# MODELING CORROSION OF STEEL REINFORCEMENT IN CONCRETE: NATURAL VS. ACCELERATED CORROSION

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**Abstract:** Chloride-induced corrosion of steel reinforcement in concrete is one of the major reasons for deterioration of reinforced concrete structures. In the present paper a recently developed coupled 3D chemo-hygro-thermo-mechanical model for concrete is used to study the difference between natural and accelerated corrosion of steel reinforcement in concrete beam-end specimen. Only active corrosion phase is simulated for natural and accelerated corrosion. The results of the analysis are compared with the available test results performed under accelerated conditions. For accelerated corrosion a good agreement between predicted and experimentally observed corrosion induced damage and pull-out resistance of corroded reinforcement bar is obtained. The comparison between natural and accelerated corrosion shows that there is a relatively large difference between distribution of current density (corrosion) over the surface of main reinforcement bar and stirrups.

### **1 INTRODUCTION**

Deterioration of reinforced concrete chloride-induced structures caused by corrosion is one of the major concerns for durability of RC structures [1][2]. Consequences of chloride-induced corrosion of steel reinforcement have negative effects on structural behavior and involve several aspects related to the life cycle of the structure, such as serviceability, safety and structural performance. The main effect of the corrosion attack is manifested as cracking and spalling of concrete cover, due to the expansion of corrosion products, which have greater volume than the steel. Direct and indirect costs of maintenance and repair are very high. Therefore, it is important to develop and numerical improve tools which can realistically simulate corrosion processes and the related mechanism of deterioration in RC structures.

Before depassivation of reinforcement a surface layer of ferric oxide covers and protects the steel in concrete. Depassivation of the protective layer can occur by reaching a critical threshold concentration of free chloride ions near the reinforcement bar surface or as a consequence of carbonation of concrete [1]. Corrosion of steel in concrete is an electrochemical process dependent on the electrical conductivity of the concrete and steel surfaces and the concentration of dissolved oxygen in the pore water near the reinforcement [3] [4]. The recently developed 3D chemo-hygro-thermo-mechanical (CHTM) model, which is employed in the present numerical simulations, couples the above mentioned physical and electrochemical

behavior processes with mechanical of concrete (damage). The model can simulate processes before and after depassivation of steel reinforcement in concrete under natural or accelerated corrosion [5-10]. In order to predict the rate of rust production and related effects, it is necessary to simulate the following physical, electrochemical and mechanical processes: (1) Transport of capillary water, oxygen and chloride through the concrete cover; (2) Immobilization of chloride in concrete; (3) Drying and wetting of concrete as well as related hysteretic property of concrete; (4) Transport of OH- ions through electrolyte in concrete pores; (5) Cathodic and anodic polarization; (6) Transport of corrosion products in concrete and cracks; (7) Creep and shrinkage of concrete and (8) Damage of concrete due to mechanical and nonmechanical actions.

In general case (natural corrosion), to compute the corrosion rate and the related induced damage it is necessary to assume position of anode and cathode on the reinforcement surface. The influence and the effect of their distribution is a complex aspect of the modeling of corrosion which is still not solved [11]. For accelerated corrosion, which is frequently used in experimental tests, the position of cathodic and anodic areas are clearly defined. However, there are relatively large differences between natural and accelerated corrosion, e.g. corrosion products with different expansion factors are generated, distribution of corrosion products over the reinforcement surface can be significantly different, duration of active corrosion phase is usually significantly different and other. Consequently, to define realistic design rules based on the experimental tests it is useful to understand the differences between natural and accelerated corrosion and their consequence on damage of reinforced concrete.

In the paper recently performed experimental pull-out tests of corroded reinforcement bars from a concrete beam-end specimen are simulated. In the tests steel reinforcement bars and stirrups were corroded under accelerated conditions. To verify the model, numerical results are first compared with experimental results. Subsequently, the simulation of the same specimen is performed assuming chloride induced natural corrosion under splash environmental conditions and the results are compared with that obtained for accelerated corrosion. The simulations are performed only for the active corrosion phase. The first part of the paper gives a short overview of the 3D CHTM model. Discussed are only processes that are relevant for the active corrosion phase. In the second part the results of simulations are presented, compared and discussed.

