dgins
DRUŠTVO GRAĐEVINSKIH INŽENJERA NOVOG SADA

SAVREMENA GRAĐEVINSKA PRAKSA
CONTEMPORARY CIVIL ENGINEERING PRACTICE
2022

DEPARTMAN ZA GRAĐEVINARSTVO I GEODEZIJU
FAKULTET TEHNIČKIH NAUKA NOVI SAD

Ivan IGNJATOVIĆ[1]

PRORAČUN UPOTREBNOG VEKA BETONSKIH KONSTRUKCIJA PREMA FIB MODELU PROPISA – KONCEPT I PRAKTIČNA PRIMENA

SERVICE LIFE DESIGN OF CONCRETE STRUCTURES ACCORDING TO FIB MODEL CODE – CONCEPT AND PRACTICAL APPLICATION

Rezime: *U radu je prikazan koncept i procedura projektovanja betonskih konstrukcija s obzirom na njihov upotrebni vek. Ovim konceptom koji se bazira na pouzdanosti, proračun trajnosti se podiže na nivo proračuna prema graničnim stanjima nosivosti i upotrebljivosti. Procedura proračuna sastoji se iz tri koraka – prepoznavanje deterioracionog mehanizma koji može najviše uticati na konstrukciju, definisanje graničnog stanja prema kome se projektuje i verifikacija, tj. proračunski dokaz odgovarajućeg graničnog stanja. Svi parametri fizičkog i/ili hemijskog modela deterioracionog procesa treba da budu kvantifikovani i statistički opisani kako bi bilo moguće da se nekim od probabilističkih pristupa izvedu proračunski dokazi. Koncept proračuna demonstriran je na primeru jednog deterioracionog mehanizma- karbonatizacije i izabranog graničnog stanja – depasivizacija armature. Opisani su svi parametri modela i komentarisan njihov uticaj. U drugom delu rada izneti su primeri proračuna prema upotrebnom veku konstrukcija iz prakse. Prikazane su dve situacije – proračun upotrebnog veka novih konstrukcija i ažuriranje preostalog upotrebnog veka postojeće konstrukcije.*

.

Ključne reči: *betonske konstrukcije, trajnost, upotrebni vek, pouzdanost, deteriorizacioni mehanizam, probabilistički pristup*

Abstract: *Concept and procedure of service life design of reinforced concrete structures is presented in the paper. The aim of this concept based on probability is to bring the durability design at the same level with the design to ultimate limit state or serviceability limit state. The design follows the three-step procedure – diagnosis of the deterioration mechanism that would have the greatest impact on structure, definition of the limit state for design and verification of a limit state. All the parameters of the model, both on the load side (the environmental actions) and on the resistance side (the resistance of the concrete against the considered environmental actions) must be statistically described and quantified, to enable verification of considered limit states with probabilistic methods. Concept of service life design is demonstrated with selected deterioration mechanism – carbonation of concrete and selected limit state- depassivation of reinforcement. All parameters are described and their impact is assessed. Numerical examples, i.e. several case studies are presented in the second part of the paper. There are two situations – service life design of a new structures and assessment of the rest of service life of existing structure.*

Key word: *concrete structures, durability, service life, reliability, deterioration mechanism, probabilistic method*

[1] V.prof, Univerzitet u Beogradu-Građevinski fakultet, Bulevar kralja Aleksandra 73, 0113218546, ivani@imk.grf.bg.ac.rs

1. INTRODUCTION

Durability design of reinforced concrete structures given in the current codes can be assessed as prescription-based or experienced-based approach. In the code BAB87 [1], for example, values for parameters are based on qualitative analysis of concrete durability such as minimum concrete cover or maximum crack width. In European (and new national) code and standard SRPS EN 1992-1-1:2015 [2] and SRPS EN 206-1:2011 [3] apart from these parameters, several others like maximum water to cement ration, minimum cement quantity and minimum percentage of air-entered have also been taken into account. With this approach structures will probably have acceptably long (at least 50 years [2]), but unspecified service life, without possibility to choose or perform design for longer/shorter service life. In both codes, rules that define durability parameters are based on simplified classification of environment to which the structures are exposed. Some rules are not adequate for some aggressive environments while some are too rigorous in non-severe environment. In practice, designer choose values from code's tables without knowing the background of these numbers. Thus, traditional concept of durability design is without quantification of exposure conditions, without specified service life, without knowing which limit states are going to be achieved and without knowledge about the base for experienced-based recommendations. Durability requirements are given implicitly and in general, durability design is considered as of secondary importance part of structural design.

In order to improve this part of design and having in mind that a majority of reinforced concrete (RC) structures being constructed between 50s and 70s have expired service life, with evidenced damages of concrete and long lasting corrosion, the new approach have been proposed by fib in Model Code for Service Life Design [4] and implemented in fib Model Code for Concrete Structures [5]. This is probabilistic base approach of durability design and it enables the design based on reliability and performance of structure, in similar way like the traditional capacity design. Service life design given in [4] can be used for the design of new structures or assessment of the rest of service life of existing structure for which the actual material characteristics are known and interaction with the environment can be quantified.

