considered. Like *Duracon* and *fib* models, *Life-365* estimates the initiation period based on Fick's 2nd Law of Diffusion. However, the diffusion coefficient is considered time- and temperature-dependent, as shown in Equation 9.

$$D(t, T) = D_{ref} \left(\frac{t_{ref}}{t} \right)^\alpha \exp \left[\frac{U}{R} \left(\frac{1}{T_{ref}} - \frac{1}{T} \right) \right] \quad (9)$$

where $D(t, T)$ is the diffusion coefficient at time t and temperature T , D_{ref} is the diffusion coefficient at a referent time (in *Life-365* = 28 days), α is the concrete aging factor, exp is the exponential function, U is the activation energy for chloride diffusion (kJ/mol), R is the gas constant (J/(mol \times K)), and T_{ref} is the reference temperature (K).

It should be noted that the *Life-365* model, unlike the *Duracon* and *fib* models, considers that the reduction in the diffusion coefficient of concrete over time, expressed by the aging factor, occurs until the age of 25 years. After this age, the diffusion coefficient becomes only temperature dependent. Furthermore, in *Life-365*, the values of α can be determined experimentally or calculated considering the type and content of mineral admixture used in the concrete.

3 NUMERICAL ANALYSIS

The depassivation probabilities were calculated using the expression previously presented in Equation 3, using the $C_{(x,t)}$ values estimated based on the three models analyzed. The Monte Carlo Simulation was used to determine the P_f , being performed 10^6 simulations in each analysis.

The influence of surface chloride content was evaluated considering mean C_S values (μ_{C_S}) = 2.0 and 3.5%. While $C_S = 2.0\%$ refers to one of the values obtained by Guimarães [79]; 3.5% is recommended by GjØrv [10] for marine environments with an average environmental load. The influences of C_S variabilities were also evaluated. For this, were considered coefficients of variation (CV) of C_S (CV_{C_S}) equals to 0.10, 0.20, and 0.30. In all cases, C_S was admitted following lognormal probability distribution.

The chloride diffusion coefficient was assumed with a normal probability distribution, being adopted in the analysis mean values equal to 3.0×10^{-12} and 5.0×10^{-12} m^2/s , and coefficients of variation (CV_D) of 0.10, 0.20, and 0.30. Regarding the cover depth, a normal probability distribution was assumed, with mean value = 50 mm and coefficients of variation (CV_{x_c}) = 0.05, 0.10, and 0.20.

In all the analyzes carried out, service life was considered equal to 100 years. When required by the models used, the aging factor was taken with a normal probability distribution, with mean = 0.4 and standard deviation (σ) = 0.04 (N(0.4; 0.04)). As for temperature, $\mu = 18$ °C, $\sigma = 3.6$ °C, and normal probability distribution (N(18; 3.6)). C_{CR} was assumed with a normal probability distribution, $\mu = 0.4\%$, and $\sigma = 0.04\%$ (N(0.4; 0.04)).

The results about the influence of surface chloride concentration on $C_{(x,t)}$ are shown in Figure 4 and Figure 5. A greater environmental aggressiveness, represented by a greater C_S value, increases $C_{(x,t)}$. However, what is most evident in all models analyzed is the strong influence of CV_{C_S} in the prediction of chloride concentration. This variation is a fact that generates many uncertainties in the use of service life prediction models since, as discussed in Section 1.1.3, the definition of C_S is extremely complex and presents great variability because it depends on a series of environmental variables not controllable.

