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DURABILITY ASSESSMENT OF REINFORCED CONCRETE STRUCTURES ASSISTED BY NUMERICAL SIMULATION

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Abstract

Carbonation and chloride ingress in concrete are main cause for steel corrosion in reinforced concrete structures. The service life of reinforced concrete structures can be divided into two main phases; the initial and the propagation ones. The initiation phase when carbonation or chlorides propagates through the reinforcement cover was investigated in previous papers by the authors and the results show high influence of cracks on accelerating the carbonation and chloride ingress in concrete structures. The main focus of this paper is the efficient modelling of the propagation period and prediction of the radial corrosion depth x_{corr}, including cracking and spalling of concrete cover. The models were implemented in ATENA software and their application to bridge structures is presented. The simulation shows reinforcement corrosion due to carbonation and chloride ingress, and its impact on a structural behaviour during ULS analysis.

1. INTRODUCTION

Reinforcement corrosion due to carbonation and chloride ingress are the most damaging mechanisms in reinforced concrete structures. The service life t_l of reinforced concrete structures is generally divided into two time phases; the initiation (induction) period t_i and the propagation period t_p. The initiation period for damaging mechanisms was described and validated in the previous paper [1] and preliminary results show strong influence of cracks to transport properties and acceleration of damaging mechanisms. Cracks 0.3 mm decrease induction time approximately 6 times for carbonation and approximately 9 times for chloride ingress from sea water. Preventing macro-cracks and designing proper concrete is essential for durable concrete [1].

Our model focuses on the propagation period t_p where corrosion of reinforcing steel takes place. During this period, reinforcement decreases and is accompanied with growing corrosion products. Corrosion of reinforcement is described according to the general diagram in Figure 1. A uniform corrosion (the most widespread form of corrosion) is characteristic for carbonation and a pitting corrosion (creation of small pits) for chlorides [2]. The cracking of concrete cover during propagation period $t_{p,cr}$ corresponds to the depth of corrosion $x_{corr,cr}$ and spalling of concrete cover $t_{p,sp}$ corresponds to the depth of penetration $x_{corr,sp}$ [3].

The degradation and reinforcement corrosion models are combined with a nonlinear mechanical model. The performance of the presented chemo-mechanical model is validated on several engineering structures suffering from chloride ingress, e.g. Nougawa bridge, Japan and a pre-stressed bridge in Prague, Czech Republic. The model is capable of predicting durability due to chloride ingress with regards to intrinsic/extrinsic factors and it opens a path for performance-based design.

2. CORROSION INITIATION PHASE

The initiation period covers the time before the concentration of chlorides exceeds a critical value in the place of reinforcement. Chloride 1D transient ingress into concrete, with an initially zero chloride content, reads [3]:

$$C(x,t) = C_S\left[1 - erf\left(\frac{x}{2\sqrt{D_m(t)f(w)t}}\right)\right] \tag{1}$$

where C_s is the chloride content at surface in [kg/m^3], $D_m(t)$ is the mean (averaged) diffusion coefficient at time t [m^2/s], x is the distance from the surface in [m] and $f(w)$ introduces acceleration by cracking. (equals to one for a crack-free concrete). C_s and $C(x,t)$ can be related to a concrete volume or to a binder mass. The model is in detail described in the previous paper [1].