## **2** GOVERING EQUATIONS

Transport of capillary water is described in terms of volume fraction of pore water in concrete by Richard's equation [6-8], based on the assumption that transport processes take place in aged concrete:

$$\frac{\partial \theta_{w}}{\partial t} = \nabla \cdot \left[ D_{w}(\theta_{w}) \nabla \theta_{w} \right]$$
(1)

where  $\theta_w$  is volume fraction of pore water (m<sup>3</sup> of water / m<sup>3</sup> of concrete) and  $D_w(\theta_w)$  is capillary water diffusion coefficient (m<sup>2</sup>/s) defined as a strongly non-linear function of moisture content [12]:

$$D_{w}(\theta_{w}) = D_{0}e^{n\bar{\theta}_{w}}$$
(2)

where  $D_0 = 2.2 \times 10^{-2} \text{ m}^2/\text{s}$  is limiting magnitude term,  $\bar{\theta}_w = (\theta_w - \theta_{wi})/(\theta_{wd} - \theta_{wi})$  is reduced water content,  $\theta_{wi}$  and  $\theta_{wd}$  are initial and saturated water content, respectively, and n = 6.4 is shape term.

Transport of oxygen through the concrete cover is considered as a diffusion problem only. It is defined by the following equation:

$$\frac{\partial C_o}{\partial t} = D_o(S_w, p_{con}) \nabla^2 C_o$$
(3)

where  $C_o$  is oxygen concentration in pore solution (kg of oxygen/m<sup>3</sup> of pore solution),  $D_o(S_w, p_{con})$  is effective oxygen diffusion coefficient (m<sup>2</sup>/s),  $S_w$  is degree of water saturation of concrete, which has great influence on oxygen diffusivity and  $p_{con}$  is concrete porosity. The convective part is neglected due to the assumption that water saturation of concrete is uniform [5].

The oxygen consumed at cathode surface is supplied from concrete by oxygen diffusion. The oxygen diffusion flux at the cathode surface,  $j_o^c$  (kg/m<sup>2</sup> s) can be calculated as:

$$j_o^c = D_o(S_w, p_{con}) \frac{\partial C_o}{\partial n} \bigg|_{cathode}$$
(4)

where n is the outward normal to the cathode surface.

The oxygen consumption at the cathodic and anodic surfaces can be calculated as:

$$D_{o}(S_{w}, p_{con}) \frac{\partial C_{o}}{\partial n} \bigg|_{cathode} = -k_{c}i_{c}$$

$$k_{c} = 8.29 \times 10^{-8} \frac{\text{kg}}{\text{C}}$$
(5a)

$$\begin{aligned} D_o(S_w, p_{con}) \frac{\partial C_o}{\partial n} \bigg|_{anode} &= -k_a i_a \\ k_a &= 4.14 \times 10^{-8} \frac{\text{kg}}{\text{C}} \end{aligned} \tag{5b}$$

where  $i_c$  and  $i_a$  are cathodic and anodic current density (A/m<sup>2</sup>), respectively. The constants  $k_c$ and  $k_a$  are calculated using the stoichiometry of chemical reactions and Faraday's law.

The electric current through the electrolyte is a result of motion of charged particles and, if the electrical neutrality of the system and the uniform ions concentration are assumed, can be written as:

$$\mathbf{i} = -\sigma \nabla \Phi \tag{6}$$

where i is electric current density vector,  $\sigma$  is electrical conductivity of concrete and  $\Phi$  is electric potential. If electrical neutrality is accounted for and electrical conductivity of concrete is assumed as uniformly distributed, the equation of electrical charge conservation reads [6]:

$$\nabla^2 \Phi = 0 \tag{7}$$

For a given boundary conditions,  $\mathbf{n} \cdot \mathbf{i} = 0$ on all surfaces of the concrete body, the solution of Eq. (5) determines the electric potential distribution, and using Eq. (4) the corrosion current density can be calculated. According to Butler–Volmer kinetics, in the present model kinetics of reaction at the cathodic and anodic surface can be estimated from:

$$\dot{i}_{c} = \dot{i}_{0c} \frac{C_{o}}{C_{ob}} e^{-2.3\eta_{c}/\beta_{c}} \qquad \dot{i}_{a} = \dot{i}_{0a} e^{2.3\eta_{a}/\beta_{a}}$$
(8)

where  $C_{ob}$  is oxygen concentration at surface of concrete element exposed to seawater (kg/m<sup>3</sup>),  $\eta_a$  and  $\eta_c$  are the anodic and cathodic overpotential, respectively,  $i_{0c}$  and  $i_{0a}$  are the exchange current density of the cathodic and anodic reaction (A/m<sup>2</sup>),  $\beta_c$  and  $\beta_a$  are the Tafel slope for cathodic and anodic reaction (V/dec), respectively.

