Application of the 3D chemo-hygro-thermo mechanical model on existing bridges exposed to chlorides and mechanical damages

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Abstract. Bridge engineering world practice has showed that implicit method of service life prediction, relying on sufficient quality and depth of concrete cover, do not guarantee a 100-year structure lifetime without major, complex and expensive repair works. Bridges exposed to harsh combination of mechanical (static, dynamic, cyclic loading) and environmental (sea salts, de-icing agencies, freeze-thawing cycles, etc.) actions are particularly vulnerable. Two such case studies: Krk Bridge and Maslenica Bridge located in aggressive maritime environment will be analysed in the paper including in-service performance and comparison between measured values on bridges and numerical results obtained by two numerical models for service life prediction: the 3D chemo-hygro-thermo mechanical (3D CHTM) model implemented into the finite element code MASA and the Life-365 model. Both models are capable to realistically predict chloride content in concrete after long-term exposure to seawater. However, the 3D CHTM model, which considers cracks and damage in concrete, anticipates the beginning of steel reinforcement depassivation much more precisely than the other model which doesn’t take concrete damage and cracks into account.

1. Introduction

Tragedies that happen from time to time, such as collapse of Morandi Bridge in Italy in August 2018 [1], remain us on importance of our infrastructure condition. Most of infrastructure, such as bridges, tunnels, dams etc., are made of concrete. The main cause of degradation of reinforced concrete (RC) structures, exposed to de-icing salts or maritime environment, is chloride-induced corrosion of steel reinforcement in concrete, leading to service life reduction and maintenance costs increase. Bridges exposed to harsh combination of mechanical (static, dynamic, cyclic loading) and environmental (sea salts, de-icing agencies, freeze-thawing cycles, etc.) actions are particularly vulnerable.

In order to assure efficient and effective bridge management system, it is important to determine current condition of structures, but also to predict their future deterioration and remaining service life using numerical models. Many chloride ingress models for service life prediction of structure have been developed in the last four decades, but their application in management of existing structures and design of new bridges are still not on the satisfactory level, due to complexity of transport and corrosion processes in reinforced concrete structures, but also due to difficulty in quantifying the material, mechanical and corrosion related parameters and their interaction.
Two case studies: Krk Bridge and Maslenica Bridge located in aggressive maritime environment will be analysed in the paper including in-service performance and comparison between measured values on bridges and numerical results obtained by two numerical models for service life prediction: the 3D chemo-hygro-thermo mechanical (3D CHTM) model implemented into the finite element code MASA and the Life-365 model.

2. Case studies
Croatian coastal region of Adriatic Sea hosts seven long span RC arch bridges that, unfortunately, have been severely damaged by the corrosion processes in concrete due to the vicious ingress of salt sea water expedited by the powerful winds [2-3]. Namely, the Bora wind blows from north and north-east, causes salt spray and deposits chlorides on all structural elements [2-5]. The Bora is a dry wind, occurring without precipitation. Hence, there is no flushing of chloride from the concrete surfaces during rain, while the layer of salt on the exposed surface of the concrete structures is constantly being renewed.

Krk Bridge (table 1, figure 1) links mainland with the Krk Island consisting of two arches: Krk I and Krk II. Krk Bridge belongs to the first generation of Adriatic arches and is built in time of low awareness on concrete structure durability. Hence, several errors in design and construction were made, e.g. too small concrete cover, poor constructional detailing, underestimated effect of creep and shrinkage and inadequate drainage systems, causing cracking of concrete cover and reinforcement corrosion [6-8].

Maslenica Bridge (table 1, figure 1) belongs to the more recently built second generation of Adriatic arches and during design special attention is devoted to durability in order to avoid errors made on older bridges: designed greater thickness of the concrete cover, better detailing, implementation of structural health monitoring system, etc. [9]. However, according to the current knowledge, porosity of designed concrete was too high for such aggressive maritime environment. Additionally, the bridge construction started during the Homeland war and was located within range of insurgents’ artillery; hence the construction quality was not at the satisfactory level [5, 9]. On the columns at the arch abutments (S3 and S10), the deepest chloride penetration and the highest chloride concentration were detected after 13 years of services. Namely, those columns are the highest (H=70 m) and are built in first stage using the climbing form-work, hence it can be assumed that the construction quality was below average value. Moreover, the columns above arch abutment were additionally loaded by stays and back-stays during arch construction by free cantilever method, hence it can be assumed that first cracks and damage on the columns arise from the construction stage [5, 8, 9].

