Modelling of Reinforced Concrete Element Damage as a Result of Reinforcement Corrosion

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Abstract

This paper presents the application of the transient layer growth model for describing concrete damage caused by the reinforcement corrosion. The computer model was developed which provided the description of the degradation process of a cover caused by chloride diffusion, including the impact of moisture and temperature and subsequent reinforcement corrosion leading to cover cracking as a result of the increased volume of corrosion products. The evolution of corrosion current density over time was assumed in the form of the empirical function \[1\]. In the mechanical analysis of that problem, concrete was described using the Drucker-Pager model, and the progress of chloride diffusion, moisture and heat were determined from thermomechanical equations. The computer calculations were performed using FEM and FVM techniques.

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1. Introduction

Environmental pollution has a negative impact on the durability of the reinforced concrete structures. Such aggressive substances as carbon dioxide or chloride ions lower the protective properties of concrete and pose a risk of reinforcement corrosion. In properly designed reinforced concrete structures, the corrosion should not occur for decades. However, despite the adequate thickness and tightness of the cover, undesirable changes occur after that time due to the diffusion of aggressive substances from the environment. The commonly adopted model expressing the condition of reinforcing bars in the reinforced concrete elements over time assumes a three-stage period of changes occurring in the cover \[2\]. In this three-stage model, the time \(t_{\text{exp}}\) of the construction exploitation is identified with the time \(t_{\text{in}}\) required for the appearance of the conditions leading to the initiation of the electrochemical corrosion process of the reinforcement (penetration time of the aggressive substances in the cover) and the time \(t_{\text{d}}\) of the cover degradation that can be divided into the time \(t_{\text{act}}\) for the activation of the cover degradation as a result of developing reinforcement corrosion, and the time \(t_{\text{c}}\) of cover cracking

\[t_{\text{exp}} = t_{\text{in}} + t_{\text{d}} = t_{\text{in}} + t_{\text{act}} + t_{\text{c}}.\] \hspace{1cm} (1)

While calculating the building durability expressed by the Eq. (1), it is very important to determine the time \(t_{\text{act}}\) of process activation and the time \(t_{\text{c}}\) of cover cracking. The attempts to evaluate these values in total can be found, inter alia, in the papers \[1\]. In these papers, the time \(t_{\text{act}}\) of the process activation was defined as the time needed to fill voids with the products of steel corrosion in the transient zone between the reinforcement and the concrete.

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A problem related to the calculation of the time $t_c$ of cover cracking has been widely discusses in the literature. In the vast majority of the papers, it is assumed that the corrosion process in the reinforce has an impact on the whole rebar circumference. Such an approach (both numerical and analytical) was also adopted in the papers [3–9]. Regarding the reinforce corrosion, the decomposition of the corrosion products on the rebar circumference is not usually uniform, yet it takes place from the cover side and is connected with the progress in diffusion of substances in concrete which activate corrosion. An attempt to describe the problem of the non-uniform distribution of the corrosion products around the rebar circumference was presented in the papers [10–12].

There are not much papers describing comprehensively the model of cover degradation as a result of the reinforce corrosion. Particularly, there are not any papers describing the connection of the effect of transporting aggressive cover pores with the corrosion products, it is extremely difficult to predict the course of the function of corrosion current density. Also few papers discuss the problem on determining the activation time $t_{act}$ of the concrete cover degradation.

This paper presents the FEM-based computer model, in which the evolution of corrosion products on the rebar surface is dependent on moisture, temperature and the transport of the aggressive substance in the concrete cover. This model can be applied to evaluate the time $t_c$ of cover cracking on the basis of the mechanical analysis using the equations of plasticity theory. This paper also presents an attempt to define the activation time $t_{act}$ of the cover degradation on the basis of the changes in the experimental function of the corrosion current density.

2. Describing the efficiency of mass sources of corrosion products

The mechanism of initiating the reinforcement corrosion depends on the cause of lowering the protective properties of concrete. In the process of concrete carbonization, $pH$ of a pore solution is reduced to the level $pH \leq 11.8$. During the diffusion of chlorides, the risk of reinforcement corrosion occurs when the concentration of $Cl^-$ ions reaches the value of 0.4% of cement mass [13]. The paper [14] presents other criteria.