From experiments [13] it is known that in the case of chloride type of corrosion the part of corrosion products penetrate into the pore of concrete around the reinforcement bar and relatively large amount of rust can be transported through radial cracks that are generated because of expansion of corrosion products. This transport is very much dependent water saturation [14]. on Mathematically speaking distribution of corrosion product (red rust) R (kg/m<sup>3</sup> of pore solution) into the pores of concrete and in the cracks is modeled as convective diffusion problem:

$$\theta_{w} \frac{\partial R}{\partial t} = \nabla \cdot \left[ \theta_{w} D_{r} \nabla R \right] + D_{w} (\theta_{w}) \nabla \theta_{w} \nabla R \tag{9}$$

in which  $D_r$  is diffusion coefficient (m<sup>2</sup>/s) of corrosion product. Note that Eq. (9) does not describe transport of rust, however, it describes distribution of rust which is produced in concrete (pores and cracks) as a consequence of the reaction of soluble species (that can dissolve in the concrete pore solution and subsequently migrate or diffuse through pores and cracks of concrete) with oxygen in pore water. For more detail see [14].

### **3 NUMERICAL IMPLEMENTATION**

By using the finite element method to solve the partial differential equations of the mathematical model, the strong form has to be rewritten into a weak form. The weak form of the system of partial differential equations is

carried out by employing the Galerkin weighted residual method. This is then implemented into a non-commercial 3D finite element code MASA, developed at the Stuttgart (Institute University of of Construction Materials). The non-mechanical part of the problem is solved by using a direct integration method of implicit type. To solve the mechanical part, Newton-Raphson iterative scheme is used. As the constitutive law for concrete the microplane model based on the relaxed kinematic constraint is used [15]. To dependency, avoid mesh size as а regularization method the crack band approach is employed [16]. In the finite element analysis, the band corresponds to a row of finite elements. As already mentioned, the coupling between the mechanical and nonmechanical parts of the model is performed by updating continuous of the governing parameters during the incremental transient finite element analysis using a staggered solution scheme. For more detail see [5-7].

One-dimensional corrosion contact elements are employed to account for the inelastic strains due to the expansion of corrosion products. They are placed radially around the reinforcement bar surface and their main function is to simulate the contact between reinforcement and the surrounding concrete. These contact elements can take up only shear forces, in the direction parallel to the reinforcement axis, and compressive forces perpendicular to the adjacent surface of the reinforcement. The inelastic radial expansion due to corrosion,  $\Delta l_r$ , is calculated as:

$$\Delta l_r = \frac{m_r}{A_r} \left( \frac{1}{\rho_r} - \frac{r}{\rho_s} \right)$$
(10)

where  $\rho_r$  and  $\rho_s$  are densities of rust and steel, respectively, *r* is the ratio between the mass of steel ( $m_s$ ) and the corresponding mass of rust ( $m_r$ ) over the related surface of the reinforcement  $A_r$  that corresponds to the contact element. Note that the ratio between densities of steel and rust depends on the type of the corrosion products and can vary from 2 to 7 [4][14].

# 4 NUMERICAL STUDY

The above presented 3D CHTM model is here employed to study the difference between accelerated and natural corrosion of steel reinforcement of a beam-end specimen. As a reference, the experimental results performed by Lin et al. [17] under accelerated corrosion are used.

# **4.1** Geometry, material properties and loading

The typical geometry of the tested specimen is shown in Fig. 1 [17]. To study the difference between natural and accelerated chloride induced corrosion. numerical simulation is performed for both cases. The specimen cross-section is 150 x 250 mm, the diameter of the main reinforcement bar is d = 20 mm and concrete cover c = 25 mm. The total embedment length of the reinforcement is 200 mm, whereas the rest of the length is isolated with a plastic sleeve (Fig. 1). Along the embedment length there are two stirrups with d = 8 mm and spacing s = 100 mm (test specimen LA2S, [17]).



Figure 1: Geometry of the test specimens (LA4S [17]).