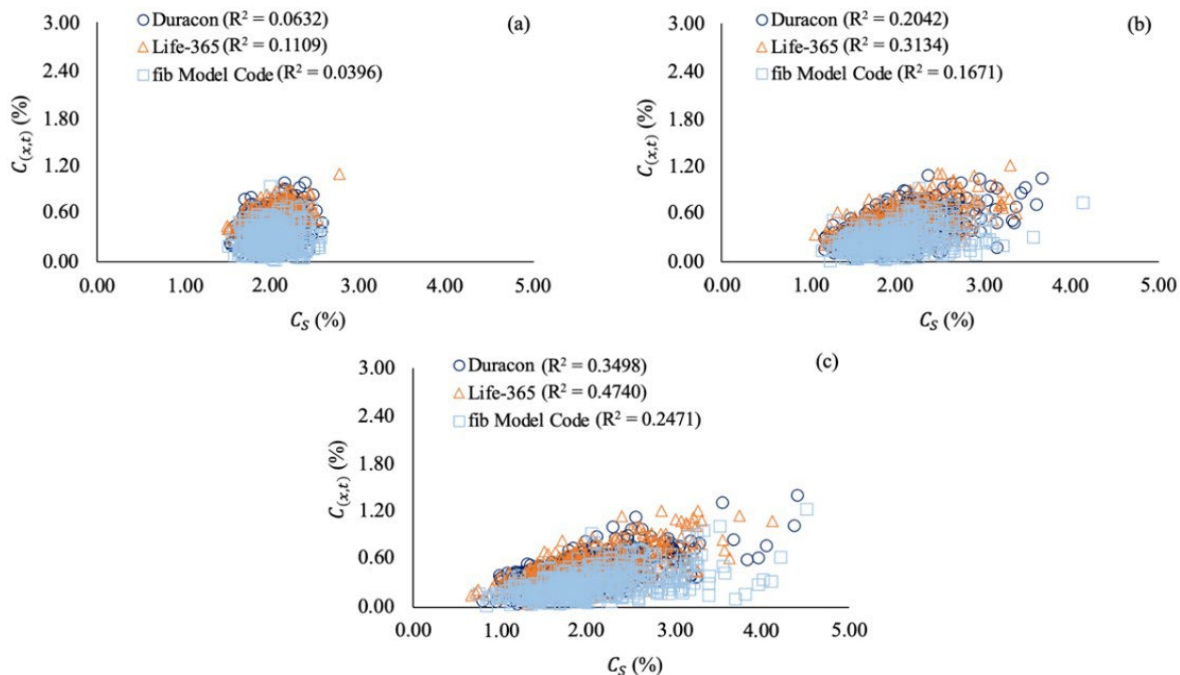


Figure 4. Influences of C_S on chloride penetration. $\mu_{C_S} = 2.00$; CV_{C_S} : (a) = 0.10, (b) = 0.20, and (c) = 0.30. Note: $D = N(3.00; 0.30)$, $x_C = N(50.00; 5.00)$, $\alpha = N(0.40; 0.04)$, $T = N(18.00; 3.60)$, and $t = 100$ years.

