Modelling of Chloride Concentration Effect on Reinforcement Corrosion

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Abstract: The corrosion of reinforcement is one of the major causes of deterioration of reinforced concrete (RC) structures, considerably affecting their durability and reliability. The rate of reinforcement corrosion is governed by, among other factors, the presence of chlorides on the surface of the steel. The assessment of such deteriorating effects necessitates the development of relevant models and utilization of advanced simulation techniques to enable the probabilistic analysis of concrete structures. In this paper an approach for the assessment of RC structures’ durability and reliability under the attack of chlorides is introduced. The concentration of chlorides in time is modelled by a cellular automata technique. This simulation is utilized for the assessment of steel corrosion based on experimental results and on modification of models adopted from the literature. The
described approach is applied to an illustrative example showing the effect of chloride concentration at different locations in the structure on the steel corrosion rate.

1 INTRODUCTION

Damage due to reinforcement corrosion is recognized as one of the major causes of deterioration of reinforced concrete structures with service life reduction as a consequence. Reinforcement concrete durability is then determined by its resistance against various chemical and physical processes. An extensive amount of research on this topic has been organized and reported over the last decades. It has been observed that many of the problems associated with corrosion can be attributed to the presence of salts (see e.g. Bogard et al., 1990).

Among the key factors influencing reinforcement corrosion in concrete are the presence of moisture and the ingress of oxygen from the air. Concrete exposed to the outside climate usually contains enough moisture required for the corrosion process, and the concrete cover enables the penetration of a sufficient amount of oxygen (Nürnberger, 1984; Schueremans et al., 2007). Less permeable and thicker concrete cover prolongs the time to corrosion initiation, and hence the service life of reinforced concrete structures. Thus, concrete quality (e.g. pore structure and the presence of cracks) plays a decisive role.

The other important factors influencing the rate of corrosion of steel bars in concrete also include relative humidity (RH) (RILEM 1996), rainfall (So & Millard, 2007) and temperature (So & Millard, 2007; Jensen et al., 1999; Nguyen et al., 2006). The corrosion rate is likewise significantly influenced by the presence of chloride ions (Karimi, 2000). When chlorides are present in RC structures, the rate of their penetration into concrete and their
concentration may play a significant role for the corrosion process. This should be reflected when designing or assessing a concrete structure with consideration of the durability issue. The effect of chloride concentration on corrosion rate has also been the focus of our experiments, which are partially presented in this paper.

Once the corrosion of reinforcement starts several degradation events may be encountered:
(i) the volume expansion of rust products develops tensile stresses in the surrounding concrete leading to concrete cracking;
(ii) when the corrosion progresses it may consequently be responsible for the spalling of concrete cover;
(iii) a decrease in the effective reinforcement cross-section area may lead to excessive deformation and finally to the exhaustion of the bearing capacity of the cross section or structural member (this type of degradation is the focus of this paper);
(iv) the reduction of bond between reinforcing bars and concrete in load bearing members may lead to reduced stiffness and strength, thus changing the structural behaviour to more brittle failure modes.

Application of design for durability may bring pronounced economical and sustainability impacts. Unfortunately, the prescriptive approach of current standards (e.g. Eurocodes EN 1990 and EN 1992) does not directly allow a design method focused on a specific (target) service life and/or a specific level of reliability. This would require the consideration of inherent uncertainties in material, technological and environmental characteristics to be dealt with while assessing the service life of a structure. To respect this, a probabilistic approach should be utilized as applied in the present paper. The service life of a building or structure is determined by its design, construction, ageing and maintenance during use. The combined effect of both structural performance and ageing should be considered
wherever relevant. The **limit state (LS) approach** is applicable and in general governed by the probability condition:

\[ P_f = P[A(t) \geq B(t)] < P_d \]  \hspace{1cm} (1)

where \( P_d \) is design (required) probability, \( t \) is time, \( A \) is action effect and \( B \) is barrier; both \( A \) and \( B \) (and hence the probability of failure \( P_f \)) are generally time dependent. This has not been considered in the cases of serviceability limit states (SLS) or ultimate limit states (ULS) in design practice very frequently up to now. The time \( t_s \) corresponding to the limit given by eq. (1), i.e. the service life, and the deteriorating effect \( A \) are assessed by utilization of the appropriate degradation models and LS applying a probabilistic approach. Note that instead of the probability of failure \( P_f \), the index of reliability \( \beta \) is alternatively (and rather frequently) utilized in practice – see e.g. (ISO 2394 and EN 1990).

In the case of design for durability a new category of LS has recently been discussed called initiation or **durability limit states (DLS)**. This kind of limit state precedes the occurrence of both “traditional” SLS and ULS and represents a simplified LS intended to prevent the onset of deterioration. It is based on the initiation of deterioration – see the future documents (fib Model Code and ISO 13823). Formally, DLS fall into the group of SLS. For durability design these initiation LS have to be combined often with other LS considering also the propagation period. The level of reliability in the context of durability should be left to the client’s decision together with the definition of a target service life, creating in this way a necessary background for his/her critical decision (e.g. economical optimization).