Table 1: Mechanical properties

Parameter	Value
Modulus of el. of concrete, $E_c$ (GPa)	30.0
Poisson's ratio of concrete, $v_c$	0.18
Tensile strength of concrete, $f_t$ (MPa)	3.85
Uniaxial comp. str. of concrete, $f_c$ (MPa)	45.7
Fracture energy of concrete, $G_F$ (J/m <sup>2</sup> )	150.0
Modulus of elasticity of steel, $E_s$ (GPa)	210.0
Poisson's ratio of steel, $v_s$	0.33

The finite element discretization of the specimen is shown on Fig. 2. Concrete is discretized with four node 3D solid finite elements and reinforcement with eight node 3D solid elements. The mechanical boundary conditions for the pull-out test are the same as in the experimental tests (see Fig. 1). The mechanical properties of concrete and steel are summarized in Tab. 1.



Figure 2: 3D finite element discretization.



reinforcement.

To simulate the expansion due to the corrosion products, formation of onedimensional (1D) radially oriented corrosion contact truss finite elements over the surface of reinforcement are used with a length of 0.10 mm. They can take up radial forces (only compressive, no tensile) and shear forces in direction of the reinforcement axis. The radial compressive resistance is assumed to be linear elastic and the shear resistance is controlled by the bond-slip relationship. The bond-slip constitutive law of contact elements (not corroded) is shown in Fig. 3, where  $\tau_{max}$  is the total bond strength and  $\tau_f$  is the frictional

strength. The bond strength and the shape of the curve is similar as observed in the experiments [17]. Note that in the numerical simulation the bond-slip relationship was assumed as independent of the corrosion level. As will be shown later, this assumption is realistic since, because of small concrete cover, the failure of corroded reinforcement was due to the splitting of concrete cover and not to the pull-out.

**Table 1**. The dependence of the oxygen diffusivity and electrical conductivity on water saturation for good quality concrete (water-cement ratio, w/c = 0.4).

Saturation (%)	50	60	70	80	90
Oxygen diff. $D_o (10^{-8} \text{ m}^2/\text{s})$	1.55	1.20	0.75	0.30	0.13
Electrical con. $\sigma (10^{-3} \Omega^{-1} m^{-1})$	0.20	0.53	1.0	6.0	11.2

 Table 2. Parameters used in the numerical simulations.

Parameter	Value
Faraday's constant, F (C/mol)	96486.7
Anodic exchange current den., $i_{0a}$ (A/m <sup>2</sup> )	$1.875 \times 10^{-4}$
Cathodic exchange current den., $i_{0c}$ (A/m <sup>2</sup> )	6.25×10 <sup>-6</sup>
Anodic equilibrium pot., $\Phi_{0a}$ (V vs. SCE)	-0.780
Cath. equilibrium pot., $\Phi_{0c}$ (V vs. SCE)	0.160
Tafel slope for anodic reaction, $\beta_a$ (V/dec)	0.06
Tafel slope for cath. reaction, $\beta_c$ (V/dec)	0.160

Numerical simulation for natural corrosion is performed only for the active corrosion phase, after depassivation i.e. of reinforcement. It is assumed that the saturation is constant over the entire volume of the specimen (S = 50%). The initial concentration of oxygen in concrete is taken the same as the concentration on the exposed concrete surfaces and is equal to 0.0085 kg of dissolved oxygen  $/ m^3$ solution. of pore Oxygen diffusivity and electrical conductivity of concrete are chosen for a good quality concrete with water-cement ratio, w/c = 0.4(Table 1). The parameters relevant for the calculation of electric potential and consumption of oxygen are summarized in Tables 1 and 2. They approximately correspond to parameters for severe splash zone environmental conditions [1][2]. In addition, for the transport of corrosion products through cracks, the rust diffusivity is set to  $D_r = 2.2 \times 10^{-16} \text{ m}^2/\text{s}.$ 



Figure 4: Accelerated corrosion test set-up [17].

In experimental tests accelerated corrosion technique was used to accelerate steel bar corrosion in concrete. Power supplies with adjustable voltage and current were used to impress constant current on the specimens (see Fig. 4). The steel bars served as the anode and the outside stainless steel net was fixed on the surface to serve as cathodes. Sponge was placed between the stainless steel net and the concrete surface to provide an adequate contact and it was every day sprayed with a 5% chloride solution, therefore, saturation was not constant. Initially it was set to 50% with the surface saturation of 85%.