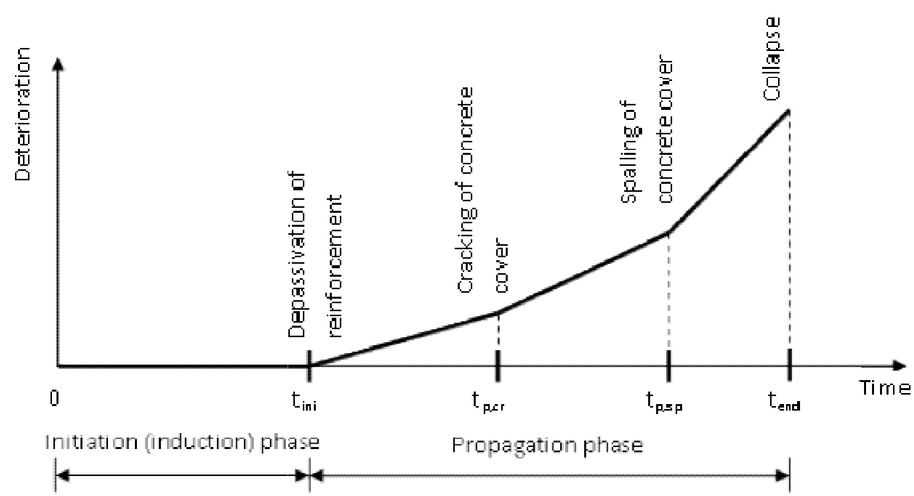

Figure 1: Corrosion phases

3. CORROSION PROPAGATION PHASE

The propagation phase is controlled by the corrosion rate, which for chlorides depends on the corrosion current density i_{corr} [μA/cm^2] and on chlorides concentration in the concrete. The proposed model predicts amount of corroded steel during the propagation period t_p. The corrosion rate is governed by Faraday's law [6] and is given by the following formula:

$$\dot{x}_{corr}(t) = 0.0116 i_{corr}(t) \tag{2}$$

where $\dot{x}_{corr}$ is the average corrosion rate in the radial direction [μm/year], i_{corr} is corrosion current density [μA/cm^2] and t is calculated time after the end of induction period [years]. By integration of Eq. (2), we obtain the corroded depth for 1D propagation:

$$x_{corr}(t) = \int_{t_{ini}}^{t} 0.0116 i_{corr}(t) R_{corr} dt \tag{3}$$

where x_{corr} is the total amount of corroded steel in radial direction [mm] and R_{corr} is parameter, which depends on the type of corrosion [-]. For uniform corrosion (carbonation) $R_{corr} = 1$, for pitting corrosion (chlorides) $R_{corr} = <2; 4>$ according to [5] or $R_{corr} = <4; 5.5>$ according to [9]. The effective bar diameter for both types of corrosion is obtained from:

$$d(t) = d_{ini} - \psi 2x_{corr}(t) \tag{4}$$

where $d(t)$ is the evolution of a bar diameter in time t, d_{ini} is initial bar diameter [mm], ψ is uncertainty factor of the model [-], mean value $\psi = 1$ and x_{corr} is the total amount of corroded steel according to (4). The corrosion rate for chlorides is affected by concentration of chlorides in the concrete. The calculation of the corrosion current density was formulated by Liu and Weyer's model [7]:

$$i_{corr} = 0.926 * exp\left[7.98 + 0.7771 ln(1.69C_t) - \frac{3006}{T} - 0.000116R_C + 2.24t^{-0.215}\right] \tag{5}$$

where i_{corr} is the corrosion current density [μA/cm^2], C_t is the total chloride content [kg/m^3 of concrete] at the reinforcement location, which is determined from 1D non-stationary transport, T is temperature at the depth of reinforcement [K], R_c is the ohmic resistance of the concrete cover [Ω] [10] and t is the time after the initiation [years] with

$$R_C = exp[8.03 - 0.549 ln(1 + 1.69C_t)] \tag{6}$$

4. CONCRETE COVER CRACKING

In the next phase the increase of reinforcement volume due to corrosion is causing cracking of concrete cover, and for chlorides it can be estimated from DuraCrete model, which provides realistic results [11]. The critical penetration depth of corroded steel $x_{corr,cr}$ is formulated as:

$$x_{corr,cr} = a_1 + a_2 \frac{C}{d_{ini}} + a_3 f_{t,ch} \tag{7}$$

where parameter a_1 is equal to 7.44e-5 [m], parameter a_2 is 7.30e-6 [m], a_3 is -1.74e-5 [m/MPa], C is the cover thickness of concrete [m], d_{ini} initial bar diameter [m], and $f_{t,ch}$ is the characteristic splitting tensile strength of concrete [MPa].