Concept of service life design of reinforced concrete structures is describe in three-step procedure and presented in this paper. Model for one deterioration mechanisms that lead to the corrosion of reinforcement is described with its parameters and assessment of their impact to the result is given. Three case studies with practical application of presented methodology are also presented.

2. SERVICE LIFE OF RC STRUCTURES

Service life assumed period for which a structure or a part of it is to be used for its intended purpose [4]. It is defined with the time given in years and the reliability level that the certain limit state will not be reached during that period and it is so called technical service life. Apart from this, terms such as functional service life and economic service life are also used, but only technical service life will be considered herein.

Generally, the service life of concrete structural elements consists of two phases – initiation phase and propagation phase, figure 1. Initiation phase is defined as a period of time with interaction between concrete and environment and the end of this phase is the moment of depassivation of reinforcement. Depassivation can occurred when the carbonation front reach the depth of reinforcing bar position or in case the chloride content in concrete at that depth reach the critical value. Depassivation means that under certain conditions, i.e. presence of moisture and oxygen, corrosion is possible.

During the propagation period depassivised bar is damaged by corrosion causing the decrease of bar diameter, cracks and spalling of concrete cover as a consequence of expanding volume of corrosion products. The service life is usually assumed as a period until the end of initiation period, i.e. beginning of corrosion of reinforcement, Figure 1. But, a part of propagation period can be also taken in the range of service life if it is accepted by owner, knowing the expected level of structural damage by the end of that period, e.g. cracks formation or spalling of concrete. The role of owner is important as the defined service life correlates with the level of investments. Taking the part of propagation period in service life is related to lower initial cost (compared to the situation in which the same duration of service life occupies only initiation period), but will result in a higher costs for structural repair at the end of service life. It is also important to assure that the level of maintenance work (and its cost) be acceptable,

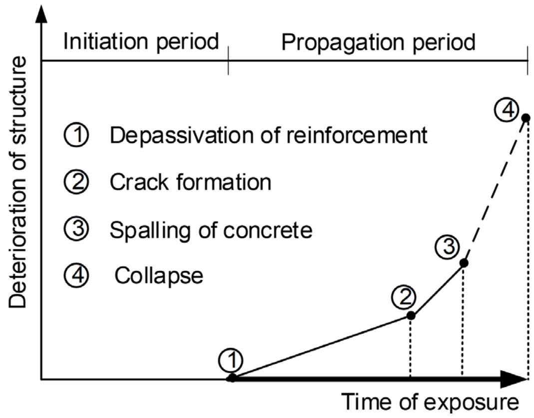

Figure 1 – Two-stage Tuutti model for service life Adapted from [4]

otherwise, the end of technical service life will be achieved before it is designed for.

3. DETERIORATION MECHANISMS

Durability of concrete is usually assessed with the degree of concrete damage caused by chemical reaction. In order to have reaction, transport of ions or molecules of aggressive substance from environment to the inner part of structural element and reactive substance is needed. If there is no transport, there is no reaction. Both the transport and reaction occur during the time causing not instant damage of concrete but damage that propagate during the time, i.e. deteriorate.

Deterioration mechanisms can be generally devided in two groups:

- corrosion of reinforcement and prestressed cables
- deterioration of concrete

Two main mechanisms that lead to the corrosion of reinforcement are: 1) carbonation of concrete and 2) chloride ingress. Of course, combination of these processes can also be detrimental for reinforcing bars, but there is still no reliable model for this coupled process. On the other hand, deterioration of concrete is caused either by the composition of concrete mixture either by the environmental impact. There are numerous mechanisms that caused deterioration of concrete such as: freeze-thaw cycles, alkali-aggregate reaction, sulfate attack, delay of etringite formation, microbiological attack.

fib Model Code [4] considers four deterioration mechanisms and offers probabilistic models for the following ones:

- corrosion due to carbonation
- corrosion due to chloride penetration
- frost induced internal damage
- salt-frost induced surface scaling

Within this paper, as a demonstration of the concept of service life design, corrosion due to carbonation will be analized. Analytical model and its parameters as well as examples from practice are presented.

4. DESIGN PROCEDURE

The three-step procedure in service life design of reinforced concrete structures is shown at Figure 2. First step is to choose and quantify the deterioration mechanism. Although a double or even triple deterioration mechanisms can act on the concrete in the same time, the one that is the most severe thread to the observed element/structure has to be chosen. Quantification is performed by physical or chemical model that describes the chosen deterioration process with certain (acceptable) precision. These models describe propagation of damage or degradation through the time, i.e. deterioration. Acceptable precision means that model should be verified by laboratory experiments and in situ observations, so that mean values and standard deviations for material (concrete) properties are known and taken by the model. Furthermore, these models should take into account the environmental impact taken by statistically defined parameters such as temperature, relative humidity, precipitation etc. For example, model for carbonation induced corrosion is based on diffusion as the prevailing transport mechanism within the concrete - Fick's 1st law of diffusion, while the model for chloride induced corrosion is based on Fick's 2nd low of