Table 1. Case studies: basic data on the Adriatic concrete arch bridges [7].

| Bridge | Completed in | Span [m] | Rise [m] | 1st Repair | Last Repair | Max wind gust [m/s] (year) |
|--------|--------------|----------|----------|-------------|-------------|--------------------------|
| Krk    | 1980         | 390, 244 | 60, 47   | 1981        | 2019        | 61.7 (2013)              |
| Maslenica | 1997    | 200      | 65       | 2017        | 2017        | 69.0 (1998)              |

Figure 1. Layouts of the Krk Bridge (left) and Maslenica Bridge (right) with marked position where chloride content is determined and compared with numerical results [8].
Table 2. Concrete mix design for Krk Bridge and Maslenica Bridge [4, 8, 10].

| Component        | Type                        | Mass [kg/m³] | Type                        | Mass [kg/m³] |
|------------------|-----------------------------|--------------|-----------------------------|--------------|
| Cement           | CEM II/A-S 42.5 20% slag    | 450          | CEM II/A-S 42.5 R 12% slag, 18% fly ash, 5% lime | 400          |
| Aggregate        | Dmax = 16 mm Alluvial crashed carbonate gravel | 1854         | Dmax = 16 mm Quarry Vrsi    | 1854         |
| w/c              | 0.36                        |              | 0.40                        |              |
| Admixture I      | Superplasticizer 0.890      |              | Superplasticizer 7.40       |              |
| Admixture II     | Air-entraining agent 0.667  |              | Air-entraining agent 0.08   |              |
| Admixture III    | -                           | Retarder 0.80 |                             |              |

Although, Krk Bridge and Maslenica Bridge are built at different times, both structures have good quality of concrete (table 2). However, first signs of reinforcement corrosion appeared early, already after few years of services and comprehensive investigation and repair works were provided for each bridge in the first two decades of service [8]. Determination of the chloride content in concrete of the bridges was carried out according to Croatian (European) standard HRN EN 14629:2007.

3. Numerical models for service life prediction

Two numerical models for service life prediction, the 3D CHTM model and the Life-365, are used for application on the case studies and for comparison of numerical results with data obtained on the bridges.

3.1. 3D chemo-hygro-thermo mechanical (3D CHTM) model

The recently developed 3D CHTM model [11-13], implemented into the MASA finite element code, is one of the most comprehensive models for service life prediction of concrete structures exposed to mechanical and non-mechanical loads.

The modelling of corrosion induced damage of concrete includes the following physical, electrochemical and mechanical processes: (i) transport of capillary water, heat, oxygen and chloride through the concrete cover; (ii) immobilization of chloride in the concrete; (iii) drying and wetting of concrete and related hysteretic property of transport of water through concrete; (iv) transport of OH- ions through electrolyte in concrete pores; (v) cathodic and anodic polarisation and mass sinks of oxygen on steel surface due to cathodic and anodic reaction; (vi) distribution of electrical potential and current density; (vii) transport of corrosion products through concrete pores and cracks; (viii) concrete cracking due to mechanical and non-mechanical actions and its interaction with non-mechanical processes. Since the chloride ingress models are in focus in this paper, only processes related to the transport of chlorides before depassivation of steel reinforcement will be explained in detail.

Transport of capillary water is described in terms of volume fraction of pore water in concrete by Richard’s equation [11, 14], based on the assumption that transport processes take place in aged concrete, i.e., the hydration of cement paste is completed:

\[
\frac{\partial \theta_w}{\partial t} = \nabla \cdot \left[D_w(\theta_w) \nabla \theta_w \right] \tag{1}
\]

where \(\theta_w\) is volume fraction of pore water (m³ of water / m³ of concrete) and \(D_w(\theta_w)\) is capillary water diffusion coefficient (m²/s) described as a strongly non-linear function of moisture content [11].

Transport of chloride ions through a non-saturated concrete occurs as a result of convection, diffusion and physically and chemically binding by cement hydration product [11, 14]:

3
\[ \frac{\partial C_c}{\partial t} = \nabla \cdot \left[ \theta_w D_c(\theta_w, T) \nabla C_c \right] + D_c(\theta_w) \nabla \theta_w \nabla C_c - \frac{\partial C_{cb}}{\partial t} \]  
\[ \frac{\partial C_{cb}}{\partial t} = k_r \left( \alpha C_c - C_{cb} \right) \]  

where \( C_c \) is concentration of free chloride dissolved in pore water (kgCl/m³ pore solution), \( D_c(\theta_w, T) \) is the effective chloride diffusion coefficient (m²/s) expressed as a function of water content and concrete temperature \( T \), \( C_{cb} \) is concentration of bound chloride (kgCl/m³ of concrete), \( k_r \) is binding rate coefficient, \( \alpha = 0.7 \) is constant [11].