5. CONCRETE COVER SPALLING

In the subsequent stage the expansion due to corrosion induces the spalling of concrete cover. The critical penetration depth of corroded steel $x_{corr,sp}$ for chlorides is calculated from [11] as:

$$x_{corr,sp} = \frac{w^d - w_0}{b} + x_{corr,cr} \tag{7}$$

where parameter b depends on the position of the bar (for top reinforcement 8.6 μm/μm and bottom position 10.4 μm/μm), w_d is critical crack width for spalling (characteristic value 1 mm), w_0 is the width of initial crack (from mechanical analysis).

After spalling of concrete cover, corrosion of reinforcement takes place in direct contact with the environment. The rate of corrosion of reinforcement after spalling is controlled by the aggressivity of environment [8].

6. NONLINEAR FINITE ELEMENT ANALYSIS

The above models are implemented in ATENA software [12], using multi-physics approach for mechanics and transport. It predicts induction time and extent of corrosion for chloride ingress, and calculates remaining steel area. The mechanical behavior and concrete cracking is simulated using the fracture-plastic model [13]. It combines plasticity based model for compresive failure and smeared crack model with tensile softening and crack band approach for tension (Figure 2). The reinforcement corrosion is evaluated based on the parameters of the surrounding environment that are specified as a special boundary condition as shown in Figure 3. Figure 3 shows a simple example of a short cantilever whose bottom surface is subjected to chlorides. A mechanical load initiated cracks starting at the bottom surface. For each reinforcement, the closest distance to the surface subjected to chlorides is calculated. The initiation phase as well as all the other corrosion phases as described previously are evaluated assuming a 1D transport process along this closest distance considering also the width of the surface crack. Based on the amount of corrosion the effective reinforcement area is reduced, which can directly effect the load carrying capacity or the deflections of the numerical model. This approach can simulate the effect of structural degradation in a very effective and efficient way.

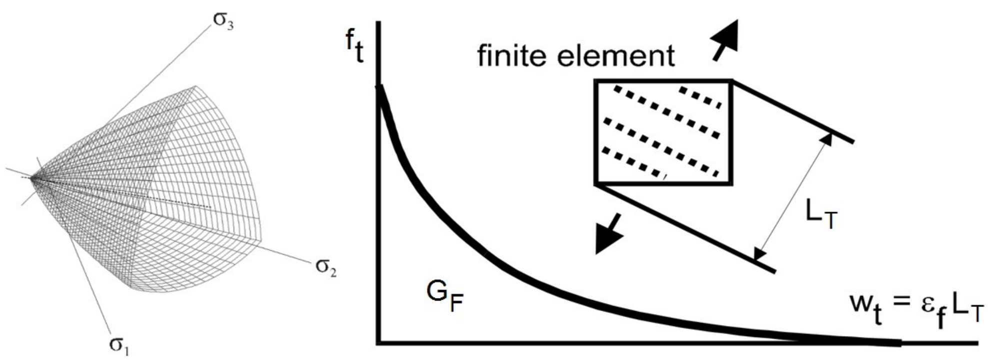

Figure 2: (left) three-parameter Menetrey and Willam failure criterion [14] for compression, (right) crack band and tensile softening model for tension [13]

