

## **DURABILITY DESIGN OF CONCRETE STRUCTURES - PART 2: MODELLING AND STRUCTURAL ASSESSMENT**

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**Abstract.** *This paper is a sequel to [19] where the theoretical foundations were given for the analysis of durability of concrete structures. An overview of some approaches to modeling the durability of RC structures is given in the paper. Experiences resulting from experimental and numerical research of some influential parameters leading to corrosion of the reinforcement, cracks and delaminating of the surrounding concrete, which leads to the decrease of service life of the RC structure's elements, are described in details. Methodologies of structure evaluation presented in this paper are based on assessment of critical influential parameters and represent a methodology of reasonable maintenance of an engineering construction, i.e. reasonable service life of the structure. The methods are based on analysis of conditions and performance (Condition Rating Method and Performance Rating Method). The results of ratings of concrete structures in the industrial zone of the city of Tuzla are presented in the final part of the paper.*

**Key words:** *concrete structures, durability, modelling, corrosion, influential parameters, construction assessment, construction classification, performance, real condition, deterioration*

### 1. INTRODUCTION

Practical experience proved that concrete structure durability analysis based on understanding deterioration processes in concrete reinforcement and prestressing steel is a necessary prerequisite to achieve a realistic service life. Failing to understand these processes may result in both decrease of service life and collapse of the structure. Examples of this practice are the following:

- Port Talbot Wales, December 1984 – Collapse of the prestressed bridge with a span of 18 m, constructed in 1953. During its service life it was loaded with small live load. The collapse resulted from inadequate protection of cables against moisture, i.e. from evaporation from the river.
- Berlin, May 1980 – Collapse of the Congress Hall's external roof as the result of corrosion.
- Wormerveer in Holland, 1990 – Collapse of the RC gallery as the result of carbonization induced corrosion. Instead of the upper zone where the cracks were formed, enabling the penetration of salt, the gallery's main reinforcement was situated in the lower stressed zone.
- Melle, Belgium – Collapse of the prestressed bridge as the result of gradual widening of cracks under large load, enabling the penetration of chloride and corrosion of prestressed cables.

The modern approach to the analysis of concrete structure requires defining mathematical models for quantifying deterioration processes as the main prerequisite to design a structure with realistic service life. Mathematical models include the analysis of carbonization, resistance against chloride, alternate freezing and thawing. In [18] fundamental analysis is based on the national and international codes and standards: fib, ACI, RILEM and so on. Review of the literature and some recommendations are given considering the design of structures which goal is to achieve greater durability of concrete structures. Fundamentals of this analysis are given through the principles of performance and service life, as well as through deterministic methods and methods using the lifetime safety factor.

In structure management, it is important to use a methodology which enables the detection of the deterioration process in its initial stage, as well as the definition of its causes. It is important to understand the intensity of influence regarding the optimization of the structure's program of reparation. The approach to structure management depends on a multitude of factors, such as:

- the structure's social and economic importance,
- its function,
- the method of design,
- calculated service life,
- potential influences on the structures and on the structure's secondary parts,
- environmental conditions,
- possible mechanism of deterioration,
- complexity of inspection and maintenance of the structure.

In the fib bulletin and [15] the classes of maintenance are proposed, taking into account the above mentioned influential factors. Regardless the class of maintenance, it is necessary to inspect the structure. Inspection may be scout, initial, routine or regular, detailed or special inspection. The program of inspection is in accordance with the importance of construction. The inspection of construction is an activity of major importance for monitoring and correcting the program of the construction's maintenance, regarding the optimization of the structure's service life. The inspection's main goal is to rate the structure, which implies adequate procedures of assessment of effects of specific deterioration mechanism on the construction. This activity needs to be followed also by adequate calculations of the structure, encompassing the factors of the structure's sensitivity, type of the structure, consequences of failure, the load's actual intensity and actual deterioration effects.

Regardless of the structural rating procedure in use, in order to enable correct diagnosis of the problem and the choice of an adequate structure management strategy, it is necessary to define the structure's deterioration mechanism and its relative importance on the entire process of decrease of the structure's properties. Methods in use in structural rating are known as Conditional Rating Method and Performance Rating Method. In this paper our consideration is limited mostly on durability related to chemical processes.

## 2. MODELLING CS FOR DURABILITY

The deterioration process is very complex and the interrelation between the: chosen structural concepts; conditions of exposure; composition of the concrete; quality of the execution process and quality of maintenance, is not fully comprehended. Long-term durability of RC structures is a major concern for safety, economical and enhancing environmental sustainability [19]. Multi-level modelling concept seems to be particularly relevant. It consists of proposing a set of models, covering different levels of sophistication. A general approach is based on so called durability indicators (DIs) which is key material properties with regard to durability. This can be used for new structure in the design stage or the residual SL on existing and deteriorated structures. DIs is in this case the main input data of the predictive models [19]. However, state of practice in design CS prediction of service life routinely is considered with oriented degree of reliability. To achieve the highest level of reliability and a desired one, durability designers should use developed mathematical models. Traditionally, designers estimate service life using judgment based on experience with similar structures in similar environments. The lack of experience and no standard guidelines cause bed solution for some important structures. Low durability of CS, especially in aggressive environments, conditions the development of the mathematical models for prediction of the service life.

Recently all professional associations are focused on active work and development guidelines and models for designing CS durability. In [25] service-life models are classified in three main categories: empirical, mechanistic (physic-chemical), and semi-empirical. Empirical models based on previously observed relationships of service life, concrete compositions, and exposure condition depend on environmental conditions, without invoking and understanding scientific reasons for these relationships. This category includes neural network models. The physic-chemical model provides predictions of service life based on mathematical descriptions of the phenomena involved in concrete degradation, for example, understanding microstructure of concrete before and during degradation process. Semi-empirical models tend to use more simple mathematical expressions than mechanistic one. Predictions are made by using fitting parameters that are calibrated on the basis of data on the concrete performance in the field or in laboratory.

