

Modelling of Deterioration Effects on Concrete Structures

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In order to predict the service life of concrete structures models for deterioration effects are needed. This paper has the form of a survey, listing and describing such analytical models, namely carbonation of concrete, ingress of chlorides, corrosion of reinforcing steel and prestressing tendons. The probabilistic approach is applied.

Keywords: concrete structures, service life, reinforcement, tendons, carbonation, corrosion, chloride ingress.

1 Introduction

There is increasing economic pressure for both newly-built and existing facilities to be kept in-service for a longer period of time. This requirement is enhanced by environmental awareness. To cope with such tasks, mathematical models can be made for the performances of a building or structure, including the process of ageing (deteriorating). Generally two classes of problems may be distinguished in this context:

- (i) Prediction of service-life for structures during the design process. This becomes a necessity in the context of the new "Performance-based design" rules, which are a trend in forthcoming engineering work – see e.g. [1];
- (ii) Assessment of existing structures, i.e., estimating residual service life.

To estimate future safety and performance one has to consider not only the real material, loading and technological characteristics but also possible deterioration (e.g., by corrosion), physical damage (environmental loadings, impacts), maintenance and possible repair works. In principal, these effects and their governing inputs are highly uncertain, so the adoption of a probabilistic approach is unavoidable.

Concrete structures – with mild steel or prestressed reinforcement – undergo a process of ageing due to environmental and loading agents (chemical and mechanical/physical processes) during their service. Experimental investigation is a basic procedure; however, in many situations – especially concerning more complex structural members or structures – it is not feasible. The solution is to model such situations analytically. The aim of this paper is to describe some existing models of various kinds of material deterioration and to discuss their probabilistic form.

Service life

First let us briefly define the service life (or structural life-time) of a structure. According to [2] service life in general is ... *the period of time after installation during which all the conditions of the structure (structural part) meet or exceed the performance requirements* It should be stressed that the basic requirement is structural safety. Much research has been devoted to methods and applications for service life – see, e.g., [3], [4].

Frequently the limit state concept is utilized, comprising a function for the influence of the load S and a function for the load bearing capacity R (or the structural resistance function). Then the following inequality should be satisfied:

$$Z = R - S \geq R(X_1, X_2, \dots, X_n) - S(Y_1, Y_2, \dots, Y_m) \geq 0 \quad (1)$$

where:

Z – safety margin (or limit state function)

X_i – basic variable ($i = 1, \dots, n$) of the loading function

Y_j – basic variable of the resistance function ($j = 1, \dots, m$).

Note that the same variable may be included in both the sets X and Y (e.g., Young's modulus in the case of a redundant structure). For the limit state of an ageing structure, time t is also included among the basic variables.

Due to the uncertainties of the parameters involved a non-deterministic analysis – i.e., a probabilistic analysis – should be adopted. Then *first*, the *initiation* period, during which the passivation of the reinforcing bars (due to

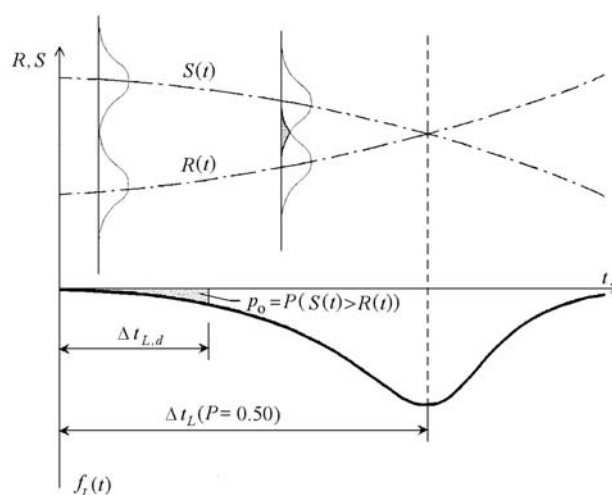


Fig. 1: Probabilistic representation of structural service life

effects) is in progress. Second, the *propagation* period, during which corrosion of the reinforcement takes place, and consequently, the failure probability is increased for both the load and serviceability limit states. This is illustrated in Fig. 2. The structural life prediction can be reached by modelling such effects.