The decomposition of passive layer initiates the reinforcement corrosion which is the electrochemical process. The anodic and cathodic reactions take place simultaneously. The electrochemical reaction [1] produces the corrosion products whose chemical composition can be treated as a mixture of oxides $Fe(OH)_2$, $Fe(OH)_3$.

According to Faraday’s law, the velocity of transporting the mass of iron ions $m_{Fe^{2+}}$ to the pore solution is described by the following relationship:

$$m_{Fe^{2+}} = kl = kl^c, \quad I^c = \int I(t) dt, \quad I = i_{kor} A_c, \quad k = \frac{M_A}{nF},$$

where $t$ is the time year, $k$ – electrochemical equivalent of iron g/µA year, $I$ – intensity of the corrosion current µA, $A_c$ – side surface area of the rebar where the corrosion process of the reinforce occurs cm$^{-2}$, $M_A$ – atomic weight of dissolved iron g, $F$ – Faraday constant C, $n$ – atom valence number. Moreover, assuming that the ratio of the mass and density of iron ions transported to the pore solution is the constant value, the mass of corrosion products and their density can be defined [1], [15] in the following form:

$$m_R = \alpha^{-1} m_{Fe^{2+}}, \quad \rho_R = \vartheta^{-1} \rho_{Fe^{2+}},$$

where $\rho_R$ is the density of corrosion products, $\rho_{Fe^{2+}}$ – the density of iron ions, $m_R$ – the mass of corrosion products, $\alpha$ and $\vartheta$ – the parameters. These parameters are strongly dependent on the composition of the corrosion products. Assuming that the corrosion products are composed of the mixture of compounds $Fe(OH)_2$ and $Fe(OH)_3$, the following relationships [1], [15] can be expected for $\alpha$ parameter: $\alpha_{Fe(OH)_2} = 0.622$, $\alpha_{average} = 0.573$, $\alpha_{Fe(OH)_3} = 0.523$.

Taking into account this relationship (3), the compounds describing the formation rate of the mass of the corrosion products and the rate of changing the volume of the corrosion products in the function of the corrosion current intensity can be developed

$$\dot{m}_R = \alpha^{-1} \dot{m}_{Fe^{2+}} = \alpha^{-1} kl = \alpha^{-1} k^c, \quad \dot{V}_R = \frac{\alpha^{-1} k}{\rho_R} I = \frac{\alpha^{-1} \vartheta}{\rho_{Fe^{2+}}} I = \frac{\alpha^{-1} k}{\rho_R} I,$$

where $I$ is the intensity of the corrosion current, whereas $V_R$ – the volume of the corrosion products.

Due to the very complex chemical composition of steel-pore solution-cement grout system and the tightening of concrete cover pores with the corrosion products, it is extremely difficult to predict the course of the function of corrosion current...
intensity over time solely on the basis of the electrochemical calculations. The empirical approach is an alternative method for describing this problem. For the purpose of this paper, the empirical values defined in the papers [1], [4] were used to describe the function of the corrosion current density

\[ i_{\text{kor}} = 0.92 \exp \left( 8.37 + 0.618 \ln \left[ 1.69 \rho_f \right] - \frac{3034}{T} - 0.000105 R_{c,\text{res}} + 2.32 t^{-0.215} \right), \quad (5) \]

\[ R_{c,\text{res}} = 90.537 \phi^{-2.2548} \left[ 1 + \exp \left( 5 - 50 \left( 1 - \phi \right) \right) \right], \quad (6) \]

where \( \rho_f \) is the content of chlorides expressed in kg/m\(^3\), \( T \) – temperature K, \( R_{c,\text{res}} \) – electrical resistivity of concrete Ω, \( t \) – time year.