Service-life prediction models may be probabilistic, when service-life is expressed in the form of probabilistic distribution functions. The probabilistic approach allows taking into account the uncertainties and the variability of the physical and chemical parameters. The quality of service life predictions depend on capability of the models used and quality of the input data. For ensuring viability of appropriate methods of characterization of concrete one should provide data for testing and use of models. The ability to specify performance requirements for concrete should be developed as well. Guidelines for selection of limits for tests for relevant degradation mechanisms should be provided, and allowed

scattering in test results. Some prescriptive tests may be still used as performance tests, for example, a limit on chloride content. In [25] nine steps are indicated, including service life predictions, that should be performed in the design and detailing of reinforced concrete structures to achieve a required service life:

- Step 1: Defining the problem (the type of structure, the required service life and the exposure conditions) in the beginning of a project. The exposure conditions for individual elements of the structure depend on natural environment, and are imposed due to it. The exposure conditions are likely to differ at different points on the surface (water due to excessive deflection, thermal effects, i.e.). Each type of degradation involves: transport of water and ions through the concrete; reaction involving a volume change, degradation of the internal structure of the concrete. Degradation may involve one or more mechanical, chemical, thermal, and electrochemical processes.
- Step 2: Proportioning the mixture, considering the surface, and providing for drainage. Proportioning the concrete additives and chemical admixtures should be carefully considered, because it can influence service life (reduced permeability of concrete). Surface treatment and overlays can increase service life. Drainage slope and camber must be designed.
- Step 3: Reinforcement selection. Different level of protection (using stainless steel, epoxy-coated steel reinforcement, cathodic protection, controlling the tensile stresses with prestressing) can increase service life.
- Step 4: Designing the cover. The quality and thickness of concrete cover over the reinforcement is one of the most important factors effecting service life. Cover deterioration and corrosion of the reinforcement can lead to reduced bond integrity and spalling and delaminating of cover.
- Step 5: Controlling cracks. Early age cracking develop tensile stresses and cause changes in the concrete material properties as elastic modulus, tensile strength, and early age creep. Construction joints and pour strips must be carefully located to minimize restraint effects.
- Step 6: Using models in estimating service life. When designing durability the many factors are considered and usually iterative process performed. In each iteration, the design parameters are being adjusted until an estimate (based on a model-based prediction) service life made in step 6, is judged to be acceptable. Service-life modelling requires an understanding of chemical reactions and transport processes within concrete, as well as thermal, mechanical, and electrochemical effects. It is very important to choose a model for specific application: the exposure conditions, validation model by numerical methods, laboratory tests and field observations; and what assumption does the model depend upon? Influence of variation in cover thickness, placing and material variation tolerances, mixture proportions, curing and exposure conditions need to be investigated for important structures. In case when service life lack to meet or exceed the required service life it will be necessary to return to Step 2, and repeat subsequent step with some of the adjusting design parameters.
- Step 7: Selecting, in the design phase, appropriate procedures for quality assurance and quality control of construction (control of locations of reinforcement before concreting, periodic cover measurement before and after concreting), duration and type of curing.

Step 8: Assuring quality construction. Providing effective measures of control.

Step 9: Developing a plan. For achieving durable structures in all phase of design, construction, use, inspection and maintenance adequate quality must be provided.

### 3. ASSESSMENT USING STRUCTURAL RATING METHODS

There are various systems of structural rating and qualification of some of the influential parameters, using multi criteria decision making with well-developed analysis tools. A range of methods is proposed within the framework of the LIFECON (Life Cycle Management of Concrete Infrastructures for Improved Sustainability) project, presented in [23]. To describe the relative importance of specific criteria, most of the tools use the weight factor. These procedures are based on integrated approach to encompass all influential factors, as the structure's operating costs, its performances, as well as the political and social aspects. The system of decision making (rating) has to be realistic as to the choice of extent of inspection and number of influential parameters. Also, it is necessary to include the investor into the process to analyze the benefits and drawbacks of the proposed system.

Today, there are two main methodologies in use for structural rating:

- Condition rating method
- Performance rating method

Condition rating method was applied before making decision about the level of repair that heavily corroded structure undergoes [10]. The obtained results are compared with the results of standard limit states verifications (update design resistance with data experimentally established). In [12] the condition rating method is compared with the safety factor of assessment LRFA (load and resistance factor used in USA).

First, a group of potential damage types is defined: corrosion phenomena, deterioration of the concrete, cracking, construction defects, etc. The damage function  $M_m$  is calculated as an addition of terms, one for each  $i$ -th damage type effectively observed on the structure:

$$M_m = \sum_i B_i K_{2i} K_{3i} K_{4i} \quad (1)$$

where:

$B_i$  basic value of  $i$ -th damage type, expressing its potential effect on the safety and durability of the structural component under observation (values range 1-4),

$K_{2i}$  intensity factor for the  $i$ -th damage, determined by qualitative visual criteria and experimental

measurements in a scale of four degrees (values range 0.5-2),

$K_{3i}$  extension factor for the  $i$ -th damage within the elements under consideration (values range 0.5-2),

$K_{4i}$  urgency of intervention factor for the  $i$ -th damage (values range 1-5)

A Condition Rating Factor (CR) is calculated as the ratio of the observed damage function  $M_m$  and the maximum possible damage  $M_{ref}$ :

$$CR = M_m / M_{ref,m} \quad (2)$$

Based on the values of CR the appropriate deterioration classes are defined and the values of the deterioration factor  $\alpha_R$ .

Table 1 Deterioration Classes  
(CEB: Strategies for testing and assessment of concrete structures, BI No. 243)

Class	Description of the condition	CR	$\alpha_R$
I	No defect, Only construction deficiencies.	0-5	0,3
II	Low degree deterioration, which only after long period of time might be the cause for reduced serviceability or durability of the affected structural component, if not repaired in proper time.	3-10	0,4
III	Medium degree deterioration, which can be the cause for reduced serviceability and durability of the affected structural component, but still not requiring any limitation of use of the structure.	7-15	0,5
IV	High degree deterioration, reducing the serviceability and durability of the structure, but still not requiring serious limitation of use.	15-25	0,6
V	Very heavy deterioration, requiring limitation of use, propping of most critical components, or other protective measures.	22-35	0,7
VI	Critical deterioration, requiring immediate propping of the structure and strong limitation of use, e.g. closing.	$\geq 30$	0,8

Evaluation procedure is based on the application of reduction factor for ultimate limit states:

$$S_d < \Phi R_d \quad (3)$$

The reduction factor can be determined by the expression:

$$\Phi = B_R \cdot e^{-\alpha_R \beta_C V_R} \quad (4)$$

where:

$B_R$  is the ratio between the real capacity of an element and its nominal value, as calculated from

code assumptions and formulas, without considering strength deterioration;

$\alpha_R$  is the deterioration factor (see Table 1);

$V_R$  is the coefficient of variation of strength, determined from the tests and inspections data on the

materials, considering the reliability of these measurements (10-20%);

$\beta_C$  is the target value of the minimum acceptable safety level (i.e. reliability coefficient), from 3.3 to 4.3 in design, with a common value of 3.8 according to [13] and [21]. In the CEB publication [10] indicates the use of 2.5 for predictions related to a short period of time until the following assessment, or 3.5 for normal service life.