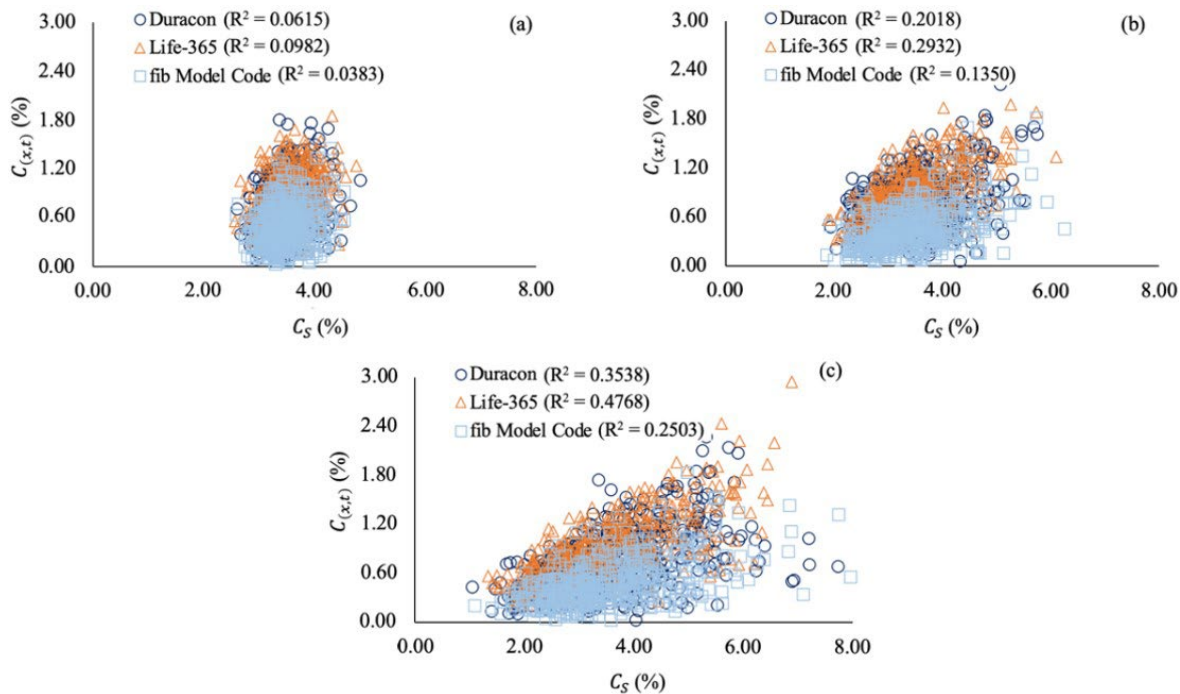


Figure 5. Influences of C_S on chloride penetration. $\mu_{C_S} = 3.50$; CV_{C_S} : (a) = 0.10, (b) = 0.20, and (c) = 0.30. Note: $D = N(3.00; 0.30)$, $x_C = N(50.00; 5.00)$, $\alpha = N(0.40; 0.04)$, $T = N(18.00; 3.60)$, and $t = 100$ years.

Figure 6 and Figure 7 present the influences of the chloride diffusion coefficient in $C_{(x,t)}$. It is important to remember that D is the main indicator of concrete resistance to chloride penetration; therefore, lower D values provide less chloride penetration and tend to give a longer service life to the structure. The variability of D , in turn, is mainly affected by the characteristics of the test method used and by the concrete production and placing processes. However, variability

related to the test method can be easily quantified and considered. As can be seen, increasing CV_D leads to significant increases in the scatter of the estimated $C_{(x,t)}$ values.

Thus, the importance of a technical framework for evaluating the concrete characteristics and its durability potential based on durability indicators test methods is reaffirmed. Additionally, it is essential to establish control methodologies of the concrete at the construction site to quantify the mean values of D and its variability, allowing the *in-situ* conformity control of durability specifications.

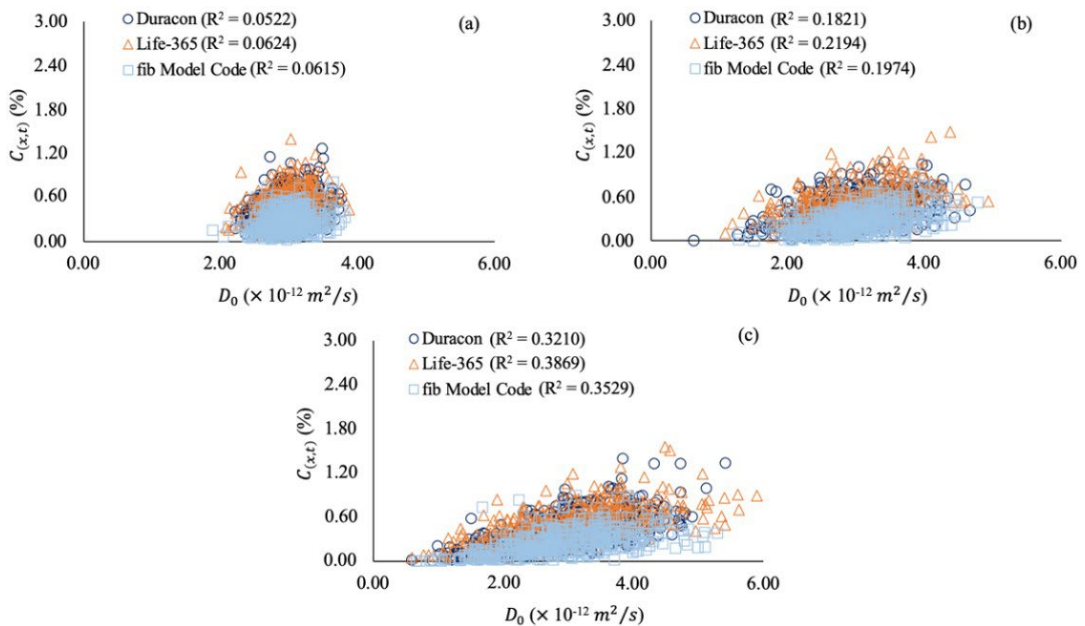


Figure 6. Effects of D_{28d} on chloride penetration. $\mu_D = 3.00$; CV_{CS} : (a) = 0.10, (b) = 0.20, and (c) = 0.30. Note: $C_S = LN(2.00; 0.40)$, $x_C = N(50.00; 5.00)$, $\alpha = N(0.40; 0.04)$, $T = N(18.00; 3.60)$, and $t = 100$ years.