### 3. Concept of the transient layer growth

The environmental impact on the reinforced concrete elements results in the progress of the zone with the reduced \( pH \) or containing chloride ions, in the cover towards the reinforcement. The subsequent places on the reinforcing rod surface are activated with the front progress of this zone. This phenomenon can be described by introducing locally a change in the material volume (increase in corrosion products). Among the ways of taking into consideration the local nature of the formation of corrosion products, there is an approach outlining that distortional strains occur in the transient layer between the reinforcement and the concrete. Such strains depend on the intensity or density of the corrosion current. By combining this concept with FEM approximation, it is possible to determine, analogous to thermal strains, the effect of the increase in corrosion products on the stress level in the cover. The process of corrosion products growth on the rebar surface is described by introducing the function of effective increase (in the mechanical sense) of corrosion products. Among the ways of defining such a function, there is an proposal suggested in the paper [3], where the effective volume of corrosion products is defined using the following equation:

\[ V_{\text{eff}} = V_R - V_F e^{2+}, \quad (7) \]

where \( V_R \) is the volume of the products of reinforcement corrosion, whereas \( V_F e^{2+} \) is the change in iron ions volume transported to the pore solution.

The main assumptions of the described method for evaluating the effects of the reinforcement corrosion in the cover can be presented in the form of three postulates:

- Corrosion cell is formed on the reinforcement surface after achieving the conditions favourable for the electrode process development (achieving the critical concentration when corrosion is caused by chloride ions or \( pH \) reduction to the level, at which protective properties of concrete disappear in the case of carbonization).
- The increase in the volume of finite elements in the transient zone is only related to the elements located in the neighbourhood of the point, where the electrode process has been initiated.
- In the case of a larger number of finite elements involved in the process (subjected to volume increase caused by the formation of products of the electrode reactions), it is assumed that the mass increase in the finite elements is distributed proportionally to the number of active finite elements.

The model of changes occurring in the concrete cover and the reinforcement as a result of the impact of environmental factors is presented in Fig. 1. By taking into account the relationships (3), (4) and the equation describing the rate of change of the first invariant of corrosion strains it is possible to formulate the tensor of corrosion strain rate

\[ \dot{\varepsilon}_{\text{kor}} = \frac{V_{\text{eff}}}{V_0} \varepsilon_{\text{kor}} = \alpha \varepsilon_{\text{kor}} \varepsilon_{\text{e}} + \varepsilon_{\text{kor}} \varepsilon_{\text{e}} = 3 \varepsilon_{\text{kor}} \varepsilon_{\text{e}} = \chi \varepsilon_{\text{e}}, \quad \chi = 1 \chi, \quad \chi = \frac{\left( \alpha^{-1} - 1 \right) k}{3 \cdot V_0 \rho F e^{2+}}, \quad (8) \]

In the relationship (8), \( \chi \) is the substitute isotropic tensor of corrosive expansion of the material, \( V_0 \) – initial volume of the examined area of the reinforcement subjected to corrosion (initial volume of the finite element), \( \chi \) – the substitute coefficient of corrosive expansion of the material, whereas \( 1 \) – the unit tensor of rank two.
4. Thermodiffusion description of concrete cover degradation

The analysis of the basic corrosive response of the reinforcement in the reinforced concrete, that is, anodic and cathodic reactions indicate that the process description should include water and oxygen molecules as well as iron and hydroxide ions. Water can be found in the liquid form, and as water vapour – in the gaseous form. The synthesis of iron and hydroxide ions produces the product of reinforcement corrosion – iron hydroxide which undergoes further chemical changes. The density (or intensity) of the corrosion current is a measurable macroscopic indicator describing the intensity of the corrosion process of the reinforcement.

The correct description of the concrete cover degradation as a result of the reinforcement corrosion should include at least six elements:

- a skeleton composed of the cement matrix with aggregate. It was assumed that oxygen was an integral part of this skeleton (in the process description, the intensity of the electrochemical reaction was expressed as the intensity of the corrosion current which enabled the negligence of oxygen concentration).
- An aggressive substance migrating in concrete pores, which initiates the process of reinforcement corrosion.
- Water migrating in concrete pores.
- Water vapour diffusing in concrete pores. A phase change can occur between water and water vapour.
- Components of the electrode reactions (iron ions $Fe^{2+}$ and $OH^-$ ions).
- Products of reinforcement corrosion.