When considering the LS caused by the degradation of reinforced concrete structures, several kinds of attack may be distinguished. The present paper concentrates on modelling of reinforcement corrosion, the assessment of chloride concentration fields by numerical
methods and on their practical utilization while evaluating the limit states, service lives and relevant reliability of reinforced concrete structures. The presented experimental work shows the effect of chloride concentration and pH values on the rate of steel reinforcement corrosion. The development of the concentration of chlorides within the cross section of an RC structure is modelled by a specific cellular automata (CA) technique. The results of the CA technique and the laboratory experiments are applied for the assessment of steel corrosion by means of some existing analytical models which were randomized (Teplý et al., 2007). Finally, the introduced approach is applied to an illustrative example showing the effect of chloride concentration level at different locations in an RC structure on the rate of steel corrosion in time and also the effect of seasonal de-icing salt application.

Prevention of steel corrosion has to be taken into account already during the design stage. However, currently the future expenses for maintenance and repair are often not taken into account. The presented approach may enhance the possibility of exploring the limits of the materials and technologies in question under various conditions and help in decision-making procedures.

2 STEEL CORROSION

In this chapter chemical and physical processes leading to corrosion initiation and propagation, the types of corrosion identified as affecting steel reinforcement and the effect of chlorides presence on steel corrosion are introduced based on a survey of the literature. Within the scope of this chapter, experimental work proposed by the authors of this paper on the effect of chloride concentration and pH level is presented.

2.1 Corrosion initiation
In common concrete the reinforcement exists in an alkaline environment (pH > 12) with a high content of hydroxide ions (OH⁻) originating from the dissociation of calcium hydroxide and likewise from the dissociation of sodium and/or potassium hydroxide. The calcium hydroxide is formed during the hydration of cement and sodium or potassium hydroxide during the hydrolysis of minerals containing Na₂O or K₂O, respectively. Under these conditions the reinforcement surface is covered with a dense and effective adhesive protective layer with a thickness of approximately 10 nm (steel passivation). This durable coating of iron oxides protects the reinforcement against corrosion preventing direct contact between steel and oxygen, water and other aggressive agents (Schueremans et al., 2007).

Generally, corrosion can begin due to the presence of oxygen and moisture when the passive film is destroyed (after reinforcement depassivation) by:

(i) a decrease in the pH value of the pore solution to values below 9. Such a reduction in alkalinity is the result of the carbonation process, i.e. of calcium hydroxide reactions with atmospheric CO₂ that diffuses through the concrete's pores. Its modelling and some consequences for RC structure durability are dealt with e.g. in (Teplý et al., 2008). The pH reduction can also be caused in some cases and in a less pronounced way by other chemical substances such as SO₂, NOₓ.

(ii) chloride ions penetration when the chloride content of material in contact with reinforcement exceeds a critical chloride concentration.

Note that the penetration of the agent may be accelerated by local damage to the concrete cover. Thus steel depassivation can be speeded up as well.

2.2 Corrosion propagation

The corrosion of reinforcement embedded in concrete is an electrochemical process during which coupled anodic and cathodic reactions take place. In the anodic reaction the iron (Fe)
dissolves into a solution as a positively charged divalent ion (Fe$^{2+}$). This oxidation reaction is mostly balanced by a reduction reaction in the cathodic areas. The oxygen accepts the electrons and together with water forms hydroxide ions. Pore water due to its electrolytic conductivity functions as an electrolyte and closes the electrical circuit between anodic and cathodic areas. The hydroxide ions form together with Fe$^{2+}$ ferrous hydroxide Fe(OH)$_2$ which precipitates at the anode. Subsequently, this hydroxide reacts with oxygen and water to produce insoluble ferric oxide Fe$_2$O$_3$, respectively FeO(OH). These rust products may also consist of other oxidic modifications depending on the amount of moisture and oxygen in the vicinity. In the most general way the composition of rust is described by formula $x$FeO$_y$Fe$_2$O$_3$zH$_2$O (Nürnberger, 1984).

2.3 Types of corrosion

The uniform and pitting types of corrosion are generally differentiated. Uniform (general) corrosion usually affects a significant part of the reinforcement surface with more or less uniform loss over the perimeter of the rebar. It occurs typically in the case of steel depassivation due to carbonation. In contrast, pitting (localized) corrosion influences small areas of reinforcement but causes a considerable reduction in the effective area of the rebar at such a location and makes the cross-section of the rebar more brittle, which may lead to sudden failure of the structure (Duprat, 2007). The pitting type of corrosion is mostly propagated in the presence of chlorides; in addition, cracks in the concrete cover or/and local damage to the bar surface are a prerequisite to this type of corrosion. Some more data about typical features of pitting corrosion may be found e.g. in (Pourbaix & L’ Hostis, 2006).

2.4 Effect of chlorides
More specialized knowledge about the progress of chloride concentration in a concrete structure as a function of time is needed as it affects the reinforcement depassivation and the subsequent corrosion process.

The level of chlorides at the reinforcement surface which results in a significant corrosion rate leading to reinforcement corrosion in concrete may be called the critical chloride concentration (the chloride threshold concentration). Corrosion rates exceeding values of 1-2 mA/m² are often regarded as being significant for reinforcing steel (Glass & Buenfeld, 1997).