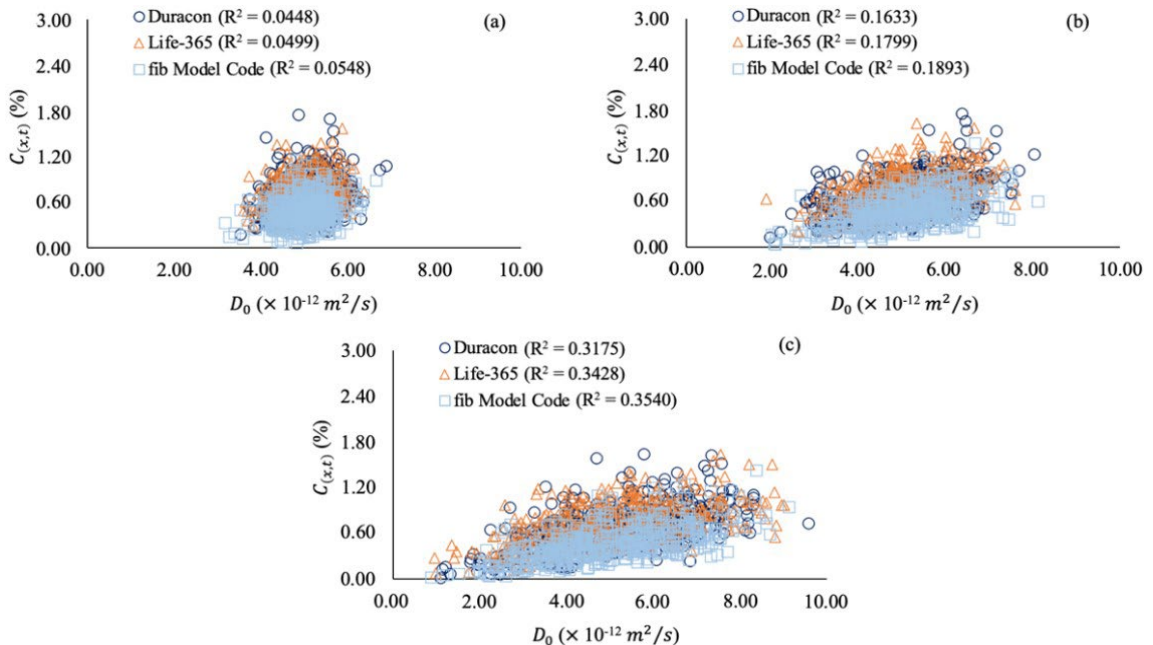


Figure 7. Effects of D_{28d} on chloride penetration. $\mu_D = 5.00$; CV_{CS} : (a) = 0.10, (b) = 0.20, and (c) = 0.30. Note: $C_S = LN(2.00; 0.40)$, $x_C = N(50.00; 5.00)$, $\alpha = N(0.40; 0.04)$, $T = N(18.00; 3.60)$, and $t = 100$ years.

The influence of cover depth variabilities in $C_{(x,t)}$ is shown in Figure 8. Among the parameters analyzed in this paper, x_c is the one that is easier to stipulate since it is recommended in prescriptive durability specifications and can also be calculated through numerical modeling. The variabilities of x_c , however, have a strong influence in $C_{(x,t)}$, significantly increasing the scatter of the results obtained, as also observed by Magalhães [74]. It should be noted that CV_{x_c} is directly related to the structure construction process, which confirms the importance of strict quality control in producing concrete structural elements, especially under aggressive environments.