It was assumed that there were not any additional chemical reactions between the skeleton, the aggressive substance, components of the electrode reactions and the corrosion products. By expressing the balance equation with an additional internal parameter – a function of corrosion current intensity \[16\], the relationships describing the degradation process in the concrete cover were obtained.

In accordance with the theory of multi-component media, the chloride ion mass balance equation can be expressed as \[16–19\],

$$\rho \frac{dc_1}{dt} + div j_1 = 0, \quad c_1 = c_b + c_f, \quad c_b = \omega c_{fb}, \quad c_f = wc_{fs},$$ \hspace{1cm} (9)

where $c_1$ is the total concentration of the mass of chlorides, $c_f$ – the concentration of unbounded chloride ions, $c_b$ – the mass concentration of bonded chlorides, $\omega$ – the adsorption coefficient connecting $c_b$ and $c_f$, $w$ – the volume concentration of water, $c_{fb}$ – the mass concentration of (unbounded) chloride ions related to the volume of the pore solution, whereas $j_1$ is the mass flux of chloride ions which can be defined as \[3\]
\[
\mathbf{j}_1 = -\rho D_c \text{ grad } c_f + \frac{\rho_f c_f}{\rho_w} \mathbf{j}_2, \quad \mathbf{j}_2 = -\rho_w D_\varphi \text{ grad } \varphi, \quad D_\varphi = D_w \frac{\partial \varphi}{\partial \varphi}.
\]

(10)

In the Eq. (10), \(\rho_w\) is the specific density of water, \(D_c\) – diffusion coefficient of chloride ions, \(D_w\) – capillary conduction coefficient, \(\varphi\) – relative humidity, \(D_\varphi\) – diffusion coefficient of relative humidity. Taking into account the Eq. (10) in the relationship (9), the free chloride ions mass transport equation can be obtained:

\[
A^{-1} \dot{\rho}_f = \text{div} \left( \rho D_c \text{ grad } c_f \right) + \text{div} \left( \frac{\rho_f}{\rho_w} \rho_w D_\varphi \text{ grad } \varphi \right) - \rho \dot{\mathbf{c}}_f, \quad c_1 = B c_f + c_f, \quad A = (B + 1)^{-1}, \quad B = \frac{\rho_2}{\rho_w}.
\]

(11)

The analysis of the transport process of chloride ion mass based on the Eq. (11) led to an assumption that the properties of the material constant values at changing parameters of physical fields: relative humidity and temperature could be expressed by the following relationships [3], [20]

\[
D_c = D_{c,ref} F_c(T) F_c(\varphi), \quad D_\varphi = D_{\varphi,ref} F_\varphi(T) F_\varphi(\varphi)
\]

(12)

In the Eqs (12), \(D_{c,ref}\), \(D_{\varphi,ref}\) are the diffusion coefficients determined under precisely defined conditions of relative humidity and temperature, \(F_c(T), F_c(\varphi)\) – the parameters correcting the diffusion coefficient of chloride ions, whereas \(F_\varphi(T), F_\varphi(\varphi)\) – the parameters correcting the diffusion coefficient of relative humidity.

The problems on humidity transport in concrete and the problems on phase changes of water and water vapour were discussed, inter alia, in the paper [21]. By summing up the balance equations of water and water vapour, the following equations are obtained

\[
\rho_w \dot{w} + \rho \dot{c}_3 + \text{div} \mathbf{j}_2 + \text{div} \mathbf{j}_3 = 0, \quad w = \frac{\rho_2}{\rho_w}, \quad \rho \dot{c}_3 \equiv 0, \quad \mathbf{j}_3 = -\frac{\delta}{\mu} \text{ grad } p,
\]

(13)

where \(c_3\) is the concentration of water vapour, \(\rho_2\) – volume density of water, \(\mathbf{j}_1\) – the diffusion flux of water vapour mass, \(\delta\) – the diffusion coefficient of water vapour in the air, \(\mu\) – the coefficient of diffusion resistance of dry material, \(p\) – partial pressure of water vapour.