The mechanical part of the model is based on the micro-plane model for concrete with relaxed kinematic constraint [15]. In the finite element analysis cracks are treated in a smeared way, i.e. smeared crack approach is employed. To assure the objectivity of the results with respect to the size of the finite elements, the crack band method is used [16].

The governing equation for the mechanical behaviour of a continuous body in the case of static loading condition reads:

\[ \nabla[D_m(u, \theta_w, T)] \nabla u + \rho_b = 0 \] 

in which \( D_m \) is material stiffness tensor, \( \rho_b \) is specific volume load, \( T \) is temperature and \( u \) is displacement field. In the mechanical part of the model the total strain tensor is decomposed into mechanical strain, thermal strain, hygro strain (swelling–shrinking) and strain due to expansion of corrosion products.

The transport processes in concrete depend on the damage (crack) in concrete. Hence, the water and chloride diffusivity, as the relevant parameters for transport processes, are employed in the model as function of crack width based on the experimental results for permeability in cracked and fully saturated concrete: the concrete permeability and diffusivity in cracked concrete (crack width, \( cw \geq 0.20 \text{ mm} \)) is 1000 times higher than in un-cracked concrete of the same quality [11].

3.2. Life-365 model

The Life-365 model calculates chloride ingress in un-cracked concrete according to the Fick’s second law, assuming diffusion as dominant transport processes [17]:

\[ \frac{dC}{dt} = D \frac{d^2 C}{dx^2} \]  

where \( C \) is chloride content, \( D \) is apparent diffusion coefficient, \( x \) is depth from the exposed surface and \( t \) is time.

The chloride diffusion coefficient is a function of time:

\[ D_{\text{ref}} = D_{28} = 1 \cdot 10^{-12.06+2.40w/c} \]  
\[ D(t) = D_{\text{ref}} \left( \frac{t}{t_{\text{ref}}} \right)^m \]  
\[ m = 0.2 + 0.4 \left( \frac{\%FA}{70} - \frac{\%SG}{70} \right) \]  

where \( D_{\text{ref}} \) is diffusion coefficient at time \( t_{\text{ref}} = 28 \text{ days} \) and temperature \( T_{\text{ref}} = 293 \text{K (20°C)} \), \( w/c \) is water-to-cement ratio, \( m \) is constant depending on concrete mixture based on the level of fly ash (\%FA) or slag (\%SG) in concrete.
4. Application of numerical models on the case studies

4.1. Numerical modelling

The analyses are carried out for two environmental exposure conditions XS1 and XS3, according to the European Standard EN 206:2013. Applied diffusivities for both numerical models are shown in table 3, while initial and boundary conditions for both exposure conditions and both numerical models are listed in table 4 [8].

Two different approaches are applied for numerical modelling using 3D CHTM model. For calculation of chloride content in concrete of the Krk Bridge, segment of structural element is modelled with an assumed parallel-face crack of constant width, ranging from 0.0 mm for un-cracked concrete to 0.2 mm presenting an open crack [8].

For the Maslenica Bridge case study, “V” shaped cracks are generated by four-points bending, with maximal crack width on concrete surface of 0.2 mm or larger (figures 2 (a)-(b)) [5]. Comparing the distribution of chlorides at different times in cracked concrete (figure 2(c)), chlorides penetrate along the crack immediately after crack opening, and after 1-year chloride content in crack decreases, because chlorides penetrate in the horizontal direction, in the region between the cracks. Therefore, there is a slight decrease of their concentration in the crack, i.e. with increase of time chlorides tend to be smeared-out into the horizontal direction. Contrary to cracked concrete, the chloride concentration in un-cracked concrete cover increases gradually by time (figure 2(c)).