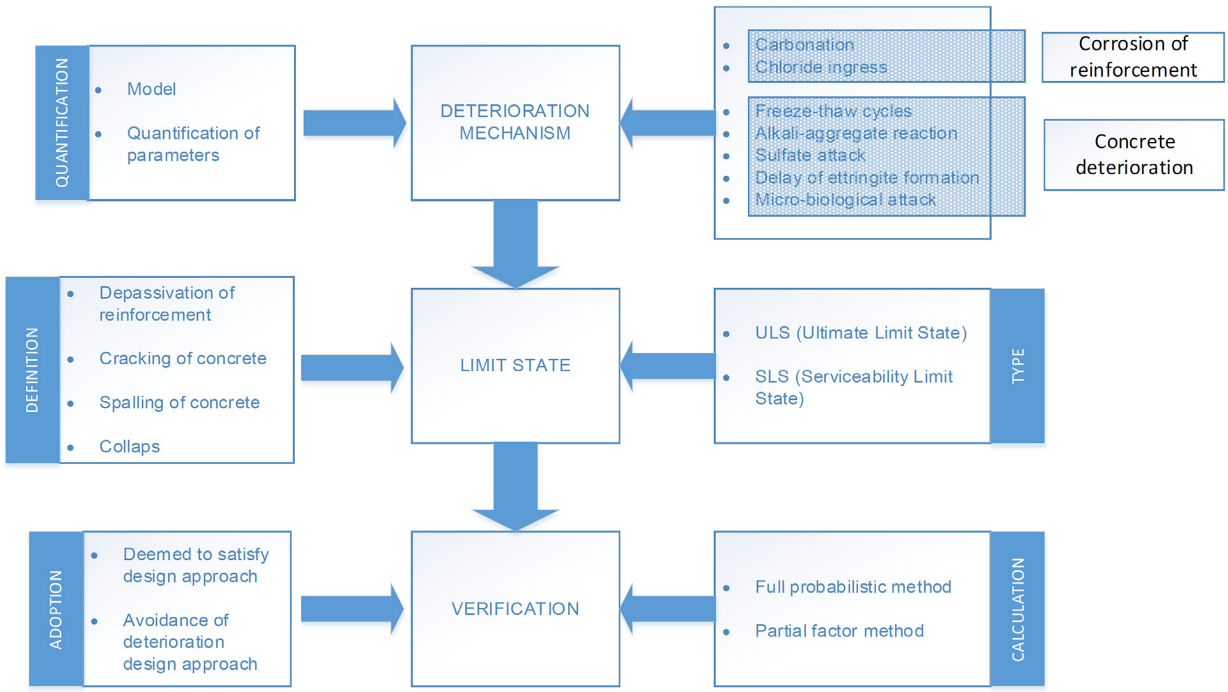

Figure 2 – Scheme of three-step procedure for service life design of RC structures

diffusion. All parameters in models of deterioration mechanisms should be quantified and for selected one it is done in section 5.

Second step is to define limit state for which the element or structure is designed and to associate the type of limit state. Limit states depend on the deterioration mechanism accompanied to that element/structure. Some of them are: depassivation of reinforcement due to carbonation, spalling of concrete cover or concrete cracking due to the reinforcement corrosion or freeze-thaw cycles, failure of structure due to the loss of bar cross section, etc. It means that each of the key points given in Figure 1 accompanied with the mechanism that caused it can be defined as a limit state.

Almost each of the limit states given in second step can belong to one of the limit state type - serviceability or ultimate limit state. For example, in case there is no consequence on the safety of the structure if it occurs, limit state – spalling of concrete cover due to carbonation will be classified as serviceability limit state, Figure 3 left. But, in case of concrete spalling concrete followed by the loss of adhesion between concrete and reinforcement that can lead to the failure of structural element, it can be assessed as ultimate limit state, Figure 3, right. Of course, collapse of structure can be assessed only as ultimate limit state.

Based on the adopted limit state type, an adequate reliability index β is chosen which defines the target level of reliability. It is actually just another way to express probability of “failure”, Pf, i.e. probability that the certain limit state will not be reached. These values can be correlated with the inverse standard function of normal distribution ΦU-1 and numerical comparison is given in the table below. Value of β=3.72, for example, means one “failure” in 10,000 cases.