The chloride threshold concentration is preferably presented by means of the total amount of chloride by weight of cement, the amount of free chloride, the concentration ratio of free chloride ions to hydroxide ions or the ratio of acid-soluble chloride content and the acid neutralization capacity (the content of acid needed to reduce the pH of concrete and cement paste suspended in water to a particular value) (Ann & Song, 2007). In terms of currently used representations, the total chloride content related to the cement weight is considered as the best alternative. This representation results in a reduction in the range of determined values of critical chloride concentration and represents the total potential aggressive ion content expressed relative to the total potential inhibitor content (Glass & Buenfeld, 1995, 1997; Duprat, 2007; Ann & Song, 2007).

One reason for the lack of agreement among the measured values of critical chloride concentration is the influence of several factors such as chloride binding, chloride mobility, steel interface (voidage, pre-rusting), cementitious binder (type of binder, C₃A content, pH), concrete barrier (cement type, amount of cement, w/c ratio, curing, concrete cover), and environmental factors (relative humidity, temperature, chloride type). The key factor was found to be the physical condition of the steel-concrete interface (Glass & Buenfeld, 1995; Ann & Song, 2007). Another reason is the differences among the methods of measurement of
the chloride threshold concentration, the chloride content at the steel surface and the time of onset of corrosion. The onset of corrosion may be detected by measuring half-cell potential, monitoring the macrocell current between an anode and a cathode, monitoring the corrosion rate measured by the polarisation technique or AC impedance method, or visual inspection. The value of 0.4 % for buildings exposed to a European temperate climate and 0.2 % for structures exposed to a more aggressive environment are currently suggested as the chloride threshold concentration (Glass & Buenfeld, 1995; Duprat 2007).

Several other findings on the effect of chlorides on corrosion (gained via a review of the literature or experimental evidence) are briefly mentioned in the following text:

• In the terms of a Pourbaix diagram (Pourbaix, 1976) an environment with a pH from 9 to 13 involves a high concentration of OH⁻ ions for steel protection. Hydroxide ions keep the protective layer of iron oxides on the steel surface. The Pourbaix graph is valid only for concrete without foreign substances such as sulphates, nitrates, and namely chlorides. When chloride ions are present at sufficient concentrations in solution in pores near the steel reinforcement, the corrosion can take place across the whole pH scale.

• With a given amount of chlorides in the pore water the corrosion risk will be much higher for carbonated concrete than for non-carbonated (Nürnberger, 1984; Schueremans et al., 2007). This is probably connected with the fact that the pH value of the pore solution is one of the factors affecting the chloride threshold concentration, whereas the chloride threshold concentration decreases with decreasing pH value (Glass & Buenfeld, 1997).

• The steel depassivation caused by chlorides usually occurs in non-carbonated concrete since chloride penetrates mainly due to moisture in the pore system whereas the diffusion of carbon dioxide needed for carbonation proceeds rather in concrete with lower moisture content (Nürnberger, 1984). The subsequent corrosion of reinforcement is the greatest at
the equilibrium moisture levels associated with a relative humidity of about 60 up to 90 \% at the concrete surface (Nürnberg, 1984).

- According to the results of Tutti’s experiment (see table 8.6 in (RILEM 1996)) the evolution of the corrosion rate with RH is very similar for both the depassivation due to carbonation and that due to chloride penetration. However, the corrosion rate in the presence of chlorides is much higher compared to carbonation. This confirms the presumption that the presence of chlorides increases the corrosion rate which is affected by chloride concentration. But also note that the results obtained by Tutti are rather inexact due to the fact that he used different types of concrete and structures and various environments.

- Increased temperature accelerates chloride ingress mainly due to thermal activation of the diffusion process (Jensen et al., 1999). The chloride diffusivity increases with temperature according to the Arrhenius relation, but the temperature level has no impact on the chloride binding (Nguyen et al., 2006).

2.4.1 Experiments

In this chapter is presented experimental work focused on the corrosion resistance of steel reinforcement in the presence of chlorides.

The reinforcing bars used were made from steel No. 10 425 according to Czech standard ČSN 42 5535. Steel bars with a diameter of 10 mm were cut to a length of 100 mm and an epoxide coating was applied on the cut faces. The steel specimens were weighed and placed into polyethylene flasks of 100 ml volume, filled with NaCl solution with different pH levels and three different chloride concentrations. Compositions of solutions, i.e. concentration of chlorides (Cl\(^-\)) and pH values, are presented in Table 1.
The specimens were taken out of the flasks every 60 days, cleaned from corrosion products and weighed in order to determine the weight loss. In the case of non-carbonated concrete, the pH value of the pore solution is about 12.5. For this pH value, the evolution of weight loss due to corrosion over time is shown in Fig. 1 for three different chloride concentrations. The influence of Cl\(^-\) concentration is significant.

The dependence of weight loss on the pH value over a time of 1.3 years obtained from experiments for three chloride concentrations is presented in Fig. 2. The pH value is clearly one of the factors influencing the chloride threshold concentration (see Chapter 2.4).

The loss in the rebar radius was calculated as a ratio of the weight loss to the specific weight of the rebar and the surface area. Due to the epoxide coating on the cut faces the lengths of the rebar samples remain the same. In the calculations, for the sake of simplification it is assumed that the corrosion is uniform even though pitting corrosion was present. Subsequently, the corrosion rate is determined by dividing the loss in the rebar radius by the corresponding time. The results of the corrosion rate for solutions with pH = 9 and 12.5 are presented in Table 2, showing the pronounced effect of Cl\(^-\) concentration.
This kind of simplified experiment differs from the conditions existing during the real (natural) behavior of reinforcement embedded in concrete: e.g. the influence of relative humidity, temperature and surrounding concrete were not taken into account. However, the results of the simplified experiment quantitatively reflect the loss of rebar due to corrosion in the presence of chlorides with different compositions of solutions. Thus, at least the trends for corrosion rate due to pH level and Cl\(^-\) concentrations may be captured in this way as it is shown in Chapter 4.2.