The factor  $\Phi$  takes values ranging from 0.5 in the case of very deteriorated structures and with no maintenance and regular inspection, to 1 in the case of structures in good conditions and undergoing accurate inspections [11].

One of the procedures for Performance Rating Method is presented in the bulletin [19]. Evaluation of a structure is carried out using performance indicators, which are assumed monotonically decreasing function of time. It can be expressed in terms of various units: mechanical, financial, reliability, etc. In all cases, after a certain period of time, the "performance indicator" decreases, for example due to corrosion of steel, carbonation of concrete, repeated opening of cracks in concrete member, spalling, etc. The principal requirement considered in the overall strategy for achieving durability is, in particular, deci-

sion with regard to the life performance required from the structural members and whether individual members are to be replaceable, maintainable or should have a long-term design life.

The development and definition of appropriate Performance Indicators requires consideration of a range of issues such as:

- The condition of the structure/substrate.
- The nature of the loading and environmental conditions to which the structure will be exposed.
- The deterioration mechanisms affecting the original structure.
- Test or investigation procedures that are sensitive to the mechanisms of deterioration concerned and hence can reliably detect/monitor these changes and effects arising over time.

The methodology is based on the definition of requirements  $R$  for the protection and repair intervention and/or the repaired structure. These are developed via a series of Performance Indicators  $PI$  that are scored from 4 (bad) to 1 (good) for the particular criterion.

An overall assessment is given by the calculation of a Repair Performance Index  $RPI$  which is obtained by the addition of the individual  $PI$  multiplied by the Relative Importance  $Im_{PI}$ .

$$RPI = \sum_1^n PI \cdot Im_{PI} \quad (5)$$

Number of  $PI$  depends on the structural importance and environmental conditions.  $PI$  for the selection of intervention in the structures can be:

- Debonding. Crack width between repair and substrate. Bond to substrate.
- Cracking in the patch material
- Carbonation front rate factor
- Chloride ion diffusion coefficient
- Water absorption
- Reinforcement potential. Reinforcement corrosion rate.
- Concrete cover resistivity
- Concrete cover mechanical strength.

The experiences and results obtained in so far conducted research on deterioration mechanisms in concrete structures along with the influential parameters and conditions are presented in the next paragraph.

#### 4. SOME EXPERIENCES RELATED TO DETERIORATION MECHANISMS IN CONCRETE STRUCTURES

Corrosion of the reinforcement can show different forms, ranging from widespread general corrosion to a very local attack (pitting corrosion). General corrosion, mostly in cases of carbonated concrete, leads to early cracking and spalling of the concrete, often with comparatively little reduction of the cross-section of the reinforcing steel bars, whereas the localised corrosion due to chloride ions results in pits, randomly distributed along the steel bars. The corrosion rate, generally, can be divided in an initiation period and a propagation period.

The initiation period depends on a variety of parameters such as concrete quality and concrete cover. The propagation period is governed mainly by the electrolyte resistance influenced by temperature and moisture content and to a smaller part by electrochemical reaction resistances (Fig.1) [7].

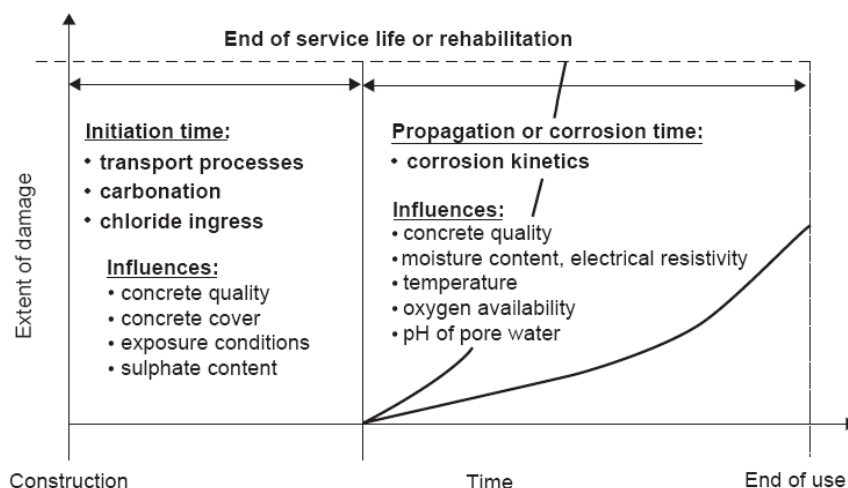


Fig. 1 Extent of corrosion damage over time [7]

Influence of cracks in the corrosion rate depends on whether or not the cracking occurs in the initiation or propagation phase of corrosion. If the cracks are caused by factors other than corroding reinforcement they could influence the initiation phase to some extent. Also crack orientation has a significant role in the corrosion rates. If the cracks are parallel to the reinforcement and over it they will pose a more significant problem than if they are perpendicular. Cracks are capable of self healing in certain circumstances – autogenous healing. This effect can be caused by the precipitation of calcium carbonate in the crack leading to clogging of the crack and thereby preventing further ingress of harmful species. Individual cracks that intersect the reinforcement locally may encourage anodic activity but access to the cathodic areas will not be different to that pertaining in crack-free surfaces. Therefore the rate of corrosion should not be different, as it will continue to be influenced by the accessibility of the cathodic sites [28].

Carbonation-induced corrosion can lead to spalling on structures. Corrosion of reinforcement can commence however if the passive oxide film protecting the reinforcement is destroyed, the cover concrete is sufficiently permeable to oxygen and moisture, and the concrete is moist enough to serve as an electrolyte. For a durable corrosion protection of the reinforcement a pH of 11 to 11.5 is necessary [7]. The lowered pH in zones of carbonated concrete may threaten the continuity of the passive film. It is important therefore to specify cover concrete that is capable of resisting the penetration of the carbonation front as far as the reinforcement during the service life of the structure.

There are many different methods for locating the carbonation front. The simplest method to use is the acid/base indicator phenolphthalein. The papers [16] and [17] describe the experience of using this method in evaluation of structures in Tuzla industrial



zone. The depth of carbonation may be regarded as the average distance from the surface of the concrete element to the zone where phenolphthalein indicator solution changes colour to purple, indicating that carbon dioxide has not reduced the alkalinity of the hydrated cement in that zone.