Depending of the limit state type and reference period (period of time that is used as a basis for assessing statistically variable actions and possibly for accidental actions), fib [5] generally recommended target reliability indexes β for structures to be designed, while for existing structures suggests to consider lower reliablity indexes. Values for minimum β given in [4] are in a good accordance with them, Table 2, although slightly lower both for SLS (1.3 compared to 1.5 [5]) and ULS

Figure 3 – Spalling of concrete as SLS (left) and ULS (right)

Table 1 - Relationship between probability of failure (Pf) and reliability index (β)

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1,28	2,32	3,09	3,72	4,27	4,75	5,2

Table 2 - Recommended values for reliability index β for use in SLD

Exposure class	Limit state	Reliability class	SLS Depassivation	ULS Collapse
XC3	Carbonation induced corrosion	RC1	1.3 (p_f≈10^{-1})	3.7 (p_f≈10^{-4})
		RC2	1.3 (p_f≈10^{-1})	4.2 (p_f≈10^{-5})
		RC3	1.3 (p_f≈10^{-1})	4.4 (p_f≈10^{-6})
XD3	Chloride induced corrosion	RC1	1.3 (p_f≈10^{-1})	3.7 (p_f≈10^{-4})
		RC2	1.3 (p_f≈10^{-1})	4.2 (p_f≈10^{-5})
		RC3	1.3 (p_f≈10^{-1})	4.4 (p_f≈10^{-6})
XS3	Chloride (from sea water) induced corrosion	RC1	1.3 (p_f≈10^{-1})	3.7 (p_f≈10^{-4})
		RC2	1.3 (p_f≈10^{-1})	4.2 (p_f≈10^{-5})
		RC3	1.3 (p_f≈10^{-1})	4.4 (p_f≈10^{-6})

(3.1, 3.8 and 4.3 compared to 3.7, 4.2 and 4.4 [5]).

The third step is the verification, i.e. calculation of the probability that the limit states defined above will be reached and that can be done in two different ways. One is the calculation, where some of the parameters of deterioration model are on the load side (e.g. coefficient of diffusion, chloride migration coefficient, etc.) while some on the resistance side (e.g. thickness of concrete cover, critical chloride content, etc.) and that is the concept well known from load-resistance design. There are two methods for calculation that can be used:

- full probabilistic method
- partial factors method

General equation of probabilistic approach means that failure probability p {} (probability that the "load"- E(t) exceeds the "resistance"- R(t)) must be limited to a target probability p0 defined by the adopted reliability index β:

$$p\{\ \} = p_{dep} = p\{(R(t) - E(t)) < 0\} < p_0 \qquad (1)$$

In case of limit state "depassivation of reinforcement due to carbonation" eq. 1 is transformed in:

$$p\{\ \} = p_{dep} = p\{(a(t) - x_c(t)) < 0\} < p_0 \qquad (2)$$

where parameter R became the value of concrete cover (a), and parameter E is the value of carbonation depth (x_c). It has to be noted that, when durability of a structure is considered, both the "resistance" (R=a(t)), and "load" (E=x_c(t)) are functions of time and, in general, probability of "failure" increases with time, Figure 4. At the moment t=0 the density distribution of the load and the resistance are far apart and the failure probability is small at first. With time, the distributions approach each other, forming an overlapping area of increasing size. The overlapping area illustrates the failure probability which defines technical service life of structure, Figure 4.

If the partial factor method is used, one should prove that the designed resistance of structure (or its part), Rd, is higher than designed value of action, Ed, using the partial safety factors, which are calibrated to reach the adopted reliability index β:

$$R_d - E_d \geq 0 \qquad (3)$$

In case of the analyzed limit state this equation changes into:

$$a_d - x_{c,d}(t_{SL}) \geq 0 \qquad (4)$$

where ad is the design value of concrete cover and $x_{c,d}$ (t_{SL}) design value of carbonation depth.

The another branch for verification of limit state do not consider calculation and can be done by 1) "deemed to satisfy approach" or 2) prevention of deterioration process. Option 1 looks like durability requirements given in current codes but contrary to them, tabulated values for key durability parameters called durability indicators come from the physical or chemical models and they are derived from the full probabilistic approach and not only from experience and observations and measurements confirmed in the practice. It has been already proposed and used somewhere, Table 3. As the service life increases, the number of parameters, i.e. durability indicators increase and the demand is becoming more rigorous. This approach is expected to be a model for assessment of durability in the future generation of Eurocode 2. Option 2 means isolation of structure from the environment by the use of non-reactive materials such as stainless steel or aggregate that do not react with alkalis, by controlling the relative humidity in the vicinity of the structure, i.e. keeping it below the critical level, etc.

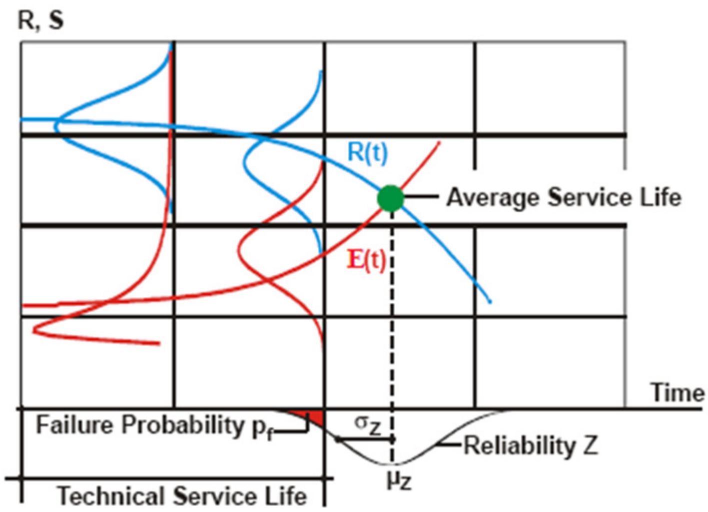

Figure 4 - Time dependant behavior of resistance R (t) and load E (t) during service life [6]