### 3 CHLORIDE PENETRATION: MODELLING

The simulation of chloride concentration distribution over the concrete structure will be introduced in the following sections by:

(a) application of analytical 1D models for chloride ingress into concrete;

(b) using the Cellular Automata (CA) that enables spatial estimation of the chloride penetration process (deterministic or stochastic alternative).

The conventional analytical 1D models serve here for certain comparisons and for finding the type of boundary rule for the CA technique further used for the numerical example in chapter 5.

### 3.1 Analytical models

Several approaches exist for description of the time-dependent process of chloride ingress in concrete. Many models utilize the widely-used Crank’s solution to Fick’s 2nd law of diffusion probably first applied by Collepardi et al. (1972). This approach is based on the fact that
observations indicate that the transport of chlorides in concrete is mainly diffusion-controlled, and thus the convection zone is relatively small (Hunkeler, 2005). The solution is derived from the assumption that concrete is homogenous, initial chloride content \( C_0 \) is zero, surface chloride content \( C_{S,0} \) is constant, chloride binding is linear and the effect of co-existing ions is constant. The model in one dimensional form reads:

\[
C(x, t) = C_{S,0} \left[ 1 - \text{erf} \left( \frac{x}{2 \sqrt{D_a t}} \right) \right]
\]  

where \( C(x, t) \) is concentration of chlorides at depth \( x \) at time of exposure \( t \) (usually \( x = a \), where \( a \) is concrete cover) and \( D_a \) is the apparent diffusion coefficient. \( D_a \) and \( C_{S,0} \) are estimated from field exposure or laboratory tests.

The problem of proper representation of \( D_a \) has been tackled on several levels of simplification. The presumption of \( D_a \) being constant in time and space is too conservative (Tang & Gulikers, 2007), nevertheless for ordinary concrete without blended materials this simplification can be used (Duprat, 2007). Tikalsky (2005) overcomes this problem partially by using the statistics of a data set collected from the in-situ measurements of chloride penetration from more than 230 bridge decks in the United States. The time-dependent diffusion coefficient \( D(t) \) may also be calculated on the basis of the proposed formula (Tang & Nilsson, 1992; Thomas & Bamforth, 1999; Mangat & Molloy, 1994; fib Model Code); however, the proper mathematical derivation of \( D(t) \) and comparison of possible errors caused by some oversimplified mathematical expressions were presented by Tang & Gulikers (2007).

Further, it has been found that the surface chloride concentration \( (C_{S,0}) \) increases with time (Nilsson, 2006; Tang & Gulikers, 2007; Costa & Appleton, 1999). Nevertheless, Duprat (2007) assumed that constant surface chloride content is a commonly accepted assumption in the case of structures exposed to de-icing salts, and that chlorides accumulate only in coastal zones.
Although the above-mentioned models based on an error function complement solution are widely used by engineers in practical applications due to their relatively simple mathematical expressions, the omission of chloride binding is a weakness of these models. However, while models based on actual physical or chemical processes would be more adequate, they are often used only for research purposes owing to the necessity for more involved types of calculations. Only some models have been simplified into an engineer-friendly form (Papadakis et al., 1996; Tang, 2007).

The majority of the above mentioned models were originally published as deterministic ones. As explained above, the probabilistic approach is believed to be more adequate - among other benefits such an approach enables one to execute a probability analysis as indicated by condition (1). Therefore, the authors of the present paper have converted some of the models into probabilistic form (Chromá et al., submitted).

3.2 Cellular Automata

A cellular automaton (CA) is a special class of evolutionary algorithm, which is a mathematical idealization of physical systems in which space and time are discrete (Wolfram, 1994). In principle, any physical system satisfying differential equations may be approximated as a cellular automaton by introducing discrete coordinates and variables, as well as discrete time steps. A cellular automaton consists of a collection of cells on a grid of specified shape that evolves through a number of discrete time steps according to a set of local rules based on the states of neighbouring cells. The rules are then applied iteratively for as many time steps as desired. The overall simulation time is then the sum of all time steps.

The CA solution of the diffusion equation was adapted from Biondini et al. (2004). The construction is for our purposes represented by a 2D grid of regular uniform cells. Each cell has its own state value representing the component concentration (e.g. the chloride ions).
The process of chloride ingress is governed by a local rule in which the evolutionary coefficients $\Phi_i (i = 1, 2, 3, 4, 5)$ assign the level of chloride concentration redistribution within the cell’s neighbourhood (see Fig. 3):

$$X_{i(t+1)} = \Phi_1 X_i + \Phi_2 N_i + \Phi_3 E_i + \Phi_4 S_i + \Phi_5 W_i$$  \hspace{1cm} (3)

where the discrete variables \{X, N, E, S, W\} represent the concentration of the component in the given cell at time $t$. The values of the evolutionary coefficients $\Phi_i$ must verify the following normality rule:

$$\sum \Phi_i = 1$$  \hspace{1cm} (4)

as required by the mass conservation law. Clearly, for isotropic media the symmetry condition $\Phi_2 = \Phi_3 = \Phi_4 = \Phi_5$ must be adopted in order to avoid directionality effects. It can be proven that the values $\Phi_1 = 0.5$ and $\Phi_{2,3,4,5} = 0.125$ lead to the automaton having a very good level of accuracy. The relationship between the cell size $\Delta x$, time step $\Delta t$, diffusion coefficient $D$ and chloride evolutionary coefficient $\Phi_1$ (governing) is mandatory for the whole grid of cells within each time step:

$$D = \Phi_1 \Delta x^2 \Delta t^{-1}$$ \hspace{1cm} (5)