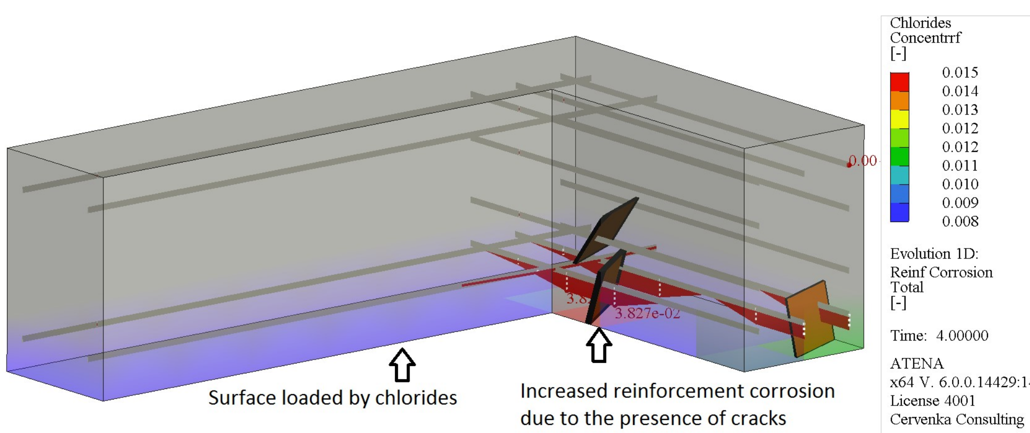

Figure 3: Corrosion modelling in finite element nonlinear analysis

7. VALIDATION

The Nougawa bridge was built in 1930 in a Japanese coastal area. The bridge is three span reinforced concrete structure with the total length of 131 m. Due to high chloride presence in the air and fast corrosion after 30 years from its erection the bridge had to be repaired by mortar. In 2006 (80 years after the construction), two beams from the bridge were cut out and further investigated [15] (Figure 4). In the mid-spans, the concentration of chlorides and the loss of reinforcement were evaluated. Carbonation depth was almost zero everywhere. Due to that fact, the assessment of chloride ingress was performed only.

Figure 4: The validation beam from Nougawa bridge [15]

The longitudinal reinforcement of beams had diameter 25.4 mm, stirrups 9.5 mm and the concrete cover was 47 mm.

First, the bridge was mechanically loaded by the self-weight and the corresponding design life load. The following material parameters of the beams were assumed: the compressive strength 26 MPa and Young's modulus 25 GPa. Cracks up to 0.3 mm emerged due to loading. Figure 6 shows the calculated deflection and the reinforcement arrangement.

Second, the surface of the beams was exposed to chlorides with the following parameters: D_{ref} = 1.2e-7 m^2/day, t_{Dref} = 3650 days, m_{coeff} = 0.37, t_{mcoeff} = 10950 days, C_s = 0.014 kg/kg on ocean side (beams 8,9) and C_s = 0.011 kg/kg on bottom surface (beams 2,5), Cl_{crit} = 0.004 kg/kg, a_1 = 7.44e-5 m, a_2 = 7.30e-6 m, a_3= -1.74e-5 m/MPa, $f_{t,ch}$ = 3.5 MPa, w_d = 0.001 m, pitting corrosion R_{corr} = 3, corrosion rate after spalling 30 μm/year. Those parameters were mostly estimated from DuraCrete model [4]. The evolution of chloride concentration at longitudinal reinforcement during 80 years is shown in Figure 7. Figure 5 shows evolution of chloride concentration at point P2 after 80 years of service. The experimental data are available only after 80 years when the experiment was performed. Therefore the comparison with numerical results is available only at this time, which is represented as a red point in the figures below.

Figure 8 shows the evolution of reinforcement area including patching of concrete cover. Predicted reinforcement area of 64% agrees well with the measured value of 62.5%.