Table 3 - Durability indicators for a given service life and type of environment. Adapted from [7]

> 120 years *So-called exceptional structures* **Level 5**	from 100 to 120 years *Large structures* **Level 4**	from 50 to 100 years *Building & civil engineering structures* **Level 3**	from 30 to 50 years *Building* **Level 2**	< 30 years **Level 1**	← Required service life / *Structure category /* **Requirement level**	Type of environment →	
• $P_{water} < 9$ • $K_{gas} < 10$	• $P_{water} < 12$ • $K_{gas} < 100$	• $P_{water} < 14$ (6)	• $P_{water} < 16$	• $P_{water} < 16$	Dry and very dry (RH<65%) or permanently humid	1	**Carbonation-induced corrosion** (th = 30 mm)
• $P_{water} < 9$ • $k_{liq} < 0.01$	• $P_{water} < 12$ • $K_{gas} < 100$	• $P_{water} < 14$ (6)	• $P_{water} < 16$	• $P_{water} < 16$	Humid (RH>80%)	2	
• $P_{water} < 9$ • $K_{gas} < 10$ • $k_{liq} < 0.01$	• $P_{water} < 9$ • $K_{gas} < 10$ (4)	• $P_{water} < 12$ (7) • $K_{gas} < 100$ (8)	• $P_{water} < 14$ (5)	• $P_{water} < 15$	Moderately humid (65<RH<80%)	3	
• $P_{water} < 9$ • $D_{app(mig)} < 1$ • $K_{gas} < 10$ • $k_{liq} < 0.01$	• $P_{water} < 9$ • $K_{gas} < 10$ • $k_{liq} < 0.01$	• $P_{water} < 12$ (7) • $K_{liq} < 0.1$ (9)	• $P_{water} < 14$ (6)	• $P_{water} < 16$	Frequent wetting-drying cycles	4	

5. MODEL FOR CARBONATION INDUCED CORROSION

Numerical model for concrete carbonation is based on the 1st Fick's low of diffusion, originally given by Tuutti [8] in which carbonation depth (xc) is in linear relationship with the square root of time (t0,5), where the carbonation coefficient (kc) depends on both environmental conditions (CO2 concentrations, humidity etc.) and concrete properties:

$$x_c(t) = k_c \cdot t^{0,5} \tag{5}$$

This equation was further improved by separation of CO2 concentration as particular parameter (Cs) and keeping the rest of environmental factors and material properties all together, defined by the coefficient K:

$$x_c(t) = K \cdot C_s^{0,5} \cdot t^{0,5} \tag{6}$$

Further improvement of diffusion model in terms of carbonation was done by fib Task Group 5.6 [4]. Namely, fib model separates the influence of concrete properties through the inverse natural carbonation resistance (RNAC,0-1) and curing conditions (kcur), while the environmental impact was taken into account with environmental function (ke) and weather function (W(t)):

$$x_c(t) = 2 \cdot k_e^{0,5} \cdot W(t) \cdot k_{cur}^{0,5} \cdot \left(R_{NAC,0}^{-1}\right)^{0,5} C_s^{0,5} \cdot t^{0,5} \tag{7}$$

The intention of model for service life design dominantly was to serve for the design of new structures so designer should know natural resistance of concrete in advance, i.e. before the structure has been constructed. Hence, the inverse carbonation resistances obtained under natural conditions (RNAC,0-1) was related to inverse carbonation resistances obtained under accelerated conditions (RACC,0-1):

$$R_{NAC,0}^{-1} = k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t \tag{8}$$

where kt is regression parameter and εt is error term.

Finally, the equation for prediction of the carbonation depth under natural exposure conditions can be written as:

$$x_c(t) = \sqrt{2 \cdot k_e \cdot k_{\mathrm{cur}} \cdot \left(k_t \cdot R_{\mathrm{ACC,0}}^{-1} + \varepsilon_t\right) \cdot C_s \cdot t} \cdot W(t) \tag{9}$$

Once the model is defined it is important to emphasize that the role of cracks, which commonly occur in concrete, is neglected in the proposed models, not only for carbonation but for all others. It was suggested that models are reliable in case of uncracked concrete or cracked concrete if the crack widths are within the limits given in codes and used for serviceability limit state check. However, in recent study [9], it was showed that the effect of crack on carbonation and reinforcement corrosion can be more significant than any other parameter of deterioration mechanism.