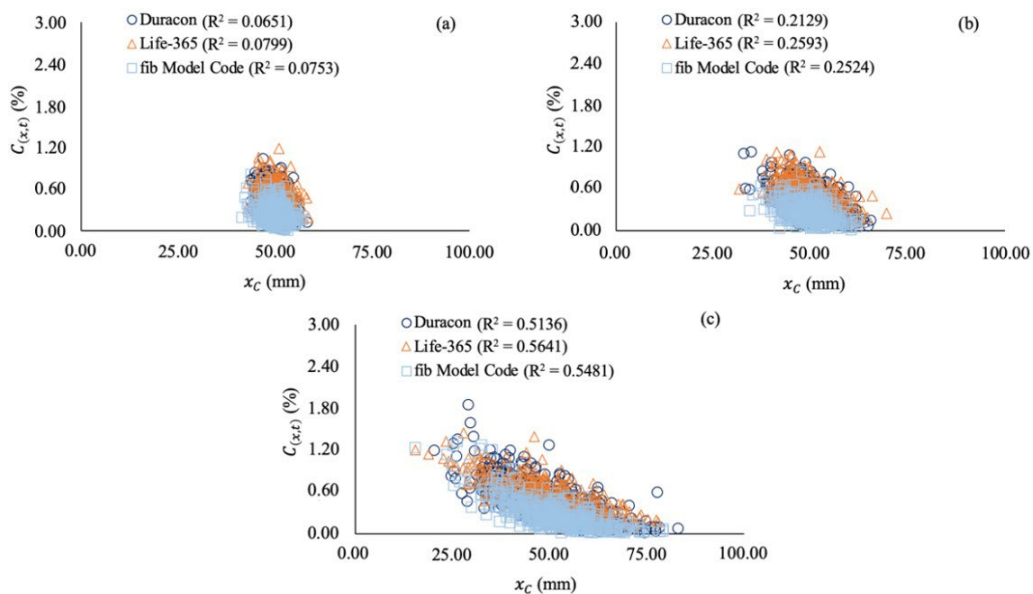


Figure 8. Influences of cover depth variability on chloride penetration. $\mu_{x_c} = 50.00$; CV_{x_c} : (a) = 0.05, (b) = 0.10, and (c) = 0.20. Note: $C_S = LN(2.00; 0.40)$, $D = N(3.00; 0.30)$, $\alpha = N(0.40; 0.04)$, $T = N(18.00; 3.60)$, and $t = 100$ years.

Lastly, the influence of C_{CR} on P_f of reinforced concrete structural elements under chloride penetration was evaluated. Since C_{CR} is widely discussed in the literature, and there is no consensus on its average value, a range of C_{CR} between 0.0 e 1.2% (Figure 9) and coefficients of variation between 0.0 e 0.5 (Figure 10) were considered. It is known that $C_{CR} = 1.2\%$ is a high value and naturally leads to a very low corrosion probability. This value was adopted to visualize the influences of the chloride threshold on the service life prediction of concrete structures. It should also be noted that, although the value of 0.4% is the most adopted for C_{CR} , the North American code ACI 318 [71] suggests a chloride threshold equal to 1.00% for concrete exposed to dry environments; on the other hand, standards such as Brazilian NBR 12655 [80] indicate values between 0.15 and 0.40%, depending on the concrete exposure conditions.

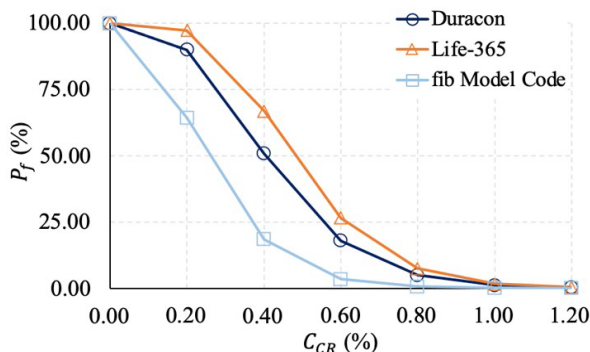


Figure 9. Influence of C_{CR} in P_f ($CV_{C_{CR}} = 0.1$). Note: $C_S = LN(2.00; 0.40)$, $D = N(3.00; 0.30)$, $x_c = N(50.00; 5.00)$, $\alpha = N(0.40; 0.04)$, $T = N(18.00; 3.60)$, and $t = 100$ years.

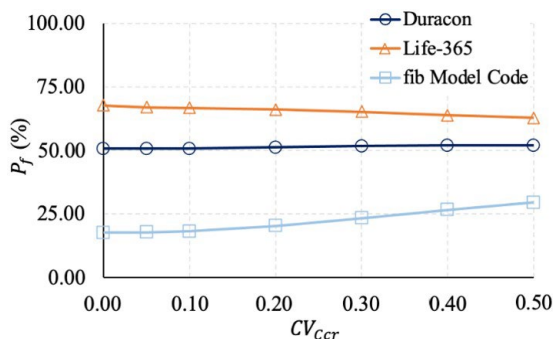


Figure 10. Effects of CV_{Ccr} in the probability of chloride-induced corrosion ($\mu_{Ccr} = 0.4\%$). Note: $C_S = LN(2.00; 0.40)$, $D = N(3.00; 0.30)$, $x_C = N(50.00; 5.00)$, $\alpha = N(0.40; 0.04)$, $T = N(18.00; 3.60)$, and $t = 100$ years.