Carbonation rate is significantly influenced by several factors as follows:

- diffusivity/permeability;
- reserve alkalinity;
- the environmental carbon dioxide concentration;
- the exposure condition.

Carbon dioxide diffusion is  $10^6$  times slower in water than in air. Rates of carbonation are highest in the range 50 to 75 per cent relative humidity. Below about 45 per cent relative humidity the water in the pores is not in a state that encourages dissolution of calcium hydroxide or carbon dioxide and this reduces the reaction rate. Above 75 per cent relative humidity the influence of water filling the pores becomes significant. The critical relative humidity level with respect to corrosion is about 80 per cent, below which there is insufficient moisture to sustain the process [28].

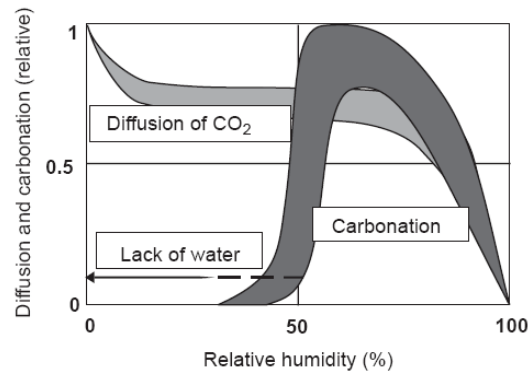


Fig. 2 Relative diffusion coefficient of CO<sub>2</sub> and relative rate of carbonation as a function of relative humidity [7]

In the paper [27] is given an expression for determination of carbonation depth at the end of initiation phase in form:

$$d = \frac{ak^{0.4}t_i^n}{c^{0.5}} \quad (6)$$

where is:

$a$  - coefficient

$k$  - air permeability of the cover concrete (units of  $10^{-16} \text{ m}^2$ )

$t_i$  - duration of initiation period to start of corrosion

$c$  - calcium oxide content in hydrated cement matrix that can react with carbon dioxide ( $\text{kg/m}^3$  of the cement matrix)

$n$  - power exponent (value close to 0.5 for indoor exposure and decreases with increasing relative humidity above 70 per cent)

The application of Parrott's formula in practice requires evaluation of specific parameters, some of which may require calibration with extensive field experience. The depth of carbonation  $d$  at the end of the initiation phase would in practice be taken as the minimum depth of cover. The coefficient  $a$  is used to calibrate the equation with observations from structures in service. Parrott has assigned a value of 64 for the coefficient, based on an extensive literature review.

The model being explored by the CEB is based on the following form of the carbonation relationship:

$$x_c = \sqrt{\frac{2k_1 k_2 D_{eff} C_s}{a}} \sqrt{t \left( \frac{t_0}{t} \right)^n} \quad (6)$$

where is:

- $x_c$  - depth of carbonation (m) at time  $t$ (s)
- $k_1$  - constant related to execution
- $k_2$  - constant related to exposure condition
- $D_{eff}$  - effective diffusion coefficient (m<sup>2</sup>/s)
- $C_s$  - concentration of carbon dioxide
- $a$  - chemical buffering capacity
- $t_0$  - age at which  $D_{eff}$  is determined
- $n$  - constant related to the environment

In the EN 206–1 exposure categories are given. Four exposure classes have been distinguished in respect of corrosion induced by carbonation and these are designated as XC1, XC2, XC3, and XC4. An informative annex in EN 206–1 presents indicative limits for maximum water/cement ratio, minimum cement content, and minimum strength class for each exposure condition, based on an intended working life of fifty years.

Tests for chloride level are generally based on acid-soluble chloride techniques. Chloride content can be determined in a number of ways. Review of literature relating to the testing of chloride penetration is presented in the paper [26]. The Volhardt titration method using silver nitrate is often used. The critical level at which passivity is lost and corrosion commences is not clearly defined and significant variations exist in the literature on the chloride level which was found to initiate corrosion. The onset of corrosion and therefore the critical chloride threshold level is dependent on several factors, including the following [23]:

- the chemistry of the binder, especially the  $C_3A$  content
- the proportion of the total chlorides that are free chlorides
- chloride ion to hydroxyl ion ratio
- the water/binder ratio
- the hydroxyl ion concentration
- temperature and relative humidity
- electrical potential of the reinforcement.

A review of the literature reveals a trend of agreement that 0.4 per cent chloride by mass of cement represents an acceptable benchmark for design purposes. European Standard EN 206–1 [11] limits the chloride content in a range from 0.10 to 0.40 per cent in the case of concrete containing reinforcement.

In the paper [12] is suggested the following description of corrosion risk:

- low risk - less than 0.4 per cent chloride by mass of cement
- medium risk - 0.4 to 1.0 per cent chloride by mass of cement
- high risk - greater than 1.0 per cent chloride by mass of cement.

A similar set of criteria for corrosion risk is presented in the paper [8]:

- negligible risk - less than 0.4 per cent chloride by mass of cement,
- possible risk - 0.4 to 1.0 per cent chloride by mass of cement,
- probable risk - 1.0 to 2.0 per cent chloride by mass of cement,
- certain risk - greater than 2.0 per cent chloride by mass of cement.

A review of some national standards and European Standard EN 206–1, for example, reveals values in a range from 0.1 to 0.4 per cent chloride by mass of binder including the following examples: 0.4 per cent in the case of reinforced concrete, 0.2 per cent where sulphate resisting Portland cement is used, 0.1 per cent for prestressed and heat cured concrete. Buenfeld [9] stated that even 0.2 per cent chloride by mass of cement can lead to corrosion if the chlorides are introduced from an external source and a large proportion remain free in the pore solution. Extensive durability failures in highway structures subjected to de-icing salt application led American Concrete Institute Committee 222 [1] to take a very conservative approach in framing recommendations. They suggested levels of 0.20 per cent for reinforced concrete and 0.08 per cent for prestressed concrete. The results of a study of bridges in the United Kingdom showed that chloride content between 0.35 and 0.5 per cent by weight of cement gives a corrosion risk of below 25 per cent. Chloride contents in excess of 1.0 per cent give a corrosion risk of over 70 per cent.

For the consideration of diffusion, many authors have performed expressions based on Fick's second law, which concerns the rate of change of concentration with respect to time. Use of error function Crank's solution in design would involve the assumption that the surface chloride level remains constant over time.