However, with eq. (9), carbonation depth at certain point of time can be calculated implying quantification of each parameter and that was the first step in the service life design procedure, Figure 2. The second step is to define limit state and that is adopted as "depassivation of reinforcement" treated as serviceability limit state. The consequences of such a choice are reliability index β=1.3 and assumption that the calculated service life will be the end of initiation period, Figure 1. The third step is verification and in this case full probabilistic approach is chosen while other options will be commented also. Obviously, with so many stochastic parameters probabilistic calculation is quite complicated. Therefore, as an instrument for application of probabilistic approach some of commercially available software such as Strurel is need to be used.

5.1. Environmental function ke

The environmental function k_e takes into account the influence of the humidity level on the diffusion coefficient and hence on the carbonation resistance of the concrete. The environmental function ke is described according to the following equation:

$$k_e = \left(\frac{1 - \left(\frac{RH_{real}}{100} \right)^{f_e}}{1 - \left(\frac{RH_{ref}}{100} \right)^{f_e}} \right)^{g_e} \qquad (9)$$

where RH_{real} is the relative humidity of the carbonated layer [%], RH_{ref} is the reference relative humidity- 65% , fe and ge are exponents with recommended constant values 5.0 and 2.5, respectively [4]. Because the carbonation process takes place from the surface of the concrete element, it seems reasonable to use relative humidity of the ambient air near the structure (RH) instead of relative humidity of the carbonated layer, which is very difficult to obtain [4], so RH_{real} is equal to RH taken from the nearest weather station.

If probabilistic approach is used, RH is a stochastic value for certain location, represented usually with beta distribution function which is defined by upper and lower limits. As an example, RH data from weather station in Belgrade for 50 years period (1960-2010) have been taken. Minimal monthly value of relative humidity was 44.4%, maximal 88.9%, overall average of 68.5% and standard deviation of 8.37%. These data are parameters of beta distribution function, Figure 5. Together with other (constant) values of parameters RH_{ref}, f_e, g_e, environmental function ke is defined as stochastic value and can be used in probabilistic calculations.

Instead of probabilistic method, partial factor method can be also used for verification of analyzed limit state. In that case, partial factor for environmental function must be introduced, usually taken as γRH=1.3, which divide characteristic value of relative humidity (RHk) so the design value is obtained. If modified in that way, function ke can be ploted, Figure 6.

One should note that this diagram does not fully represent the influence of relative humidity on carbonation. Namely, there is a plateau for almost all values of RHk lower than 50% which cannot not be representative having in mind that the diffusion and consequently carbonation in dry condition is rather low, similar to one in a wet conditions, i.e. for high RHk. However, it is on the conservative, i.e. safe side.

Analysis of collected data taken from Belgrade weather station showed that the average annual values of relative humidity vary in a narrow range between 62% and 73% for a period of 50 years. If we consider them as representative (characteristic) values, function ke will return values equal to 1.28 and 1.18, respectively, so the influence of relative humidity variation can be roughly estimated as 8%. Probability to obtain values for ke higher than 1.28 is 0.00174 (β=2.92), which means it can be hardly happen, while to be higher than 1.18 is 0.0801, i.e. 8% (β=1.4) which can be considered as similar to the reliability demanded for serviceability limit state. It means that the value of ke=1,18 can be used as upper limit value, for analyzed set of data.

5.2. Execution transfer parameter, k_c

The execution transfer parameter kc takes into account the influence of curing on the effective carbonation resistance. Based on the regression analysis on the data collected during the DARTS project [4], effect of curing is recommended as:

$$k_c = \left(\frac{t_c}{7} \right)^{b_c} \qquad (11)$$

- k_c - execution transfer parameter [-]
- b_c - exponent of regression (mean value μ= -0,567, standard deviation σ= 0,024) [4]
- t_c - period of curing [day]

The reference value of curing period is 7 days which gives the unit value for execution transfer parameter. Influence of concrete curing period is enormous - decreasing the curing period leads to significant increase in value of execution function. Therefore, in case of 1 day of curing, the value of parameter kc is even 3 times higher than in case of 7 days curing period leading to increase of carbonation depth and consequently design concrete cover for 73%. Indicated by former national code BAB'87 [1], curing period in normal condition is 3

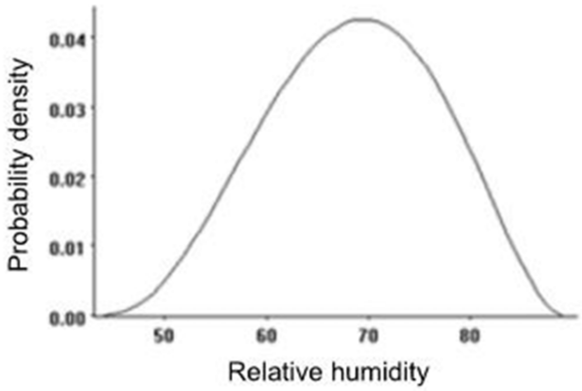

Figure 5 - Beta probability density function for relative humidity

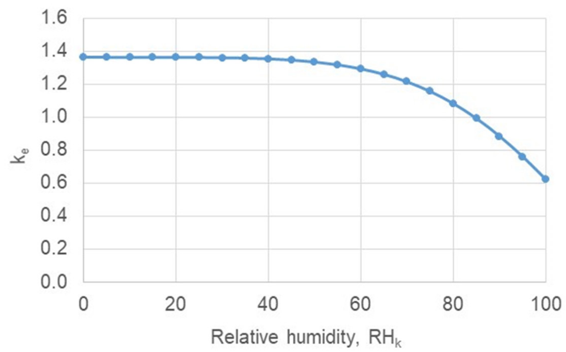

Figure 6 - Environmental function, k_e, related to characteristic value of relative humidity, RH_k