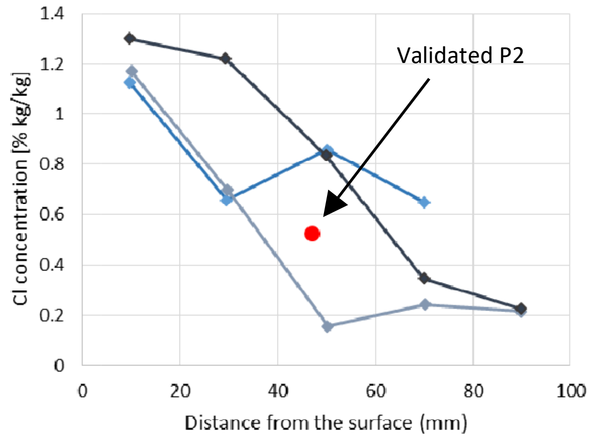

Figure 5: Evolution of chloride concentrations for beams 2, 5, 9 in 80 years

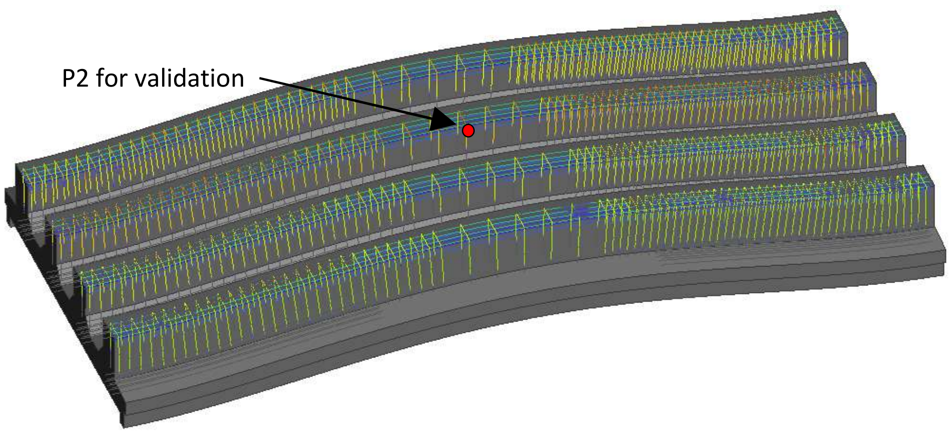

Figure 6: Deformed shape of the bridge section with the reinforcement model, bottom view of the model

8. APPLICATION TO PAVEL WONKA BRIDGE

The presented chemo-mechanical model can be applied for a wide range of structures from civil engineering. This is documented on an assessment of a prestressed box-girder concrete bridge of Mr. Pavel Wonka over the river Elbe in Pardubice, Czech Republic. The bridge is subjected to a monitoring program due to its importance for the city transportation system. It is beyond the scope of this paper to present detailed results from this analysis and hence, only the most important global results and results related to the presented durability analysis of the bridge are given. More details are available in [16], including material parameters, description of the bridge geometry and pre-stressing tendons etc.

The bridge was designed and erected between 1956 and 1959. The structure is depicted in Figure 9. The bridge consists of three arches, having spans 50 + 70 + 50 m. Average depth of cross sections is up to 3.5 m.

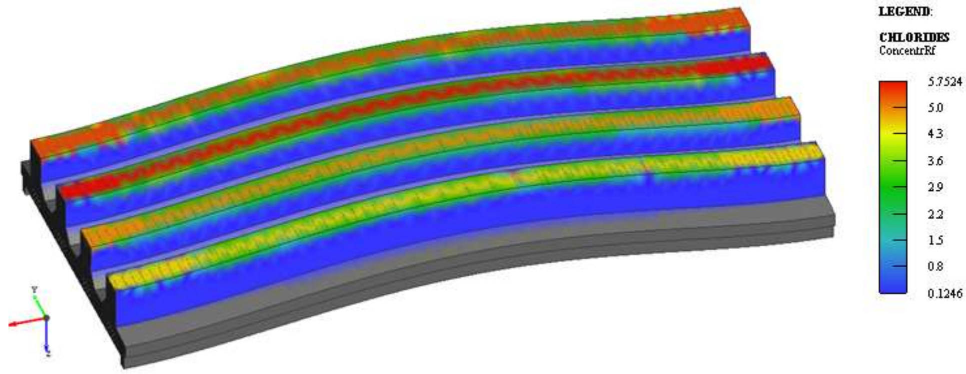

Figure 7: Concentration of chlorides (% kg/kg) in the place of reinforcements after 80 years at depth 60 mm