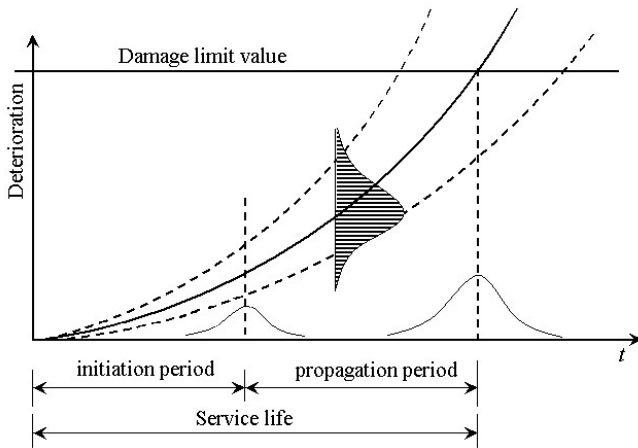


Fig. 2: Two periods of structural service life

As mentioned above the parameters in the model are in principle stochastic in nature; the actual value of the parameter is subjected to scatter. This implies that the probability can be described by, e.g., probability density functions for all parameters. Therefore it is necessary to model the uncertainties and add them to the mathematical performance model or the probabilistic service life design.

2 Analytical models

Carbonation of concrete

Carbonation is a chemical process in concrete driven by ambient CO_2 penetrating from the surface and decreasing pH up to a value approximately equal to 9. When the carbonation depth x_c equals the cover c the steel is depassivated and corrosion may start (see also the initiation time explained above).

• Papadakis et al

The model of time-dependent carbonation depth x_c of OPC concrete developed by Papadakis et al [6] is an analytical model based on the mass conservation of CO_2 , $\text{Ca}(\text{OH})_2$ and CSH (hydrated calcium silicate) in any control volume of the concrete mass. The simplified carbonation depth formula for OPC concrete is expressed here as:

$$x_c = 0.35 \rho_c \frac{\left[\frac{w}{c} \approx 0.3 \right]}{\left[1 + \frac{\rho_c w}{1000 c} \right]} \sqrt{f(RH)} \sqrt{1 + \frac{\rho_c w}{1000 c} + \frac{\rho_c a}{\rho_a c} c_{\text{CO}_2} \frac{22.4}{44} 10^6 t}, \quad (2)$$

where x_c is the carbonation depth (mm) for time t (years), ρ_c , ρ_a are the mass density (kg/m^3) of cement and aggregates, respectively, w/c , a/c are the water/cement and aggregate/cement ratio, respectively, RH is the ambient relative humidity, c_{CO_2} is the CO_2 content in the atmosphere (mg/m^3). Aggregate unit content a is calculated using the equation:

$$a = s + g_{4-8} + g_{8-16} \quad (3)$$

where s , g_{4-8} , g_{8-16} are the unit content of sand, gravel size 4–8 mm and gravel size 8–16 mm, respectively. Then, the mass density of the aggregates can be expressed as:

$$\rho_a = \frac{a}{\frac{s}{\rho_s} + \frac{g_{4-8}}{\rho_{g_{4-8}}} + \frac{g_{8-16}}{\rho_{g_{8-16}}}}, \quad (4)$$

where ρ_s , $\rho_{g_{4-8}}$, $\rho_{g_{8-16}}$ are the mass densities of sand, gravel size 4–8 mm and gravel size 8–16 mm, respectively.

In the original formulation the model does not provide satisfactory results for high values of RH . This has been overcome by implementing the step-wise linear relationship $f(RH)$ extracted from the experiments reported by Matoušek [7], see also [8].

A total of 11 variables are involved in this model, and in some cases not all appropriate pieces of information are available. Therefore, a simpler model may be useful:

• Bob and Affana [9]

$$x_c = 150 \frac{C k d}{f_c} \sqrt{t}, \quad (5)$$

where x_c is the average depth of carbonation (mm), f_c is the concrete compressive strength (MPa), t is the time of CO_2 (years), C is the coefficient of cement type, k introduces the influence of humidity (environmental conditions), and d is the coefficient for CO_2 content.

Both the models were probabilised and compared – see [10]. There are some other carbonation models – see a review [11].

Chloride ingress

Chloride ions diffuse through the protective concrete cover. After reaching the critical threshold of chloride concentration, the protective passive film around the reinforcement is dissolved and the corrosion of the reinforcement may be initiated.