It is found to be a reasonable assumption to use values of the order of those observed in practice shortly after exposure. Notional surface chloride levels can be determined either from extrapolated values in particular cases or from the literature [4]. The paper [4] proposed somewhat lower value of 0.75 per cent on the basis of eight-year exposure trials. Design values of 0.50 and 0.75 per cent were proposed for the spray zone and marine atmosphere respectively. It also distinguished between values for Portland cement concretes and those containing slag. The values for Portland cement concretes were about 20 per cent lower than for blended cement concretes.

The paper [19] reported that a  $C_s$  value of 0.9 per cent by weight of concrete is a reasonable design estimate for the tidal and splash zone. This would equate to a value of approximately 6.0 per cent by weight of cement.

In the marine splash and tidal zone values of surface concentration 17.8 kg/m<sup>3</sup> is determined, which equate to about 4 to 5 per cent by weight of cement for a typical structural concrete. The values are similar in the case of car park decks [6]. Siemes et al. [30] used  $C_s$  values of 3 and 4 per cent by weight of cement in the case of a tunnel exposed to saline ground waters.

The rate of chloride build-up in bridge decks is thought to be lower than that in the marine environment. The paper [6] proposed a rate of 0.7 kg/m<sup>3</sup> per year due to the effect

of rain and the surface concentration built up to about 14.8 kg/m<sup>3</sup>. This equate to about 3.5 to 4.5 per cent by weight of cement for typical structural concretes. Surface concentrations of 2 to 4 per cent by weight of cement were found by Polder and Hug in a thirty-year old bridge subject to de-icing salt application at an annual dosage rate of about 250 grams of chloride per square metre.

A review of a wide range of published values of diffusion coefficients determined for Portland cement concretes are of the order of 10<sup>-12</sup> m<sup>2</sup>/s.

The paper [30] proposed the following values of diffusion coefficient which was described as typical: 1.50 x 10<sup>-12</sup> m<sup>2</sup>/s for a dense Portland cement concrete, 0.75 x 10<sup>-12</sup> m<sup>2</sup>/s for a high slag content concrete, and 0.3 x 10<sup>-12</sup> m<sup>2</sup>/s for a high quality ash concrete.

Exposure categories in EN 206-1 [13] are divided into exposure classes for chlorides from seawater (XS1, XS2, XS3) and are identified in each set: and chlorides other than from seawater (XD1, XD2, XD3).

According to standard ASTM C 876 [3] and measurements presented in the paper [31] corrosion condition related to half-cell potential measurements, with respect to a saturated calomel electrode (SCE) are:

- Extensive corrosion < -426mV.
- High corrosion (< 90% risk of corrosion) < -276mV
- Intermediate corrosion -126 to -275 mV
- Low corrosion (10% risk of corrosion) > -125 mV

The paper [14] noted that corrosion of steel in concrete typically occur in the potential range from -450 mV to -600 mV with respect to the SCE.

The electrical resistivity of concrete is being used indirectly to evaluate concrete characteristics such as the chloride ion diffusivity, the degree of concrete saturation and its aggressiveness. The electrical resistivity of concrete is inversely proportional to the corrosion rate. The resistivity of concrete is strongly dependent on the concrete quality and on the exposure conditions, such as the relative humidity and degree of concrete pore saturation.

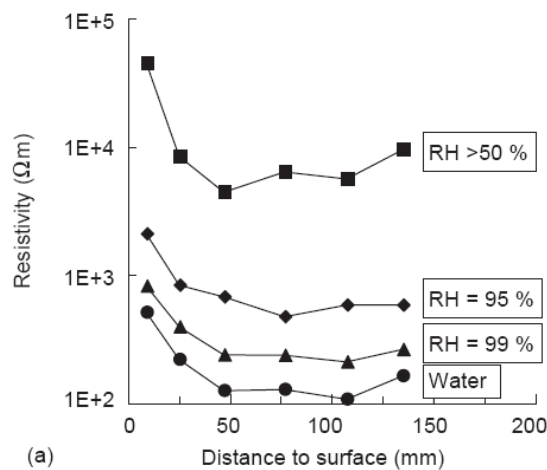


Fig. 3 Resistivity of a concrete core as function of the storage conditions (relative humidity RH) [7]

According to investigations performed by a number of authors in the paper [31] corrosion risk ranges from resistivity are presented as the following:

- Negligible corrosion risk  $> 20.000 \Omega\text{cm}$ .
- Low corrosion risk  $10.000 - 20.000 \Omega\text{cm}$ .
- High corrosion risk  $5.000 - 10.000 \Omega\text{cm}$ .
- Very high corrosion risk  $< 5.000 \Omega\text{cm}$ .

Tremper et al. [33] investigated a structure in a marine environment and stated that resistivity of  $60.000 \Omega\text{cm}$  is required to inhibit or prevent corrosion. Saturated concrete has resistivity in the range of 10 to  $60 \Omega\text{m}$  and in moist concrete it can be up to  $100 \Omega\text{m}$  [26]. The resistivity increases one hundred fold as the relative humidity falls from saturation to 50 per cent. Resistivity low enough to cause problems is of the order of 50 to  $100 \Omega\text{m}$ , which is typical of concrete in moist environments [22].

Linear Polarization Resistance Measurement (LPR) is used to measure corrosion rate. In LPR measurements the reinforcing steel is perturbed by a small amount from its equilibrium potential. This can be accomplished by changing the potential of the reinforcing steel by a fixed amount  $\Delta E$  and monitoring current decay  $\Delta I$  after a fixed time or contrary.

$$R_p = \Delta E / \Delta I \quad (2)$$

$$I_{corr} = B / R_p \quad (3)$$

$$Corr.rate = \frac{0.129 \cdot I_{corr} \cdot E.W.}{dA} \quad (\mu\text{m} / \text{yr}) \quad (4)$$

where is:

$B$  – constant value in range 10-30 mV, according to [25] equal to 26 mV

$I_{corr}$  – the corrosion current intensity ( $\mu\text{A}/\text{cm}^2$ )

$E.W.$  – the equivalent weight of steel (atomic weight of an element that has the same combining capacity as a given weight of another element, where the standard is 8 for oxygen)

$d$  – the density of the reinforcing steel ( $\text{g}/\text{cm}^3$ )

$A$  – exposed surface area of the reinforcing steel (cm).