days, while in hot conditions is 7 days. Based on experience and author's knowledge, in our engineering practice, duration of curing period is in range between 2 and 3 days. Even minimum difference of one curing day (in this interval of 2-3 days) leads to increase of the design concrete cover of 12%. Due to the fact that number of curing days can be prescribed value (by designer), this parameter may be taken as constant value. On the other hand, the influence of the variation of exponent of regression is rather small, Figure 7. Having in mind the property of normal distribution that about 95% of all the values lie within two standard deviations, except for a very short period of curing the influence of exponent is negligible.

5.3. Environmental impact C_s

The CO_2 concentration represents the direct impact of the environment on the reinforced concrete structures and, therefore, main trigger for carbonation process. Global data with regard to CO_2 concentration are presented on Figure 8. In a ten years period ago, global CO_2 content in the atmosphere has been increased from 392 ppm in 2011 to 416 ppm in 2021, which corresponds to concentration of 0.00071 up to 0.00075 kg/m^3. However, in urban environments even few times higher concentrations are expected, while in rural areas and seaside areas the concentrations are lower than global average [13]. The data for analyzed local area (Belgrade, Serbia), according to the author's best knowledge, are not available

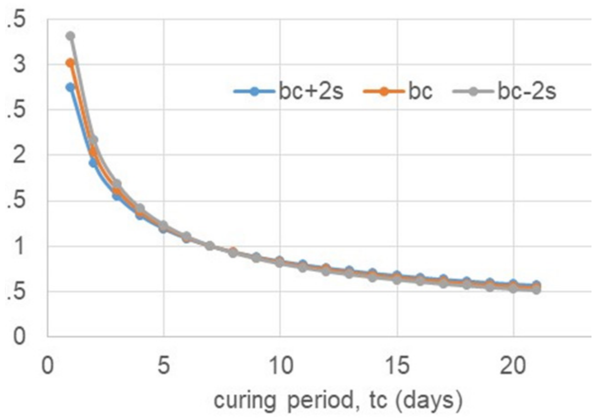

Figure 7 - Execution function, kc, related to curing period

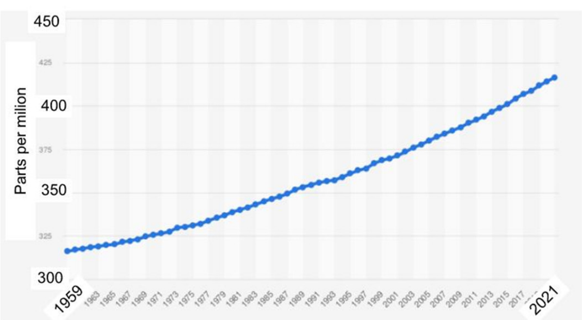

Figure 8 - Historic average CO_2 level. Adapted from Statista 2002

and, as a matter of fact, it does not exist at all. Estimation of CO_2 concentration for Belgrade could be based on the data obtained from urban areas in Europe featuring approximate population.

For buildings, the reference value of C_S is the CO_2 concentration in atmosphere of the local area around the analyzed structure. But, for tunnels, chimneys, constructions exposed to use of combustion engines (public garage etc.), value of C_S is obtained by adding the concentration of CO_2 due to emission source to the CO_2 concentration in atmosphere. This is very important because the total concentration of CO_2 can be even 10 times higher than the average concentration in atmosphere, which enlarges carbonation depth up to around 3 times.

Estimated increase rate in CO_2 concentration in the atmosphere is around 1.5 ppm/year which, for design service life of i.e. 50 or 100 years, would give expected values of global concentration of 0.00088 kg/m^3 and 0.00109 kg/m^3, respectively. These mean values of concentration at the end of service life are commonly used for calculation. Doing so, design is on the safe side, although the change of concentration during service life could be also taken into account through the characteristics values. Nevertheless, the impact of whether taking mean or characteristic values, is less than impact of actual estimated concentration of CO_2. Recommended average value in [4] (released 16 years ago) was 8.2·10-4 with standard deviation equal to $1.0 \cdot 10^{-4}$ [kg/m^3] which cannot be considered on the safe side any more.