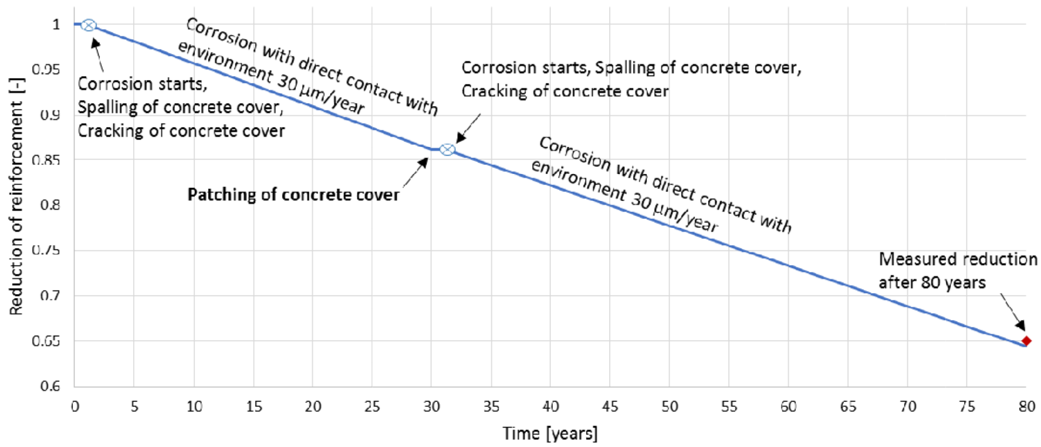

Figure 8: Nougawa bridge, the evolution of reinforcement loss at point P2 in 80 years

The structure was analyzed by ATENA with implemented durability models. The bridge is modelled by 4512 layered shell elements. The structure near supports and some other details are modelled by hexahedral and wedge solid elements. The pre-stressed tendons are realized by 3022 external cable truss elements, while the conventional reinforcement is introduced by embedded reinforcement within shell elements (see Figure 10).

The numerical study involved several analyses. First the model was calibrated using the known results from the bridge load test performed after the bridge rehabilitation in 2006. In the subsequent analysis the bridge is loaded by the permanent load and then it is subjected to the environmental actions: **carbonation**: C_p = 350 kg/m^3, SCM = 0, W = 175 kg/m^3, CO_2 = 0.00036, RH = 0.60. Progressive period a_1 = 7.44e-5 m, a_2 = 7.30e-6 m, a_3 = -1.74e-5 m/MPa, $f_{t,ch}$ = 3.5 MPa, d_{ini} = 0.001 m, pitting corrosion R_{corr} =1, corrosion rate after spalling 30 µm/year, **chlorides**: D_{ref} = 1.19e-7 m^2/day (mean value would be D_{ref} = 7.72e-13·86400 = 6.67e-08 m^2/day), t_{Dref} = 3650 days, m_{coeff} = 0.37, t_{mcoeff} = 10950 days, C_s = 0.103, Cl_{crit} = 0.0185. Progressive period a_1 = 7.44e-5 m, a_2 = 7.30e-6 m, a_3 = -1.74e-5 m/MPa, $f_{t,ch}$ = 3.5 MPa, w_d = 0.001 m, pitting corrosion R_{corr} = 3, corrosion rate after spalling 30 µm/year. third part of the analysis is devoted for durability study.