According to [2] ranges of corrosion current  $I_{corr}$  for condition of the rebar are:

- Passive condition –  $I_{corr} < 0.1 \mu\text{A}/\text{cm}^2$
- Low to moderate corrosion –  $I_{corr} = 0.1-0.5 \mu\text{A}/\text{cm}^2$
- Moderate to high corrosion –  $I_{corr} = 0.5-1.0 \mu\text{A}/\text{cm}^2$
- High corrosion rate –  $I_{corr} > 1.0 \mu\text{A}/\text{cm}^2$
- No corrosion expected –  $I_{corr} < 0.2 \mu\text{A}/\text{cm}^2$
- Corrosion possible in 10-15 years –  $I_{corr} = 0.2-1.0 \mu\text{A}/\text{cm}^2$
- Corrosion expected in 2-10 years –  $I_{corr} < 1.0-10 \mu\text{A}/\text{cm}^2$
- Corrosion expected in 2 years –  $I_{corr} > 10 \mu\text{A}/\text{cm}^2$

A corrosion current density such as  $100 \text{ mA}/\text{m}^2$ , could theoretically lead to an annual loss of approximately one kilogramme per metre squared of metal [28].

Galvanostatic Pulse transient Method, Electrochemical Impedance Spectroscopy, Harmonic Analysis Method, Electrochemical Noise Analysis and Embeddable Corrosion

monitoring sensor are used in addition to previously described methods. Coronelli-Parrott (1994) [12] found that the corrosion depth that causes visible damage is about 100  $\mu\text{m}$ . The corrosion rate varies from about 0.3  $\mu\text{m}/\text{year}$  at 50% of relative humidity to a maximum of about 50  $\mu\text{m}/\text{year}$  at 98% of relative humidity (Fig.4). Dependence of corrosion rate and the concrete conductivity is shown in Figure 5.

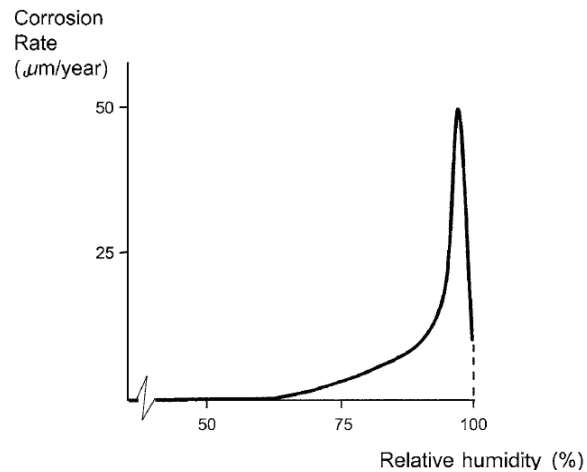


Fig. 4 Influence of relative humidity on corrosion rate [26]

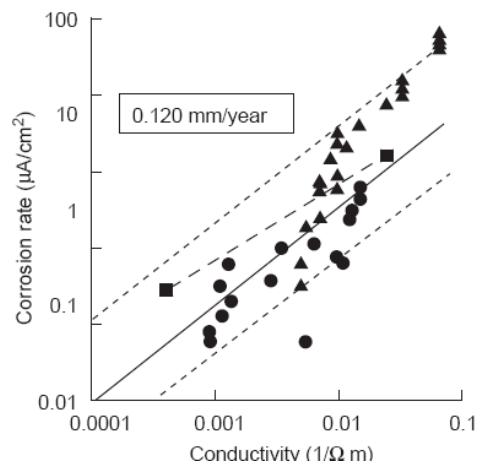


Fig. 5 Corrosion rate as a function of the conductivity of concrete [7]

The rate of corrosion increases with increasing temperature (Fig.6). In the paper [8] Browne reported that an increase in temperature from 20°C to 40°C in not humid climates could increase the rate of corrosion by a factor of five.

The temperature dependency of the corrosion rate is given by equation,

$$i_{corr} = i_0 e^{b_{i,corr} \left( \frac{1}{T} - \frac{1}{T_0} \right)} \quad (5)$$

where

$I_0 = i_{corr}$  with  $T_0$  ( $T_0$  approximately between  $-25$  and  $-40^\circ\text{C}$ )

$T, T_0$  = absolute temperature in K

$b_{i,corr}$  = constant in K.

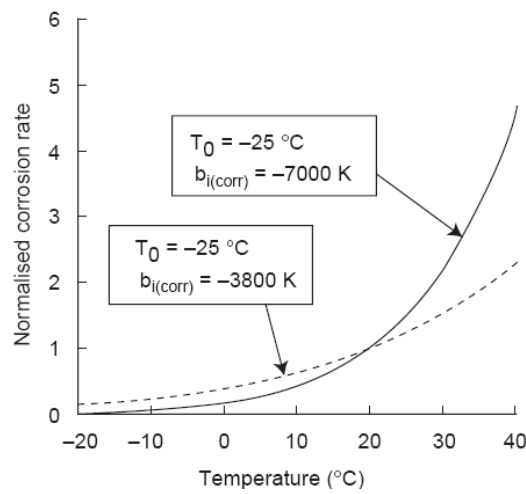


Fig. 6 Influence of the temperature on the corrosion rate (normalised to  $20^\circ\text{C}$ ) [7]

Presented overview of the values of certain parameters relevant to evaluating the structure and experience of research on structures in Tuzla industrial zone, presented in [16], [17] and [20], were used as a database of rating of individual structures in the Tuzla industrial zone. Some results of structural rating are presented in this paper, and they are discussed in details in the paper [16].

## 5. CASE STUDIES IN THE TUZLA INDUSTRIAL ZONE

Complexity of the problem of durability of the structure arises in very aggressive environments as the industrial zones. Tuzla happens to be one of such environments in which the heavy industry is concentrated and in which the environmental aggressiveness represents a substantial problem for reinforced concrete structures and demands constant investments in reparations of structures. By analysis of air and water pollution in industrial zone of Tuzla, presence of chemical compounds that are harmful for both concrete and reinforcement (i.e.,  $\text{CO}$ ,  $\text{CO}_2$ ,  $\text{SO}_2$  and  $\text{SO}_3$ ) is discovered. While sampling of concrete from structures of the industrial zone, it is established that the main cause of corrosion in concrete is the presence of chloride, while in the same time the sulphate content increased in small amount as well. The consequences of such reactions are shown on Figures 7, 8, 9 and 10.