| Table 3. Applied diffusivities in numerical models. |
|---------------------------------------------------|
| Model | Parameter | Krk Bridge | Maslenica Bridge |
|-------|-----------|------------|------------------|
| 3D CHTM | Chloride diffusion coefficient in un-cracked concrete, $D_{c,0}$ (m²/s) | $1.00 \times 10^{-12}$ | $5.50 \times 10^{-12}$ |
| | Chloride diffusion coefficient in un-cracked concrete, $D_{c,0}$ (m²/s) | $2.20 \times 10^{-10}$ | $2.20 \times 10^{-10}$ |
| Life-365 | Chloride diffusion coefficient in $t_{ref}=28$ days $D_{ref}$ (m²/s) | $6.37 \times 10^{-12}$ | $7.94 \times 10^{-12}$ |
| | Chloride diffusion coefficient $D(t)$ (m²/s) | $1.11 \times 10^{-12}$ | $0.96 \times 10^{-12}$ |
| | $t=20$ years | $t=13$ years |

| Table 4. Initial and boundary conditions. |
|------------------------------------------|
| Exposure class | Condition | 3D CHTM model | Life-365 model |
|----------------|-----------|---------------|----------------|
| XS1, XS3 | Capillary water | Initial $\theta_{w}=0.010$ | Initial $\theta_{w}=0.010$ |
| | Boundary | Initial $C_c=0$ kg/m³ | Krk Bridge: $C_s=0.25 \% m_{con}$ |
| | | | Maslenica Bridge: $C_s=0.27 \% m_{con}$ |
| XS1 | Chlorides | Initial $C_c=8.5$ kg/m³ | Krk Bridge: $C_s=0.6 \% m_{con}$ |
| | Boundary | Maslenica Bridge: $C_s=0.64 \% m_{con}$ | |
| XS3 | Boundary | Initial $C_c=20$ kg/m³ | |
| | | Maslenica Bridge: $C_s=0.64 \% m_{con}$ | |
4.2. Comparison of numerical results and measured chloride values

In order to compare numerical results with measured values, the total amount of chlorides, in numerical simulation provided by the 3D CHTM model are expressed as percentage of concrete mass:

\[
\frac{m_{\text{free+bound chlorides}}}{m_{\text{concrete}}} = \frac{C_c \cdot p \cdot S + C_{ch}}{\rho} \tag{6}
\]

where \( p \) is porosity, \( S \) is saturation and \( \rho \) is density of concrete.

Comparisons of numerical results, calculated by the 3D CHTM model and the Life-365, and measured values on the Krk Bridge and Maslenica bridge are shown on figures 3 and 4, respectively. Values obtained on the Krk and Maslenica bridges are within the range of numerical results calculated by two different numerical models leading to a conclusion that boundary conditions for the numerical models (exposure classes XS1 and XS3) are well assumed. The 3D CHTM models provides wider range of results since it includes several scenarios without and with a crack in concrete with different crack widths, while Life-365, not considering cracks in concrete, provides an average chloride profile.

A specificity of Adriatic climate can be seen in figure 4. Superstructure main girder of the Maslenica Bridge is approximately 70 m above sea level, but measured chloride values are very high and belong to exposure class XS3. This phenomenon happens because of strong bora wind, which blows and rise sea foam depositing chlorides on all structural elements.

Although the both models are capable to predict the chloride content in concrete after long-term exposure to the sea with satisfactory accuracy, early presence of cracks in concrete can caused reinforcement depassivation already during the first month of service (figure 2(c)). This phenomenon is confirmed on the analysed bridge case studies: during investigation works, when the concrete samples were taken for chloride content in concrete determination, obvious signs of reinforcement corrosion in form of brown spots and concrete cover delamination were detected, although the chloride content at the reinforcement level did not achieve the threshold value.

Moreover, it is important to model chloride ingress in 3D domain, since chloride penetrate fast into the crack and after certain time chlorides penetrate in the region between the cracks. Hence, not only crack width, length and depth, but also distance between cracks has significant impact on prediction of depassivation time.
Figure 3. Total chloride content after 20 years of sea exposure for the airborne salt zone, XS1: comparison of the numerical results and measured values on the Krk Bridge.

Figure 4. Total chloride content after 13 years of sea exposure for the splash and spray zone, XS3: comparison of the numerical results and measured values on the Maslenica Bridge.

5. Conclusion
Two numerical models for service life prediction, the 3D chemo-hygro-thermo mechanical model and the Life-365, are used for application on two existing structures: Krk and Maslenica Bridges exposed to very harsh maritime environment of the Adriatic Sea. Both models are capable to predict the chloride content in concrete after long-term exospore to sea. However, presence of cracks in concrete significantly decreases reinforcement depassivation time. Influence of crack width and depth, but also distance between cracks on chloride ingress in concrete should be considered to predict depassivation time more realistically.

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