The model of chloride diffusion-adsorption in concrete by Papadakis et al. [12] can be used. This model allows us to predict the chloride concentration in the solid and liquid (aq) phases of concrete as a function of the initial concentration $[\text{Cl}^- (\text{aq})]_i$, of the concentration $[\text{Cl}^- (\text{aq})]_0$ (mol/m^3) on the nearest concrete surface and the distance x (mm) from it, and that of time t (years). The assumption of the formation of a moving “chloride front”, where the concentration of $\text{Cl}^- (\text{aq})$ decreases to zero, gives the following expression for the distance x_{Cl} (mm) from the surface:

$$x_{\text{Cl}} = 1000 \sqrt{\frac{2D_{e,\text{Cl}} [\text{Cl}^- (\text{aq})]_0}{3.1536 [\text{Cl}^- (\text{solid})]_{\text{sat}}}} t \cdot 10^{17} \quad (6)$$

$[\text{Cl}^- (\text{solid})]_{\text{sat}}$ denotes the saturation concentration of Cl^- in the solid (mol/m^3) and $D_{e,\text{Cl}}$ is the effective diffusivity (m^2/s):

$$D_{e,\text{Cl}} = 0.15 \frac{1 + \rho_c \frac{w}{c} \left[\rho_c \frac{w}{c} \approx 0.85 \right]^3}{1 + \rho_c \frac{w}{c} + \frac{\rho_c a}{\rho_a c} \left[1 + \rho_c \frac{w}{c} \right]} D_{\text{Cl}^-, \text{H}_2\text{O}} \quad (7)$$

$D_{\text{Cl}^-, \text{H}_2\text{O}}$ denotes the diffusion coefficient of Cl^- in an “infinite solution”.

If $[Cl^-]_{cr}$ indicates the diffusion coefficient of Cl^- in the aqueous phase (mol/m^3) required for the depassivation of steel bars (a critical value), and c_c is the concrete cover (mm), then time t_{cr} (years) to depassivation is given by:

$$t_{cr} = \frac{[Cl^-]_{solid} - [Cl^-]_{sat}}{3.1536 \cdot 10^7 \cdot 2D_{e,Cl} \cdot [Cl^-]_{aq} - [Cl^-]_{aq} - [Cl^-]_{cr}} \quad (8)$$

Again, other models do exist – let us mention (without making a detailed description), e.g., the model based on Fick's second law and utilized in [13].

Simultaneous effect of Cl^- and CO_2

Simultaneous contamination of carbon dioxide and ingress of chloride ions is rather common in reality, but unfortunately there are few works devoted to this problem. In this context the research work [14] has to be mentioned. This experimental study investigates the effect of fly ash addition on the corrosion process in reinforced concrete exposed to carbon dioxide and chloride. It has been shown that in a strongly aggressive chloride environment (a sea coast or locations with frequent use of de-icing salts) the addition of fly ash is beneficial. Conversely, in locations with a high concentration of CO_2 fly ash may accelerate the corrosion process. Reference mortar samples without fly ash were also studied, and it has been proved that the carbonation process affects the chloride profile.

To the author's best knowledge, no analytical model exists for predicting the possible synergetic effect of carbon dioxide and ingress of chlorides.

Steel corrosion – mild steel

Reinforcement corrosion is in progress during the propagation period and its rate is governed by the availability of water and oxygen on the steel surface. Due to corrosion, the effective area of steel decreases and rust products grow, causing at a certain stage longitudinal cracking and later spalling of concrete, or delamination.

• Andrade et al

Note that both the uniform (or general) type and the pitting (localized) type of corrosion should be modelled. Recently, the model used by, e.g., [15], [16] seems to be sufficient for the prediction of uniform corrosion. The formula for the time related rebar diameter decrease reads

$$d(t) = \begin{cases} d_i & t \leq T_i \\ d_i - 0.0116 i_{corr} \alpha (t - T_i) & t > T_i \end{cases} \quad (9)$$

where d_i is the initial bar diameter (mm), T_i is the time to initiation (years) and i_{corr} is the current density (normally expressed in $\mu A/cm^2$). Parameter α expresses the type of corrosion. For homogeneous corrosion α equals 2, however, when localised corrosion (pitting) occurs, α may reach values from 4 up to 8 (see [16]). In this latter case the representation of the corrosion is not accurate enough.