Fig. 7 Nitrogen factory inside of Coke industry



Fig. 8 Salt factory in Tuzla



Fig. 9 Cooling tower mantle



Fig. 10 Beam for sprinkler installation

Structures in the Tuzla industrial zone are 40-50 years old. During the inspection of the structures in Tuzla industrial zone (Power Plant, Salt Factory, Coke Industry) (Fig. 11 and 12) the following damage was identified (Fig.7-10):

- On the structural elements directly exposed to water or high percentage of moisture layers of algae and mud were observed.
- The protective antirust covering was destroyed on almost all structural elements. The depth of the damage reached 30 mm and more. At places with deeper damage, the reinforcement corroded.
- On most of the columns surface damage was identified like concrete abrasion, drop-off of the protective reinforcement layer, direct exposure of reinforcement to corrosion; sporadic decrease of the reinforcement's diameter.



- On some of the beams, the upper part of the concrete was damaged and washed-off; on the lower part stirrups were visible and corroded (thin cover).
- Cracks in concrete and pitting and delaminating of protective concrete cover were observed in some structural elements.

So, reinforced concrete and steel structures for industrial facilities, aged from 40 to 50 years, are significantly damaged and is currently in the pool of Tuzla, allocated significant funds for rehabilitation. The experience of prior conducted rehabilitation indicates that its efficiency and rationality depends on the quality of improvement program, which must include:

- Field and laboratory research,
- Making optimized design solution,
- A detailed program of works (construction technology)
- Quality assurance program of the works.

These items are described in detail in the papers [17] and [20].



Fig. 11 Cooling tower, Thermoelectric Power Plant in Tuzla



Fig. 12 Plant 4, Tuzla salt factory

Adequate structural rating is necessary for optimized structural rehabilitation procedures which will cover in detail all affecting parameters specified in item 3. Such an assessment was performed for the structural elements of the cooling tower power plant in Tuzla. The results of the structural assessment are observed in details in the paper [16].

The cooling tower's entire structure is made of reinforced concrete. Damage was castred (mapping) and a program of extraction of concrete samples was designed. The programme comprises drilling out more than 200 samples of concrete cylinders.

The following chemical investigations were accomplished on the cooling tower:

- Deep penetration of chloride and its quantitative determination
- The pH value;
- Quantitative determination of sulphates; and
- The depth of carbonation.

Chemical research were performed with in situ indicators of the colour and drilled out samples of concrete in the laboratory. In situ investigation is also used to determine the location of concrete samples. The presence of chloride was determined through the application of the colour indicator of two chemical mixtures of 1% AgNO<sub>3</sub> and 5% K<sub>2</sub>CrO<sub>4</sub>, while the presence of carbonization process was determined using 0.1% of phenolphthalein (pH > 9 solution). The amount of chloride and sulphate in samples of drilled out concrete was determined in laboratory testing. The amount of chloride was determined by filtering the sample in distillate water with K<sub>2</sub>Cr<sub>2</sub>O<sub>7</sub> solution, which is subsequently added to the standard solution of AgNO<sub>3</sub>. The amount of sulphate was determined by filtering which considered using the solution of HCl and BaCl<sub>2</sub>.

Concrete of cooling tower was made of Portland cement with the addition of up to 30% slag and fly ash, and up to 5% gypsum. Cement clinker contains: 50-60% C<sub>3</sub>S, 15-25% C<sub>2</sub>S, <25% (C<sub>4</sub>AF and C<sub>3</sub>A) and 1-3% free CaO. Strength class of cement is 45.

On spots with smaller or larger damage, there is a difference in average compressive strength of the concrete. On spots with smaller damage, the average compressive strength ranges between 27.2 and 54.2 MPa, while on spots with larger damage it is between 26.3 and 42.0. Investigation results show that the depth of chloride penetration is up to 20mm. On some spots large chloride contents are identified in samples, up to 0.136%. The slight increase of sulphate content (up to 1.39%) indicates that there is a small amount of concrete degradation created by the water's sulphate content or the air's sulphur-hydrogen content. The depth of carbonation on the cooling tower's columns was determined in situ, while the depth of the shell's carbonation was determined by means of samples. In places with visible damage of structure the samples of drilled out concrete were taken, and afterwards the strength and chemical testing was carried out.

Mechanical properties of concrete are determined by using the combination of non-destructive method (Schmidt Hammer) and destructive methods (samples of concrete). Results of preliminary performed tests with the Schmidt Hammer and testing samples of drilled out concrete were used for correlation analysis. Then, an extensive testing with the Schmidt Hammer was performed, and the results are corrected with the correlation factor. Statistical analysis of research results was made so that the structure was divided according to the types of structure elements. Based on the investigation results of cores taken from the cooling tower's shell it was found that carbonation depth on the shell is up to 15mm. This indicates low alkalinity of the concrete, thereby not protecting the reinforcement against corrosion. The pH values of investigated core samples range from 9.5 to 11.5. From the enclosed results it can be seen that the amounts of soluble chlorides, sulphate contents, carbonation depth and pH values in some samples are sufficient for arising corrosion of concrete and reinforcement.

The designed strength of shell and sprinkler concrete of the tower of Thermoelectric Power Plant Tuzla is 40 MPa. After 50 years of utilization, the probability that the strength of the concrete will be smaller than 40 MPa is: for Tower shell  $P(f_{ck} < 40) = 0.6914$ ; for Sprinkler  $P(f_{ck} < 40) = 0.838913$ . The compressive strength, a 5% - fractal value, is: for Tower shell  $f_{c, 0.05} = 18.12$ ; for Sprinkler  $f_{c, 0.05} = 13.86$ . The coefficient of variation of the samples under investigation is: for Tower shell  $V = 40.55\%$ ; for Sprinkler  $V = 32.96\%$ . When compared to the coefficients of variations of "sound" concrete, these values are significantly larger.

The analysis of conditions and performance show a high degree of deterioration and the need for interventions. It is important to emphasize that the condition and performance rating of individual elements provide a preliminary response to the need for remediation of certain structure elements and their capacity and serviceability.

For rating of cooling tower structure Condition Rating Method proposed by CEB [10] and Performance Rating Method proposed by fib Bulletin 44: Concrete structure management [19] were applied. Results of analysis are presented in tables 4, 5, 7 and 8 [16].

According to Condition Rating Method structural elements are classified as Class IV, Class V and Class VI (see Table 1) [10], which means that it is necessary to perform emergency interventions on the structure.

For the  $V_R$  coefficient experimental value 32.96% is adopted, reliability coefficient  $\beta_c = 3.8$  (common value according to EC0 [13]) and  $\alpha_R = 0.6$  (see Table 1).

$$\Phi = B_R \cdot 0.472$$

The capacity reduction factor calculated from the local CR indicates residual capacity 47.2% of the non-corroded column design resistance. Herein is shown the worst case. Values for other structure elements range from 47.2% to 80%.