A differential equation describing heat transport with a phase change water-water vapour could be derived by considering the energy balance equation of a multi-component medium. This equation for the physical changes examined in this paper, can be expressed as [21]

\[
\rho c \dot{T} = -\text{div}(\mathbf{q}) - \tilde{c}_3 \tilde{L}, \quad \tilde{c}_3 = \text{div} \mathbf{j}_3 + \rho \dot{c}_3 \equiv \text{div} \mathbf{j}_3, \quad \mathbf{q} = -\Lambda \text{ grad } T,
\]

(14)

where \(\tilde{L}\) is the heat of vaporisation, \(c\) – the specific heat of moist material, \(\Lambda\) – the tensor of thermal conductivity.

The problem on a mechanical degradation of the cover as a result of concrete cracking caused by the reinforcement corrosion products were described by formulating the momentum balance equation of a multi-component medium. It was assumed that components of a stress tensor \(\sigma\) depended on, among other things, an additional internal parameter, a function of corrosion current which could be expressed as [16]

\[
\text{div } \sigma + \rho \mathbf{b} = 0, \quad \sigma = C^e : \left( \varepsilon^e - \mathbf{\mu} (T - T_0) - \chi \left( \varepsilon^e - \varepsilon^p \right) \right), \quad \varepsilon = \varepsilon^e + \varepsilon^p
\]

(15)

where \(\varepsilon\) is the strain tensor, \(\varepsilon^e\) – the elastic strain tensor, whereas \(\varepsilon^p\) – the plastic strain tensor.

5. Verification of the computer model

The concrete element subjected to experimental testing presented in the paper [22] was analyzed. The process of reinforcement corrosion was initiated by adding sodium chloride to concrete mix. A spatial (detailed and simplified) FEM model was developed for the numerical analysis, which mapped \(\frac{1}{4}\) of the element applied in the tests. The analysed element along with the marked range of the precise model (lighter grey background) and the simplified one (darker grey background) is illustrated in Fig. 2a. The analysed precise computer model is presented in Fig. 2b. The models was
supported in a way providing a free strain of the element. A sequence of the calculation tests was performed on the element models to determine the time \( t_d \) of the cover degradation. The calculation model was verified assuming the constant value of the corrosion current density function [1]. The calculations were performed to find answers to the following questions:

- Does the proposed theoretical model provide results close to the results of the experimental tests?
- Is it possible to propose an equivalent simplified model of the analysed problem?

Both the precise and simplified models were analysed with FEM technique using 6- and 8-node solid elements. The material parameters were adopted from the paper [22, 23]. A bilinear model of concrete damage was assumed [24]. The concrete was assumed to be described with the Drucker-Pager model. The concrete damage was assumed to appear when the main tensile strain exceeded the value of the limiting strain. The value of this strain was calculated on the basis of the following relationship [23, 25]

\[
 w_c = \alpha_F \frac{G_F}{f_{ctm}}, \quad \varepsilon_{tc} = \frac{w_c}{h_c}, \quad h_c \approx 3d_{max}.
\] (16)

The material parameters required to conduct the mechanical analysis of the problem are: average tensile strength of concrete, \( f_{ctm} = 3.3 \) MPa; average compressive strength of concrete, \( f_{cm} = 31.5 \) MPa; Poisson’s ratio, \( \nu = 0.18 \); fracture energy, \( G_F = 78.7 \) Nm/mm\(^2\); secant modulus of elasticity, \( E_c = 18 \) GPa; limiting strain, \( \varepsilon_{tc} = 2.867 \times 10^{-3} \); limit value of elastic strain in concrete, \( \varepsilon_{c,d} = 0.00175 \); maximum diameter of aggregate particles, \( d_{max} = 19 \) mm; cohesion coefficient, \( c = 4.237 \times 10^6 \) N/m\(^2\); internal friction angle, \( \Theta = 59.886^\circ \); limit value of the width of a crack, \( w_c = 15.79 \times 10^{-2} \) mm. The mechanical analysis of the precise and simplified model was made using the ANSYS system.

The results from calculating the time \( t_d \) of the cover degradation based on the concept of the transient layer growth for the precise model are presented in Fig. 3a, whereas the results for the simplified model are shown in Fig. 3b. The obtained calculation results were compared to the experimental test results according to the paper [22] – Table 1. The compared values presented in Fig. 3a and 3b and in Table 1 indicate that the calculation results obtained with the use of the precise and simplified models and the experimental testing results have very good convergence.