Stochastic effects may be dealt with as well, modifying the procedure by assuming the evolutionary coefficients $\Phi_i$ to be random values with a given PDF.
One of the essential parts of the CA configuration is the boundary rule setting, or in other words the definition of the system’s behaviour in the areas where there is contact with the outer medium, i.e. with a place beyond the simulation region. Several types of boundary rule have been implemented. The rule which the most suitably fits the results of conventional analytical 1D models is the mirror neighbour rule of hemisphere action, where the state values of the adjacent cells are governed according to the boundary region condition in such a way that the sum of all state values within a defined cell’s neighborhood (Fig. 3) is equal to the same sum in the successive time step. Comparison with the model proposed by Papadakis et al. (1996) renders almost identical results in a geometry where component concentration at a certain point at a certain time is affected only by propagation from one direction (this comparison is not presented within the scope of this paper). Note that this approach to the boundary problem is suitable only for the case where it is supposed that the transported quantity does not leave the simulation region once it enters. This might be the case for chloride ions, although it is known that in reality there is some kind of back-propagation. It is a topic of the authors’ ongoing research to appropriately define the boundary system interaction for different transport phenomenon; however for the purpose of this work it was rational to use some degree of simplification when dealing with the task of boundary interaction.

In order to test and further exploit the proposed methodology the standalone software CATES is being developed – a deterministic and stochastic 2D application featuring cellular automata. The application supports cooperation with ATENA 2D non-linear fracture mechanics finite element software (Červenka & Pukl, 2005). The geometry of the construction is primarily modelled in ATENA and afterwards loaded by CATES. At this point
a grid of CA is created according to the initial parameters. The main parameters are: the diffusion coefficient, cell size, time step and the evolution coefficient – see Eq. (5). After defining the aggressive environment (e.g. supply of chlorides from a de-icing salt application) the transformation process may begin. After each time step a text file with the current system state is created. The whole process is displayed on the screen as a real-time graphical visualization. Proper interpretation of these results provides information about local changes in the structure (e.g. cross section) properties (chloride concentration) over time. An example is shown in Fig. 4 where a visualization of the degradation evolution in a beam cross-section is shown for the total times of 0, 5, 25 and 50 years. The colours from grey to black represent the state from zero to higher chloride concentration in cells.

The environmental impact, which doesn’t have to be invariant, is expressed via boundary cells. These may change their state value over time, either periodically, by a specified function, or randomly. Also, the contact region may be subjected to differentiation by the use of various configurations within particular time – space states. In this manner it is possible to simulate various events, for example the seasonal effect of de-icing salt application. It was presumed in this example that the salt feed occurs periodically for 22 years in the winter season. Fig. 5 illustrates a detail of such variability in time. Fig. 6 shows the resulting chloride concentrations at certain depths of concrete caused by periodic salt application. It is not surprising that the concentration fluctuation is more obvious at the propagation surface and becomes less apparent with the depth of the specimen. It is also worth mentioning that the transport effect proceeds continually, even if there is currently no chloride feed, e.g. in the summer. Effects as salt washing out during summer rain in the exposed parts of the structure can also be treated by CA (not executed in the present paper). Clearly, the CA technique is more powerful compared to analytical 1D models.

*Fig. 4 here*
4 REINFORCEMENT CORROSION: MODELLING

This chapter briefly describes analytical models adopted from the literature and further used in the numerical example (chapter 5), and the possible application of the experimental data presented in section 2.4.1 to the utilized analytical models.

4.1 Analytical models

The formula for the penetration depth $x$ [mm] at exposure time $t$ [years] (time period after reinforcement depassivation) for the prediction of corrosion based on the application of Faraday’s law reads (Rodriguez et al., 1996):

$$ x = 0.0116 i_{\text{corr}} t $$

(6)

where $i_{\text{corr}}$ is the value of the corrosion rate [$\mu$A/cm$^2$]. The constant 0.0116 is a conversion factor from $\mu$A/cm$^2$ to mm/years under the assumptions that iron (Fe) has $n = 2$ (number of electrons freed by the corrosion reaction), $M = 55.85$ g/mol (molar mass) and $\rho = 7.88$ g/cm$^3$ (specific gravity).

The time related net rebar diameter $d(t)$ can be estimated according to Rodriguez et al. (1996) as (see Fig. 7a):
where $d_i$ is the initial bar diameter [mm] and parameter $R_{corr}$ [-] depends on the type of corrosion. Due to the assumed rotational symmetry in the case of uniform corrosion obviously $R_{corr} = 2$ (see Fig. 7a). In the case of pitting corrosion $R_{corr}$ represents a ratio between maximum and average corrosion penetration (i.e. the ratio between the rate of pitting and uniform corrosion penetration) and equals 4-8 according to Gonzales et al. (1995); while Darmawan & Stewart (2007) suggest $R_{corr} = 8-11$ and Tutti (1982) recommends $R_{corr} = 4-10$.