It is observed that the P_f presented in Figures 9 and 10 are quite high. However, it should be noted that this occurs due to the set of the evaluated scenario. A diffusion coefficient lower than the one considered ($= 3.0 \times 10^{-12} \text{ m}^2/\text{s}$) would lower corrosion probabilities.

Lastly, the evolution of P_f over time was evaluated. The results are shown in Figure 11. At all ages, the probability of failure calculated based on the *fib* Model [11] was significantly lower than that obtained from the other models. Another fact to note is that if $P_{f,lim} = 10\%$ is taken, the characteristic service life obtained through the Duracon model is less than 50 years, while the *Life-365* model leads to SL_k of approximately 52 years, and the *fib* Model indicates SL_k next to 80 years.

Although probabilistic approaches are an important tool in the service life design of reinforced concrete structures, it is necessary to remember that different models can lead to very different scenarios. Thus, it is essential that studies on concrete durability also seek to understand the relationship between the estimates made considering accelerated test methods and chloride penetration prediction models and the actual behavior of concrete structures in marine environments. It should be noted that many intervening factors and uncertainties are associated with the degradation process of reinforced concrete structures due to chloride penetration. So, the results obtained through numerical modeling should not be taken as absolute numbers of the concrete service life but as a basis for analyzing its behavior over time and making decisions related to maintenance and other interventions.

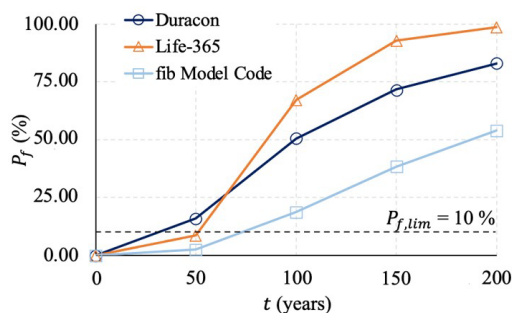


Figure 11. Evolution of P_f over time. Note: $C_S = LN(2.00; 0.40)$, $D = N(3.00; 0.30)$, $x_C = N(50.00; 5.00)$, $\alpha = N(0.40; 0.04)$, $T = N(18.00; 3.60)$, $C_{CR} = N(0.40; 0.04)$, and $t = 100$ years.

4 CLOSURE

This paper discussed the main factors affecting the service life of reinforced concrete structures in marine environments. Emphasis was given to the surface chloride content, which indicates the aggressiveness of the environment, and the cover depth and chloride diffusion coefficient, which refer to the resistance of the structural element to the penetration of ions. Important concepts related to probability-based durability design were reviewed and discussed.

The importance of adequate control of specification and construction of the concrete structure was observed concerning the cover depth and the chloride diffusion coefficient. The variability of these parameters, directly related

to the conditions of the construction process, significantly affects the estimated $C_{(x,t)}$. In many of the cases analyzed, there was a low correlation between the analyzed variables and $C_{(x,t)}$, corroborating the data presented by Yu et al. [73].

The significance of considering adequate C_{CR} values was discussed. Although the variability of C_{CR} and the effect of several parameters on the chloride threshold is widely discussed in the literature; in many cases C_{CR} is considered a value applicable to structures regardless of their properties, as discussed by Käthler et al. [81]. Due to their significant influence on service life estimates, test methods for determining C_{CR} of structural elements are important tools for designing and evaluating reinforced concrete structures.

Regarding the service life prediction models adopted in this paper, it should be noted that considering $P_{f,lim} = 10\%$, the SL_k range obtained was greater than 30 years. This fact shows the strong influence that the model used has on the estimates made. While certain models can lead to overestimated service life analyses, generating the need for unforeseen interventions, others can present underestimated results, increasing the cost of the project in order to achieve the desired service life. Thus, although probability-based approaches represent the analyzed phenomenon better, service life prediction models must be used judiciously. As possible, calibration processes with long-term exposure data should be carried out.

Concrete durability approaches move towards performance specifications. Thus, the introduction of durability indicators, service life prediction models, and the concept of characteristic service life are fundamental in evaluating reinforced concrete structures' durability under aggressive environments. Therefore, it is necessary that experimental programs linked to the numerical application be established. In this way, clear procedures can be set to determine the characteristic service life and admitted probabilities of failure for each environment and structure through new normative references. These procedures would contribute to decision-making in the design phase and the design of structural elements capable of fulfilling a minimum service life, guaranteeing the desired performance, and avoiding premature costs with conservation activities.

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