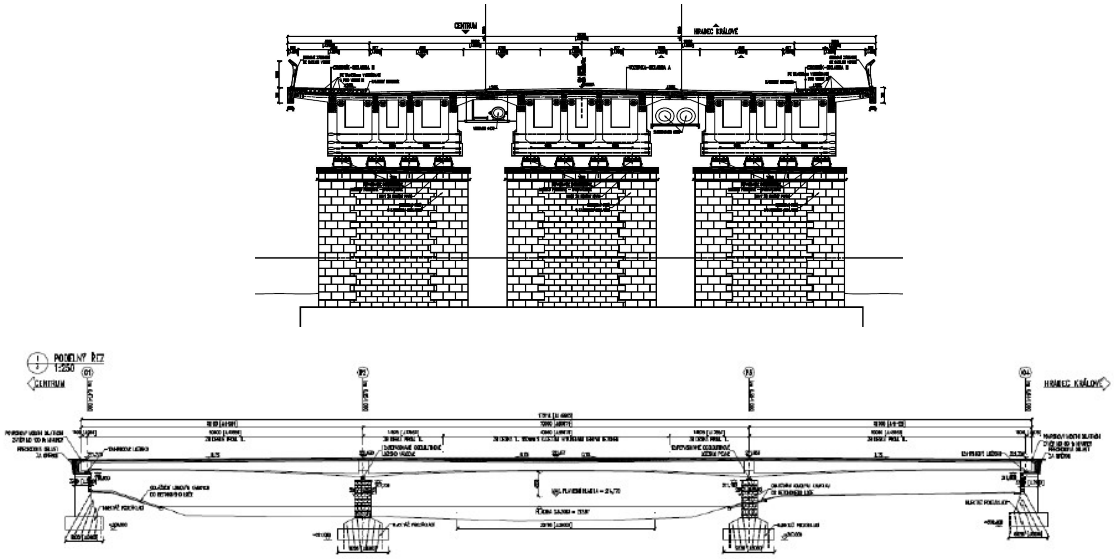

Figure 9: Dimensions of the bridge: drawing of cross section near the pillars, (top picture), mid-span cross section, (middle picture), and side (longitudinal) view of the bridge, (bottom picture) [16]

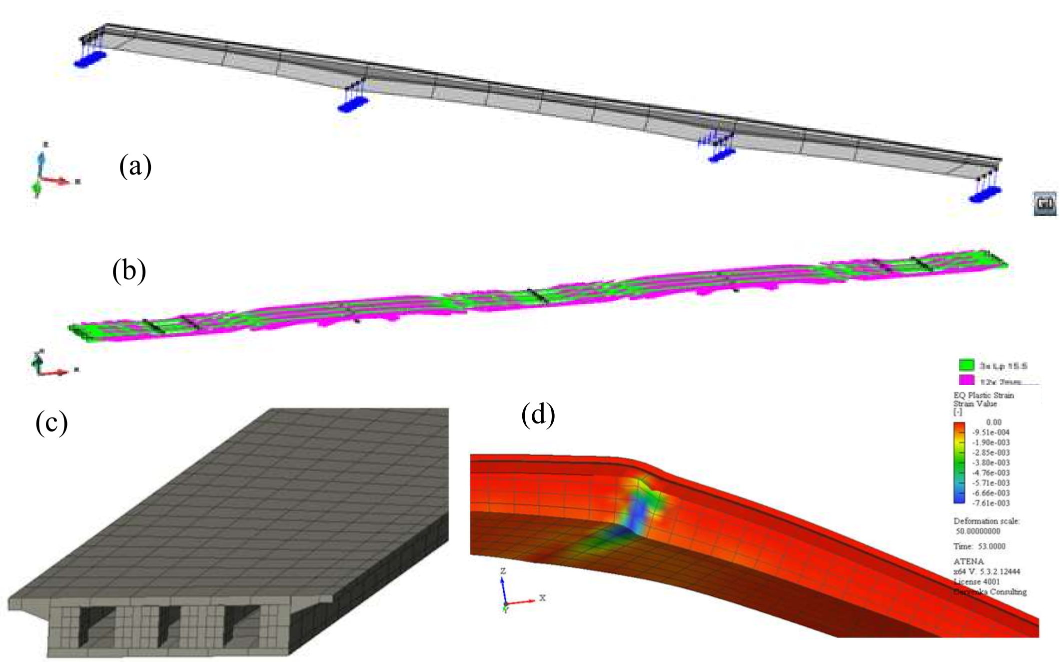

Figure 10: (a) Geometrical model of P. Wonka bridge, (b) geometry of pre-stressing cables, (c) the finite element model of the end segment, (d) failure mechanism at peak load near the right middle support during overloading up to failure