The studies by Gonzalez (1995) were performed on reinforcing steel embedded in concrete with a constant amount of chlorides added to the cement for both natural and accelerated conditions. Darmawan & Stewart (2007) performed accelerated tests on prestressed wires embedded in concrete with a constant amount of chlorides added to the cement. Tutti (1982) did not consider the effect of chlorides and used an external voltage source as a corrosion accelerator. However, all of the above-mentioned experimental work was performed without consideration of pH level and without exact consideration of different values of chloride concentration.

There is thus a very broad range of values for the parameter $R_{corr}$, and it is rather uncertain. From the geometrical point of view Eq. (7) predicts a conservative value for net rebar diameter in the case of pitting corrosion – see Fig. 7b. This simplification has been improved by Val & Melchers (1998), who considered the hemispherical shape of pit, see Fig. 7c, where the pit depth $p$ in time $t$ is given as:

$$p(t) = x R_{corr}$$ (8)
4.2 Effect of chloride concentration

The effect of chloride concentration near the reinforcement and other factors such as humidity, pH level, etc. on corrosion rate penetration (Eq. 6) should be included in the parameter of $i_{corr}$. Let us mention a work by Engelund & Faber (2000), where they published the formula for electrical resistivity, which may be recalculated to $i_{corr}$ according to the frequently used empirical expression (Alonso et al., 1988). They consider the effect of concrete curing, temperature, the presence of chlorides and relative humidity via individual coefficients. According to them a more detailed description of the estimation of model parameters is given in (Duracrete, 1998). Unfortunately, this document is not available to the public.

On the basis of our experiments presented in section 2.4.1 the parameter $i_{corr}$ was derived from the measurements as a function of chloride concentration for two different pH values of 9 and 12.5 (Fig. 8). It was recalculated from the corrosion rate found on the basis of experiments and rearranged on purpose to create a transformation into a uniform type of corrosion (Table 2) according to Eq. (6). The pH values of 9 and 12.5 were chosen due to the fact that the pH value of the pore solution is about 12.5 in the case of non-carbonated concrete and pH = 9 in the case of carbonated concrete. The proposed values of $i_{corr}$ are applied in an illustrative example in section 5. Note that the proposed functions for $i_{corr}$ may be applied for calculations of both the uniform (Eq. 7) and pitting (Eq. 7 or 8) types of corrosion; however the coefficient $R_{corr}$ for pitting corrosion has to be adopted from other sources as it was not determined in our experiments. By comparison of the method used in these experiments and the real conditions our $i_{corr}$ values are certainly somewhat on the safe side.
5 NUMERICAL EXAMPLE

In the presented example a reinforced concrete rectangular cross section is exposed to chloride ingress simulated by the CA technique. To illustrate the versatility of CA four different cases are assumed that differ in the surface areas exposed to chloride action as shown in Fig. 9, together with their cross section geometry. Surfaces bounded with a dashed line were supplied with chlorides in such a way that the surface concentration remained constant over the whole time period. The figure also documents the chloride distribution in the cross sections after 30 years of exposure. In case IV the seasonal chloride effect was also applied as illustrated in Fig. 5.

The following input data for chloride diffusion simulation by cellular automata were used: surface concentration of chlorides 60 mol/m$^3$ (in the case of seasonal effect this value corresponds to the maximum concentrations in Fig. 5), cell size 0.0032 m, time step 7.402 days, diffusion coefficient $2 \times 10^{-12}$ m$^2$/s and evolution coefficients 0.5 and 0.125 for central and surrounding cells, respectively. As already mentioned above the mirror neighbour boundary rule was applied in this example. The developments of chloride concentrations over time in the vicinity of steel reinforcements R1, R2 and R3 are plotted in Fig. 10. Fig. 10a compares the chloride ingress evolution for boundary conditions I-IV with a constant surface concentration (CC) and Fig. 10b shows the difference between constant surface concentration and seasonal effect (SE) for boundary condition IV. The chloride concentration in the individual time steps is determined as an average concentration from the cells representing
concrete that bounds the steel cells. Due to certain symmetries in boundary conditions some of the rebars are attacked identically.

Fig. 9 here

Fig. 10 here

When we apply Eq. (7) using $R_{corr} = 2$ (uniform corrosion), $d_i = 16$ mm and the functions for $i_{corr}$ in Fig. 8 (namely pH = 12.5 pointing to non-carbonated concrete) we obtain a drop in the rebars’ diameters over time due to corrosion. The results of the analysis are plotted in Fig. 11a for boundary conditions IV (R1-R3) and III (R1 and R3). Both CC and SE conditions are compared for boundary type IV. Note that the rebars R1, R3_III and R1_IV are the ones with the highest and the lowest chloride concentrations, respectively, as seen in Fig. 10a. Let us assume random input parameters and perform a probabilistic computation for rebar R3_IV (CC) using FReET-D software developed by the authors of the present paper (Teplý et al., 2007) with the input parameters given in Table 3.