Table 1. Test results [22] for verifying numerical models

| Sample marking acc. to [22] | Cover thickness \( c \) [mm] | Density of corrosion current \( i_{corr} \) [µA/cm\(^2\)] | Time for process activation \( t_{act} \) [month] | Time of cover degradation \( t_d \) (experiment) |
|-----------------------------|-----------------------------|------------------------------------------|---------------------------------|----------------------------------|
| OE(F)18512.0               | 27                         | 3.7674                                   | 1.5                             | 8.75                            |
| OA2859.6                   | 47                         | 2.3465                                   | 2.5                             | 22                              |
| OB3859.6                   | 70                         | 1.7976                                   | 3.25                            | 42.5                            |
6. Application of the transient layer growth concept to predict the degradation time of the cover

It was assumed that the analysed reinforced concrete element was influenced by the climate impact characteristic for the city of Vancouver. To obtain the history of changes in the corrosion current density, the following calculations were made: transport of moisture, heat and chloride ion flow in concrete cover. The flow calculations were performed for the material parameters: diffusion coefficient of chloride ions at temperature $T_{ref} = 23 \degree C$ and reference relative moisture $\varphi = 1$, $D_c = 5 \cdot 10^{-12} m^2/s$; energy for activating chloride ions diffusion $[3]$, $U = 44.6 \text{ kJ/mol}$; water-to-cement ratio, $w/c = 0.5$; concentration of chloride ions at the element boundary, $c_0 = 0.75$. The analysis of the combined transport of water, water vapour and heat was conducted on the computer with WUFI (FVM) software – using the material databases defined in this software. The chloride transport and mechanical analysis using the original software.

The courses of relative moisture and temperature changes at internal points in the concrete element were obtained from thermal and moisture calculations. The further stages involved the analysis of chloride ions transport in that element including the impact of moisture and relative humidity. Because of low variability of the function of relative moisture on the element thickness, the impact of moisture gradient on the transport of chloride ions was neglected. Also the phenomenon of chloride ions adsorption was neglected.

The concentration of chloride ions was determined on the outer surface of the analysed rebar, at points presented in Fig. 4.

The times $t_d$ for initiating the corrosion process were obtained as a result of the performed calculations. These times corresponded to the critical concentration of chloride ions equal to $0.4\%$ of concrete mass on the rebar surface. The results describing the evolution of chloride ions concentration over time are presented in Fig. 5a.

The next stage involved the analysis of the evolution of the corrosion current density function over time during the first six years after initiating the reinforcement corrosion process, cf. Fig. 5b. This function was presented in relation to the relative values of: temperature $T_{rel}$, concentration of chloride ions $c_{rel}$ and moisture $\varphi$. For the examined time interval, the relative values $T_{rel}$ and $c_{rel}$ were calculated as a ratio of a given value to apriority accepted value in accordance with the relationship

$$T_{rel} = \frac{T}{T_{max}}, \quad c_{rel} = \frac{c}{c_{max}},$$

where $T_{max}$ and $c_{max}$ are values adopted as specified in Table 2.
Table 2. Maximum values of the temperature function $T_{\text{max}}$ and the concentration of chloride ions $c_{\text{max}}$ on the rebar surface in the examined time interval

| Cover thickness [mm] | $T_{\text{max}}$ [$^\circ\text{C}$] | $c_{\text{max}}$ [%] | $t_{\text{ini}}$ [rok] | $t_{\text{d}}$ [rok] |
|----------------------|-----------------------------------|---------------------|-----------------------|---------------------|
| 27                   | 22,7                              | 0,477               | 17,6                  | 23,6                |
| 47                   | 22,9                              | 0,477               | 46,7                  | 52,7                |

The mechanical analysis of the problem was the final stage of calculations for determining the time $t_d$ of the cover degradation. The calculations were performed with the time increase $\Delta t \approx 0.5$ month. Table 3 presents the results calculated for the initiation time $t_{\text{ini}}$, the degradation time $t_d$ and the exploitation time $t_{\text{exp}}$ of the reinforced concrete element subjected to chlorides impact. The time $t_{\text{act}}$ for activating the reinforcement corrosion required to calculate the degradation time was adopted as the time, after which the corrosion current density function significantly dropped.