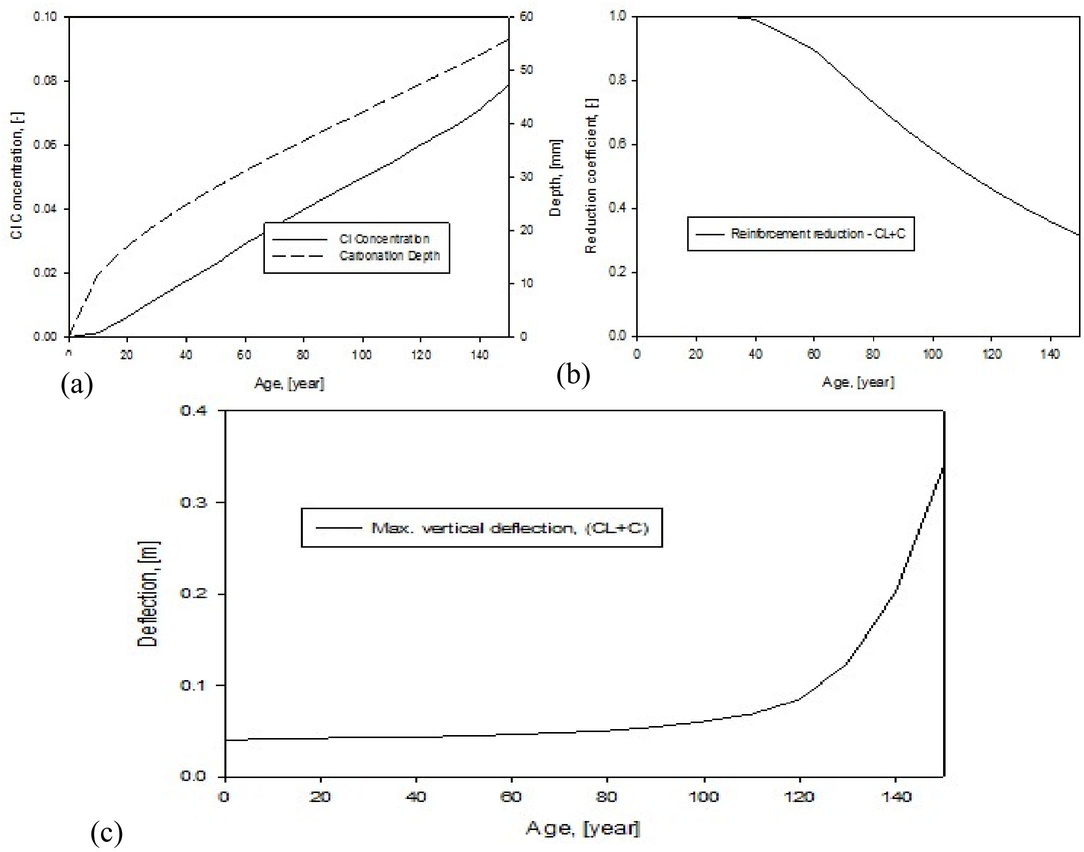

Figure 11: (a) Concentration of chlorides and carbonation at tendon depth, (b) tendon cross-sectional area reduction due to corrosion, (c) deflection increase in time

Chloride ingress assumed concentration of sea water on the surface; this resembles situation when salt brine water had leaked through insulation and the Cl concentration rose up substantially. Also, 90% confidence was considered for diffusivity D_{ref}, which is about twice higher than the mean value for this concrete strength class.

The main results are summarized in Figure 11, which shows that carbonation depth is about 55 mm for 150 years on uncracked concrete. The induction period for chlorides is approximately 45 years for uncracked concrete with concrete cover of 60 mm. Figure 11b depicts calculated reduction coefficient for a pre-stressing tendon with concrete cover 100 mm. For the first 40 years, the tendon do not corrode, but at the age of 100 years about 50% of the tendon’s cross section area has corroded away. Figure 11c shows also the maximum vertical displacement of the bridge vs. age of the structure. Note that the significant increase of the deflection at later times is due to tendons corrosion only as creep is neglected in computation and the force load is kept constant. It shows that the bridge is currently in a very good shape, without significant cracking and the possible service life of about 40 years.

11. CONCLUSIONS

The paper focuses on reinforcement corrosion due to chloride ingress. Implemented models in ATENA software allow the simulation of the most important degradation events during the service life of a structure, i.e. induction time, time of concrete cover cracking, time of concrete cover spalling and direct corrosion of reinforcement and reinforcement loss rate. The models take into account diffusion acceleration due to cracks in the structure.

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