Table 3 here

FReET-D is a product associated with the multipurpose probabilistic software for statistical, sensitivity and reliability analysis of engineering problems, FReET (Feasible Reliability Engineering Tool), which is based on efficient reliability techniques (Novák et al., 2003; Vořechovský & Novák, 2003). The FReET-D tool has been developed by implementing a number of degradation models for reinforced concrete structures. A mutual statistical dependence between input variables can be prescribed and is arranged by a
simulated annealing method. Some models may be highly input-demanding; in order to simplify the handling of inputs their statistical sensitivity is provided by means of Spearman rank-order correlation coefficients, i.e. the user may easily gain measures of the relative effect of each basic variable. For performance of the statistical analysis the Monte Carlo method, its modification the Latin Hypercube sampling method, and FORM technique are available. For the output quantity the best-fitted PDF may be found by means of the Kolmogorov Smirnov goodness-of-fit test.

The results of the probabilistic analysis are given in Fig. 11a by means of the best fit of the probabilistic distribution function (PDF) of the resulting rebar diameter at time $t = 30$ years and the standard deviation (std) region of the output during the whole analyzed time interval.

According to various authors (Glass & Buenfeld, 1995, 1997; Duprat, 2007; Ann & Song, 2007), the corrosion of steel embedded in concrete starts to propagate at a certain level of chloride concentration called critical concentration depending also on other factors such as pH level, etc. Let us apply for pH = 12.5 a critical concentration of 9.48 mol/m$^3$, which corresponds to a Cl$^−$/OH$^-$ ratio of 0.3, considered as a signal for steel depassivation due to chlorides recommended by Papadakis et al. (1996). The results of the deterministic analysis performed with the assumption of zero corrosion penetration if the chloride concentration in the vicinity of steel rebar is lower to the critical one are plotted in Fig. 11b. By comparison of both Fig. 11a and Fig. 11b it is evident that the differences for R3_IV (CC) and R1, R3_III (CC) at $t = 30$ years are quite negligible; they are 0.25 and 0.14 %, respectively. The other rebars in Fig. 11b did not reach the assumed critical chloride concentration (compare in Fig. 10) in the analyzed time interval, thus corrosion had not yet been initiated.
Let us construct a probability condition:

\[ P_f = P[A_{R1}(t) + A_{R2}(t) + A_{R3}(t) \leq A_{cr}] \quad (9) \]

where \( P_f \) is the probability of failure, \( A_{cr} \) is the critical area of steel in the whole cross section (leading e.g. to the exceeding of SLS or ULS) and \( A_{R1}(t) \), \( A_{R2}(t) \) and \( A_{R3}(t) \) are the areas of steels R1_IV, R2_IV and R3_IV, respectively, at time \( t \). We assume that \( A_{cr} \) is 90 % of the initial rebars’ areas in the concrete cross section, and that it is deterministic. The other inputs are taken from Table 3 without consideration of the critical chloride concentration. Again utilizing the FReET-D module we arrive at \( P_f \) and corresponding reliability coefficients \( \beta \) for the chosen time steps – see Table 4 for both CC and SE types of chloride supply. Comparing these values with recommended \( \beta \geq 1.3 \) for SLS (\textit{fib} 2006) and/or \( \beta \geq 3.8 \) for ULS (EN 1990) it is evident that in this example for the analyzed time interval, the degradation of the structural capacity of the cross section is acceptable with respect to SLS but not to ULS for both CC and SE boundary conditions.

\textit{Table 4 here}

6 CONCLUSIONS AND IMPLICATIONS IN PRACTICE

A probabilistic approach for the durability assessment of concrete structures, focusing on reinforcement corrosion, has been presented together with suitable numerical models and software tools enabling the user to assess RC structure durability and reliability. The impact
of chloride presence on corrosion rate is shown together with a special technique – cellular automata – for capturing the distribution of chloride concentration in a structural member. This creates the potential for more complex design/assessment of RC structures, utilizing statistical, sensitivity and reliability analyses of durability, serviceability and ultimate limit states in the context of current trends represented e.g. in future documents (fib Bulletin 34 and ISO 13823). The presented approach supports design procedures accommodating possible corrosion of RC structures and may help to reduce heavy repair costs in the future.

ACKNOWLEDGEMENT

This outcome has been achieved within the activities of the CIDEAS research centre with the financial support of the Czech Ministry of Education, project No. 1M0579. In this undertaking, theoretical results gained in the project GA CR 103/07/0034 were partially exploited. It is also a part of the work carried out in the frame of the CIB committee W080 tasks.
REFERENCES


ISO 2394 (1998), General principles on reliability for structures, ISO.


<table>
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<tr>
<th>pH</th>
<th>9.0</th>
<th>9.5</th>
<th>10.0</th>
<th>10.5</th>
<th>11.0</th>
<th>11.5</th>
<th>12.0</th>
<th>12.5</th>
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<td>0.1413</td>
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<td>[mol/l]</td>
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<td>0.0141</td>
<td>0.0141</td>
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Table 2 The corrosion rate for pH = 9 and 12.5.

<table>
<thead>
<tr>
<th>pH value</th>
<th>Cl⁻ [mol/l]</th>
<th>Corrosion rate [μm/year]</th>
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<td>0.0141</td>
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<td></td>
<td>0.1413</td>
<td>17.90</td>
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### Table 3 Input parameters.