The value of the activation time $t_{\text{act}}$ can be adopted with some approximation on the basis of the Fig. 5b.

In the analysed example, the values of activation time $t_{\text{act}}$ for the cover of 27 and 47 mm thickness were equal to $t_{\text{act}} = 3$ and 2.5 months, respectively. Fig. 6 illustrates the simplified FEM models which were used to calculate the time $t_c$ of concrete cover cracking. Fig. 7 illustrates the maps of maximum tensile strengths in the element with the cover width $c = 27$ mm at the moment of cracking $t_c = t_d - t_{\text{act}}$ including the composition of the corrosion products $\alpha$. Fig. 8 illustrates the distribution of the distortional strains depending on the number of the active finite elements. The calculations were performed on the hypothetical assumption about the creeping process in concrete which was included by reducing the elasticity modulus to the effective value $E_{\text{c,eff}} = 0.333E_c$.

Table 3. The calculated time of imitation $t_{\text{ini}}$, degradation $t_d$, and exploitation $t_{\text{exp}}$ of the reinforced concrete element, the concept of transient layer growth, elastic and plastic material with mechanical distortions

| Cover [mm] | $t_{\text{ini}}$ | $t_d$ | $t_{\text{exp}}$ | Impact of corrosion products $\alpha$ |
|-----------|-----------------|-------|-----------------|--------------------------------------|
|           | $t_{\text{ini}}$ |       |                 |                                      |
| 27        | 210             | 9,5   | 10              | 0,523 0,573 0,622 0,523 0,573 0,622 |
| 47        | 561             | 11,5  | 12              | 571,5 572,5 574                       |
Fig. 6. The FEM calculation model for the mechanical analysis of the concrete cover degradation in the reinforced concrete element caused by the reinforcement corrosion (authors software): a) cover thickness \( c = 27 \) mm, b) cover thickness \( c = 47 \) mm.

Fig. 7. Maps of the principal tensile total strains at the moment of the apparent destruction of the concrete cover of \( 27 \) mm thickness: a) \( \alpha = 0.622 \), b) \( \alpha = 0.573 \), c) \( \alpha = 0.523 \).

7. Conclusions

The paper demonstrates the validation of the model based on the concept of the transient layer growth, which provided modelling of the concrete cover damage as the result of the corrosion reinforcement. The calculation results were validated in two stages. In the first stage, the results from computer calculations were analysed regarding: the constant value of the corrosion current density over time and the uniform reinforcement corrosion around the rebar circumference. The obtained results presented great convergence with the results of the experimental analysis. For the examined cover thicknesses of \( 27 \), \( 47 \) and \( 70 \) mm, the error percentage of the calculated time \( t_d \) of the cover degradation obtained for the averaged composition of the corrosion products in comparison to the results of the experimental tests varied between 4.1 and 28.6%. Considering the comparison of the cracking time \( t_c \) results to the experimental test results, shows the calculation error ranged from 1.2 to 11.4%. Taking into account the level of the problem complication, considerable difficulties in determining the activation time \( t_{act} \) and the adopted concrete model, the obtained results can be regarded as very satisfactory. In the second stage, the application of the transient layer growth model was presented for the non-uniform reinforcement corrosion around the rebar circumference. The calculations were aimed to estimate the increase in the exploitation time \( t_{exp} \) of the reinforced concrete element. The evolution of corrosion around the rebar circumference was assumed to be the function dependent on variable climate conditions over time and the variable concentration of chloride ions over time and around the rebar circumference. The performed calculations demonstrated that the percentage increase in the exploitation time of the structure was approximately between 2 and 5% in comparison to the traditionally understood exploitation time (equal to the initiation time of the reinforcement corrosion). This value can be regarded as an important reason which enables less rigorous requirements for concrete cover thickness to be included in the standards.
Fig. 8. Maps of maximum distortional strains in the model with cover thickness of 47 mm: a) first active element – time $t_c = 11.5$ months, b) second active element (destruction moment), – time $t_c = 29$ months.

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