According to Performance Rating Method the values of Repair Performance Index in range from 2.50 to 3.23 were obtained. Target value for the intervention is 3.0 [15].

Condition and the performance analyses of structures in Tuzla industrial zone show a high degree of deterioration and the need for direct interventions. It is important to emphasize that the condition and performance rating of individual elements provides a preliminary response to the need for remediation of certain structure elements and their capacity and serviceability. Statistical parameters of the scattering of test results of concrete mechanical characteristics must be set aside as very important in the global analysis of structures. However, the global analysis of the structure must be separated as very important statistical parameters of scattering results of examination of the mechanical characteristics of concrete built in. Namely, the tested samples embedded in concrete of the bearing structure show a very large scattering of the coefficient of variation ( $V > 20\%$ ), which results in significant reduction of the value of the nominal capacity of cross section (Fig. 13).

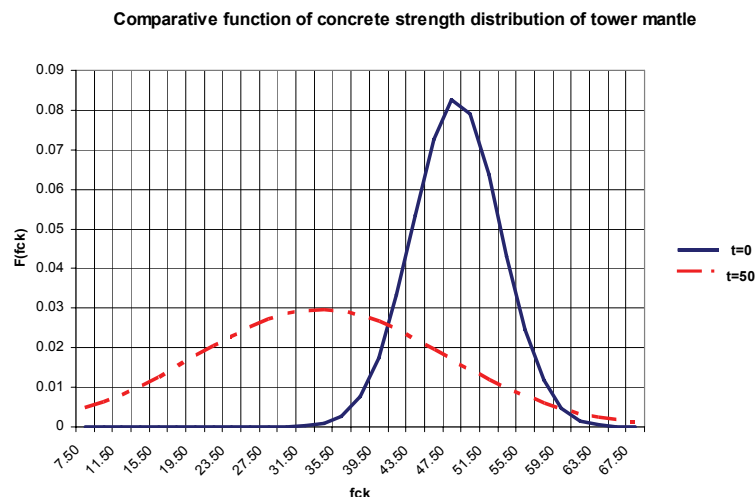


Fig. 13 Comparative function of concrete strength for shell structure elements at the beginning of exploitation and after 50 years [16]

## 6. CONCLUDING REMARKS

Concrete and steel, and all other materials, have limited durability, i.e. in the course of time there is a degradation of properties due to environmental actions. The interaction between the concrete and the environment determines possible deterioration mechanisms. Practical experience have shown that corrosion of the reinforcement and/or concrete are the main causes of degradation of the RC's physical and mechanical properties. Corrosion of the reinforcement is due to the carbonization of concrete and penetration of the chloride into it. Previous experiences indicate the lack of attention to the issue of durability of RC structures. It has been restricted to quantitative evaluation and construction codes (thickness of the protective concrete layer, controlling cracks), without considering the mechanisms of deterioration. Traditionally, designers estimate service life using judgment based on experience with similar structures in similar environments. The highly increased level of environmental pollution has led to acceleration of deteriorating processes in concrete structures, thereby decreasing their service life. Therefore, in addition to evidence of bearing capacity and usability, in calculations of engineering structures the evidence durability has been introduced. For this reason, mathematical models for quantification of deteriorating processes have been introduced. This issue has been analyzed in details in [19]. Therefore, an adequate methodological approach to this issue is necessary through control the structure's behavior over the time and optimum interventions in order to keep the maintenance costs low. In this sense, the methodology is based on the use of an empirical data base, in situ and laboratory experiments and verifications of the deteriorating process mathematical model.

Attention should be paid to the problem of durability of structures in all stages of their creation (design, construction and maintenance). Designing for service life will become increasingly important in the near future. Prediction of durability gives clearer view on extending service life. To achieve the highest level of reliability and desired durability designers must use adequate mathematical models. One of the necessary steps of the methodology of analysis of deteriorating processes is evaluation of the structure, its classification regarding the need for rehabilitation, as well as the extent of rehabilitation measures. Besides the ways of modeling the service life, this work presents two methods condition assessment: Condition Rating Method and Performance Rating Method. They are based on detailed analysis of influential parameters which are leading to degradation of structure properties. Experiences which have been resulted from evaluation of the RC structures (age around 50 years) in the industrial zone of city of Tuzla, have shown a significant decrease in the structures properties and in the most necessary urgent interventions. Besides the aggressive influence from the environment, the reason in this particular case was the inadequate maintenance program during the structure's operation, i.e., the inadequate methodological approach to the issue. During the research works on the structures in industrial zone of Tuzla, a large number of concrete samples were inspected. A huge diffusion of results was identified with coefficients of variation over 20%, even up to 40%, which significantly differs from the inspection of "healthy" samples, with diffusion of up to 15%. Such statistical data are leading to significant decrease of statistical fractal value of 5%, which is the usual nominal value. This represents an additional issue in verification of engineering structures with lack of adequate maintenance program.

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## **PROJEKTOVANJE BETONSKIH KONSTRUKCIJA SA ASPEKTA TRAJNOSTI – DEO 2: MODELIRANJE I PROCENA KONSTRUKCIJE**

**Radimir Folić, Damir Zenunović**

*Ovaj rad predstavlja nastavak rada [19], u kojem su date teoretske osnove za analizu trajnosti betonskih konstrukcija. U radu je dat pregled nekih pristupa modeliranja trajnosti armiranobetonskih konstrukcija. Detaljno su opisana iskustava sprovedenih eksperimentalnih i numeričkih istraživanja pojedinih uticajnih parametara, koji dovode do korozije armature, prslina i delaminacije okolnog betona, što dovodi do smanjenja eksploatacionog veka elemenata RC konstrukcije. Ova analiza je neophodan korak za adekvatan pristup oceni konstrukcije. Metodologije procene konstrukcije, prezentirane u radu, zasnivaju se na oceni ključnih uticajnih parametara i predstavljaju metodologiju za racionalno održavanje građevinske konstrukcije, odnosno racionalan eksploatacioni vek konstrukcije. Date su metode zasnovane na analizi uslova i performansi (Condition Rating Method, Performance rating Method). Na kraju rada dati su rezultati procene betonskih konstrukcija u industrijskoj zoni grada Tuzle.*

*Ključne reči: Betonske konstrukcije, trajnost, modeliranje, korozija, uticajni parametri, ocena konstrukcije, klasifikacija konstrukcije, performanse, realno stanje, deterioracija*