<table>
<thead>
<tr>
<th>Input parameter</th>
<th>PDF</th>
<th>Mean</th>
<th>COV [%]</th>
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<tr>
<td>Initial rebar diameter, $d_i$</td>
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<td>2.5</td>
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<tr>
<td>Current density, $i_{corr}$</td>
<td>normal</td>
<td>Fig. 8</td>
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<tr>
<td>Coefficient, $R_{corr}$</td>
<td>deterministic</td>
<td>2</td>
<td>-</td>
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Table 4 The results of the probability condition (9).

<table>
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<tr>
<th>Time [years]</th>
<th>30</th>
<th>20</th>
<th>10</th>
<th>5</th>
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<td>Cornell $P_\gamma$ [-]</td>
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<td>Cornell $\beta$ [-]</td>
<td>1.74</td>
<td>1.87</td>
<td>1.96</td>
<td>2.00</td>
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</table>
The list of figures’ legends:

Fig. 1. Steel rebar weight loss over time for pH = 12.5.

Fig. 2. Steel rebar weight loss for different pH values.

Fig. 3. Von Neumann’s neighbourhood of a single cell - cf. equation (3) (left) and a detail of a grid of uniform cells (right).

Fig. 4. Numerical example: AASHTO IV 711.2/1828.8 mm; cross section degradation evolution visualization; 1871 cells of 20×20 mm; the interval ⟨0;1⟩ represents the relative chloride concentration.

Fig. 5. Schematic of seasonal salt application.

Fig. 6. The effect of seasonal de-icing salt application at different depths of concrete and the concentration on the surface (external salt feed), which is multiplied by 0.1.

Fig. 7. The geometry of corroded steel rebar: (a) uniform corrosion (Rodriguez et al., 1996); (b) pitting corrosion (Rodriguez et al., 1996) and (c) pitting corrosion (Val & Melchers, 1998).

Fig. 8. The dependence of current density on chloride concentration.

Fig. 9. Chloride ingress for four different boundary conditions (I, II, III and IV). Cross sections attacked by chloride are in grey and sections without chloride are black. Steel reinforcements are denoted R1, R2 and R3.

Fig. 10. Development of chloride concentration in the vicinity of the steel rebars: (a) constant surface chloride concentration for boundary conditions I-IV and (b) comparison of constant surface chloride concentration (CC) and seasonal effect (SE) for boundary condition IV.

Fig. 11. The drop in rebars’ diameters over time due to corrosion for boundary conditions III and IV (a) without and (b) with the assumption of critical chloride concentration according to Papadakis et al. (1996).
Fig. 1

![Graph showing weight loss over time for different concentrations of a solution.](image)

- **0.0014 mol/l**
- **0.0141 mol/l**
- **0.1413 mol/l**
Fig. 2

![Graph showing weight loss percentage vs pH for different concentrations of a solution.](image-url)

- **Weight loss [%]**
- **pH**
- **Concentration Levels:**
  - 0.0014 mol/l
  - 0.0141 mol/l
  - 0.1413 mol/l

The graph displays weight loss percentage (%) on the y-axis and pH on the x-axis for different concentrations of the solution.
Fig. 4

Surface exposed to chloride action

Surface exposed to chloride action
Fig. 5

\[ \text{Time, } t \text{ [years]} \]
Fig. 6

![Graph showing chloride concentration over time at different depths]

- Depth 7.4 mm
- Depth 14.8 mm
- Depth 30 mm
- Surface (×0.1)

Chloride concentration, $C$ [mol/m$^3$]

Time, $t$ [years]
Fig. 7

a) $x$, $d(t)$, $d_r$

b) $x$, $R_{orr}$, $d(t)$

c) $a_r$, $p(t)$
Chloride concentration, $C$ [mol/m$^3$]

Current density, $i_{\text{corr}}$ [A/cm$^2$]

- $i_{\text{corr}} = 0.0368C + 1.2033$ for $C < 14.1$
- $i_{\text{corr}} = 0.0073C + 1.6186$ for $C = (14.1; 141.3)$
- $i_{\text{corr}} = 0.0718C + 0.0347$ for $C < 14.1$
- $i_{\text{corr}} = 0.0039C + 0.9924$ for $C = (14.1; 141.3)$

Experiment, pH = 9
- Experiment, pH = 12.5
Fig. 10

Chloride concentration, $C$ [mol/m$^3$]

Time, $t$ [years]

a) R1, R3, III
   - R1, II
   - R2, III
   - R1, R3, I, R3, II, R3, IV
   - R2, IV
   - R1, IV
   - R3, IV

b) R3, IV (CC)
   - R2, IV (CC)
   - R2, IV (SE)
   - R1, IV (CC)
   - R1, IV (SE)

0 5 10 15 20 25 30
0 5 10 15 20 25 30
Fig. 11

Net rebar diameter, $d(t)$ [mm]

Time, $t$ [years]

R1_IV (CC)
R2_IV (CC)
R3_IV (CC)
R1_IV (SE)
R2_IV (SE)
R3_IV (SE)
R1, R3_III (CC)

Beta PDF

{... R1_IV (SE)
... R1_IV (CC)
... R2_IV (SE)
... R2_IV (CC)
... R3_IV (CC)
... R1, R3_III (CC)}