**Figure 5**: Natural corrosion: position of anodic (green) and cathodic (red) areas on the surface of reinforcement.

Accelerated corrosion was carried out under relatively constant current density of  $400 \sim 600 \mu$ A/cm<sup>2</sup>. Time necessary for obtaining the

target corrosion was evaluated based on the Faraday's law. Note that the current density was very high, approximately 500 times larger than observed for natural corrosion under extreme conditions.



Figure 6: Accelerated corrosion: position of anodic and cathodic areas on the surface of reinforcement.

As mentioned above, to simulate natural corrosion it is necessary to define anodic and cathodic areas on the reinforcement surface in advance. For the present case study these surfaces are defined as shown in Fig. 5. However, for accelerated corrosion boundary conditions are known, i.e. the surface of concrete specimen is cathode and the reinforcement bars are anodes (Fig. 6).

In the simulation of accelerated corrosion electric potential is imposed on the surface of concrete that is in contact with reinforcement bars. The potential was chosen such that the resulting current density was approximately 500  $\mu$ A/cm<sup>2</sup>. To calculate corrosion induced damage of concrete it was also necessary to calculate distribution of water since in the experiments the surface of the specimen was constantly sprayed with water. Furthermore, the electric potential and the transport of corrosion products through pores and cracks of concrete are also computed. For natural corrosion it is computed the same as for the accelerated, however, instead of distribution of water, distribution of oxygen is needed in order to be able to calculate current density at anodic and cathodic surfaces (Eqs. 3-8). The volume expansion factor of rust is assumed to be  $\alpha_r = \rho_s / \rho_r = 4.0$  and 1.90 for natural and accelerated corrosion, respectively.

### 4.2 Results: accelerated corrosion

Numerically predicted and experimentally observed typical crack patterns at the surface of the specimen after 13 days of accelerated corrosion are shown in Fig. 7. The red zones in the numerical crack pattern represent the crack width of 0.10 mm or larger. As can be seen, there are one crack parallel to the main reinforcement and two cracks along the stirrups.



**Figure 7**: Experimental and numerical crack pattern after 13 days of accelerated corrosion (specimen LAS2 [17]).

Figure 8 shows distribution of current density after 4 and 8 days of accelerated corrosion along the top surface of main reinforcement bars and along the stirrups, respectively. The current density on the main reinforcement bar is the highest between the stirrups and relatively low below the stirrups, i.e. it looks like the stirrups "protects" main reinforcement. Similar is also observed in the experiments (see top of Fig. 8). However, current density along the stirrups is the highest at the edges and it is approximately 3.5 times higher than that on the main reinforcement bar. This is reasonable since stirrups are closer to the surface of the specimen (cathode). The experimental results [17] confirm these results.

The distribution of current density over the cross-section of the main reinforcement is shown in Fig. 9. As expected, current density on the top surface of the main reinforcement bar is much higher than that at the bottom surface.

Figure 10 shows distribution of corrosion induced radial pressure in the mid-section of

the main reinforcement bar. Although the current density at the top surface is much higher than that at the left and bottom sides, maximum radial pressure is oriented in the lateral direction since concrete cover along the main reinforcement concrete is cracked.



Figure 8: Distribution of current density over the top surface of main reinforcement (a) and stirrups (b).



**Figure 9**: Distribution of current density (A/m<sup>2</sup>) over the mid cross-section of main reinforcement bar.



Figure 10: Distribution of radial pressure (MPa) over the mid cross-section of main reinforcement bar.

 
 Table 3: Summary of measured and predicted pullout loads.

Duration (days)	P <sub>max,e</sub> (kN)	P <sub>max,s</sub> (kN)
0	146.17	141.12
4	132.87	135.53
13	89.58	94.37
16	90.42	77.96



Figure 11: Crack pattern after 16 days of accelerated corrosion and subsequent rebar pull-out.

To investigate the influence of corrosion on the bond resistance, for different duration of accelerated corrosion the corroded reinforcement bar was pulled-out from the specimen. Except for the reference test (no corrosion), were failure was due to the combined failure (vielding of reinforcement splitting), all corroded specimens failed due to splitting of concrete cover. This was also the case in the simulation. Figure 11 shows typical crack pattern, which corresponds to 16 days of corrosion and subsequently pull-out of rebar. In Tab. 3 are summarized experimentally measured and numerically predicted pull-out loads. Having on mind the complexity of the problem the agreement between experimental tests and numerical results is good. For the average corrosion level of 8.84%, measured in the experiment [17], there is the reduction of pull-out capacity for approximately 40%.