For localised corrosion, the following models can also be taken into consideration. The studies by Gonzalez et al [17] show that the maximum rate of corrosion penetration in the

case of pitting corrosion is 4–8 times that of general corrosion. The depth of the pit at time t can be estimated by the equation

$$p(t) = 0.0116 i_{corr} R_{corr} t \quad (10)$$

with i_{corr} taken as $3 \mu A/cm^2$, a value indicative of a high rate of corrosion, and $R_{corr} = 6$. The corrosion is assumed to be confined to discrete pits.

• Val and Melchers

Advanced modelling of localised corrosion was presented by Val and Melchers [18]. The residual section at the pits can be predicted by simplification in a hemispherical form. The net cross sectional area of a corroded rebar, A_r , at time t , is calculated as

$$A_r(t) = \begin{cases} \pi \frac{d_i^2}{4} - A_1 - A_2 & p(t) \leq \frac{\sqrt{2}}{2} d_i \\ \pi \frac{d_i^2}{4} - A_1 - A_2 - \frac{\sqrt{2}}{2} d_i p(t) & \frac{\sqrt{2}}{2} d_i < p(t) \leq d_i \\ 0 & p(t) > d_i \end{cases} \quad (11)$$

where

$$A_1 = \frac{1}{2} \theta_1 \frac{d_i^2}{2} - a_p \left| \frac{d_i}{2} - \frac{p(t)^2}{d_i} \right|, \quad (12)$$

$$A_2 = \frac{1}{2} \theta_2 p(t)^2 - a_p \frac{p(t)^2}{d_i}$$

$$a_p = 2p(t) \sqrt{1 - \frac{p(t)^2}{d_i^2}}, \quad (13)$$

$$\theta_1 = 2 \arcsin \left| \frac{a_p}{d_i} \right|, \quad \theta_2 = 2 \arcsin \left| \frac{a_p}{2p(t)} \right|$$

and p is the radius of the pit.

It should be noted that pitting corrosion is highly localized on individual reinforcement bars. It is unlikely that many bars could be affected; hence pitting corrosion will not significantly influence the structural capacity of a cross-section.

Stress corrosion cracking (SCC)

The tension of a prestressing wire may induce the presence of cracks and a special type of corrosion – Stress Corrosion Cracking (SCC). The brittle fracture and fatigue limit state should be considered in this case. Fracture mechanics is used to analyze such effects.

According to [19] the SCC effect can be modelled in the following way: Let σ be the effective tension in a wire, and a be the crack depth caused by corrosion. The tension at the depth of the crack is calculated as

$$\sigma_a = \frac{P}{S} K(a) \quad (14)$$

P is the force in the wire (in N), S is the cross sectional area of the wire and $K(a)$ is the stress intensity factor. Its value can be calculated with the aid of the equation of linear fracture mechanics

$$\frac{K}{s\sqrt{\pi a}} = \prod_{i=0}^4 C_{ij} \left(\frac{a}{D} \right)^{C_{ij}} \left(\frac{a}{b} \right)^{C_{ij}} \quad (15)$$

where b is the width of the crack (in m), D is diameter of wire and C_{ij} are given in [19] – table 4. When the value of $K(a)$ reaches the critical value K_{ISCC} (fracture toughness) the wire breaks suddenly.

Fig. 3 represents the dependability of time, crack depth and fracture toughness (a_0 , a_c are the initial, resp. critical crack depths).

very uncertain. Their variability may be significant and stochastic modelling is desirable. Therefore, the input variables should be considered to be random variables described by theoretical probability density functions.

Then the predicted carbonation depth $x_c(\mathbf{X})$ or the time to depassivation $t_{(a_0, a_c)}$ can be calculated.

Note: In the cases of precast prestressed members or well-grouted tendons, reanchoring should be considered at a certain distance from the point of failure. This distance may be evaluated according to *Building code requirements for reinforced concrete* (ACI 318-89), R 12.8. The referred minimum value is 6 inches.

Pitting and SCC corrosion types develop lesser expansion activity than the general case, and therefore the chance of visual detection (cracks on the concrete surface) is often negligible.

3 Assessment of structures – random nature

- *Statistical analysis*

The basic parameters of the carbonation (or other degradation effects) computational models described above are

Bayesian statistical prediction combined with LHS is described in [21]; application to improved (a posterior) prediction of the carbonation time profile is shown in [8]. The objective was to use the measurement to improve or update the long-term values of carbonation depth.

Note that if the data obtained by measurement is too remote from the prior results, then Bayesian updating fails [21].

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