#### 4.3 Results: natural corrosion

accelerated corrosion Since rate is approximately 500 times higher than natural corrosion rate, results for the same duration time of natural corrosion cannot be directly compared with the duration time of accelerated corrosion. Moreover, generated corrosion products are rather different, e.g. expansion factor in natural corrosion (red rust) is much larger than in the accelerated corrosion (e.g. goethite). Therefore, here are results for natural corrosion after 7 years compared with accelerated corrosion after 4 and 8 days since the corrosion induced crack widths at the surface of concrete specimen are for these three cases in the similar range.

Figure 12 shows surface cracks (red = crack width of 0.10 mm or larger) for natural corrosion after 7 years (a) and for accelerated corrosion after 4 and 8 days, respectively. It can be seen that natural corrosion after 7 years caused corrosion induced damage that is between cases (b) and (c).



Figure 12: Crack pattern after 7 years of natural corrosion (a), 4 days (b) and 8 days (c) of accelerated corrosion.

Distribution of current density over the surface of main reinforcement bar and stirrups is shown in Fig. 13. It is quite different than that for accelerated corrosion (see Fig. 8). The largest values of current density are located at the anodic surfaces of the main reinforcement and they are approximately 20 times smaller. Furthermore, in the contrary to the accelerated corrosion, maximum current density over the stirrups surface is almost the same as on the

main rebar. Similar was observed for splices under accelerated corrosion [18]. There are two main reasons for these differences: (i) In natural corrosion there are micro-cells and current is running between anodic and cathodic surfaces. However, in the case of accelerated corrosion current is running between the concrete surface (cathode) and reinforcement (anode) and a part of the main reinforcement is in the "shadow" of stirrups. Moreover, surfaces of reinforcement bars oriented into direction of the top concrete surface are directly exposed to the current whereas the bottom surfaces are in "shadow"; (ii) In the natural corrosion saturation was assumed to be constant and in the accelerated corrosion concrete surface was sprayed with water. Therefore the stirrups were in the zone of higher saturation than the main reinforcement bar. Consequently, current density on stirrups is larger.



Figure 13: Distribution of current density over the top surface of main reinforcement (a) and stirrups (b).

Distribution of current density over the anodic section of main rebar is shown in Fig. 14. It can be seen that the highest current density is located at the top surface of main rebar that coincide with the vertical corrosion induced crack of concrete cover. Due to the crack the supply of oxygen is good and consequently the corrosion rate high.



**Figure 14**: Distribution of current density (A/m<sup>2</sup>) over the anodic cross-section of main reinforcement bar.



**Figure 15**: Distribution of radial pressure (MPa) over the anodic cross-section of main reinforcement bar.

Figure 15 shows distribution of radial corrosion induced pressure over the anodic surface of main reinforcement bar. The pressure in direction of free surface is relatively low due to the splitting crack. The highest pressure is approximately 60 MPa that is much smaller than that obtained for the accelerated corrosion (approximately 100 MPa).

The pull-out capacity of corroded main reinforcement after 7 years of corrosion was reduced from 141.12 kN to 132.86 kN, which is approximately in the range of the pull-out resistance of accelerated corrosion between 4 and 8 days. The typical crack pattern after pull-out of corroded main reinforcement bar is shown in Fig. 16. Similar as for accelerated corrosion, failure mode is due to splitting of concrete cover.



Figure 16: Crack pattern after 7 years of natural corrosion and subsequent rebar pull-out.

### **5** CONCLUSIONS

Based on the presented results of numerical of simulations accelerated and natural corrosion of the beam-end specimen, the following can be concluded. The results of numerical simulation for accelerated corrosion show good agreement with the experimental results. The corrosion level of stirrups is much higher than that of main reinforcement bar for which the corrosion level is the highest in the mid-section. Comparing corrosion distribution of natural and accelerated corrosion, for the approximately the same corrosion induced damage of concrete, it can be noted that the distribution of corrosion level is different than in the case of accelerated corrosion. Moreover, for natural corrosion current density of stirrups and main reinforcement is approximately the same, however, it is much smaller than that observed for accelerated corrosion. In spite of this, the corrosion induced damage is for both cases similar. This can be attributed to different expansion factors of corrosion products and different distribution of current densities. It is shown that with increase of corrosion level pull-out capacity of rebar decreases. Finally, it has to be noted that the results of accelerated corrosion should be carefully evaluated before their use in formulation of design rules.

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