5.4. Inverse Carbonation Resistance, $R_{ACC,0}^{-1}$

Inverse carbonation resistance under accelerated conditions defines the quality of concrete in terms of transport characteristics and permeability, i.e. diffusion coefficient of CO_2. It mostly depends on type of cement and w/c ratio. It has to be obtained by testing the concrete samples in carbonation chamber for 28 days under certain environmental conditions defined with CO_2 concentration (2%), humidity level (65%) and temperature (200) [4]. After measuring of carbonation depth (xc), an inverse carbonation resistances obtained under accelerated conditions can be calculated:

$$R_{ACC,0}{}^{-1} = \left(\frac{x_c}{\tau}\right)^2 \tag{12}$$

with time constant τ= 420 [(s/(kg/m^3)^{0,5}], for abovementioned conditions. For different CO_2 concentration and duration of test, τ can be calculated:

$$\tau = \sqrt{2 \cdot C_S \cdot t} \tag{13}$$

Over the inverse carbonation resistance, a direct impact on design of concrete cover could be achieved, primarily by choice of adequate cement type. According to some studies [10, 11, 13, 14, 15], it is evident that depending of chosen cement mixture and cement type, inverse carbonation resistance differs by an order of magnitude. It is obvious even from the oriented values of $R_{ACC,0}^{-1}$ which are between 1.9 and $60x10^{-11}$ (m^2/s)/(kg/m^3), where the lower value means better quality of concrete in terms of carbonation resistance, Table 4.

In case it is not possible to obtain $R_{ACC,0}^{-1}$ from experimental testing, the calculated values can be used in function of concrete compressive strength (fcm), eq. (14) [16]. The relation came from statistical analysis of numerous results from the literature, Table 5.

$$R_{ACC,0}^{-1} = a \cdot f_{cm}^{\ b} \tag{14}$$

5.5. Weather function, W

The weather function W takes into account the meso-climatic conditions due to wetting events of the concrete surface and is calculated according to the following equation:

$$W = \left(\frac{t_0}{t}\right)^{\frac{(p_{SR} \cdot ToW)^{b_w}}{2}} \tag{15}$$

- t_0 – time of reference [years], 28[days]→0.0767[years]
- t – time [years]
- ToW – time of wetness- (“decisive” 2 rain event)/365 [-]
- pSR – probability of driving rain [-]
- bw –exponent of regression [-] (normal distribution → μ=0.446, standard deviation σ=0.163)

Weather function therefore, depends on 3 parameters – probability of driving rain, time of wetness and time of exposure. Maximum depth of carbonation takes place in case of minimum amount of precipitation (minimum number of rainy days per year) and if interior structural elements are treated (minimum probability of driving rain). It is due to the fact, that a rain event will lead to saturation of the concrete surface which will, at least temporarily, prevent a further carbonation progress since the pores are widely filled with water.

Based on data obtained from Belgrade weather station, it is estimated that the average number of rainy days per year, for reference period of 50 years is 67, with standard deviation of 10 days. Varying the value for ToW in these boundaries (67±10), practically negligible differences in values of weather function are obtained. The differences in the weather function and hence, in the carbonation depth are up to 7.5%.

Probability of driving rain, p_{SR}, is defined by Model Code [4] as “the average distribution of the wind direction during rain events”. If horizontal element is treated, p_{SR}=1, while for elements sheltered from rain (interior) p_{SR}=0. In general, pSR is evaluated as ratio between sum of days during one year with wind in considered direction, while at the same day a “decisive” rain event is taking place, and sum of days during one year with “decisive” rain events [6]. For vertical elements it can take the value between 0 and 1. The impact of changing the parameter p_{SR} on weather function, with adopted constant value ToW=67, should be obviously seen on Figure 9.

Minimum values for weather function, W(t), are obtained for horizontal structural elements considering that these elements are directly exposed

Table 4 - Recommended values for inverse carbonation resistance under accelerated conditions [4]

$R_{ACC,0}^{-1}$ [10^{-11}(m^2/s)/(kg/m^3)]	$w/c_{eqv.}^{\ -1}$					
Cement type	0.35	0.4	0.45	0.5	0.55	0.6
CEM I 42.5 R	n.d.[2]	3.1	5.2	6.8	9.8	13.4
CEM I 42.5 R + FA (k=0.5)	n.d.[2]	0.3	1.9	2.4	6.5	8.3
CEM I 42.5 R + SF (k=2.0)	3.5	5.5	n.d.[2]	n.d.[2]	16.5	n.d.[2]
CEM III/B 42.5	n.d.[2]	8.3	16.9	26.6	44.3	80

[1] ekvivalent water to cement ratio,considering FA (fly ash) or SF (silica fume) with the respective k-value (efficiency factor)
[2] n.d. inverse effective carbonation resistance has not been determined for this concrete mix

Table 5 - Values of coefficient a and b for different types of concrete [16]

Concrete	a	b
NAC	$8 \cdot 10^6$	-2.100
RAC 10–50%	$8 \cdot 10^6$	-2.100
RAC 100% (f_{cm} ≤ 36.2 MPa)	$2.12 \cdot 10^8$	-3.013
RAC 100% (f_{cm} > 36.2 MPa)	$8 \cdot 10^6$	-2.100
FAC 10–35% fly ash	$8 \cdot 10^6$	-2.100
FAC 40–70% fly ash	$2.80 \cdot 10^7$	-2.352

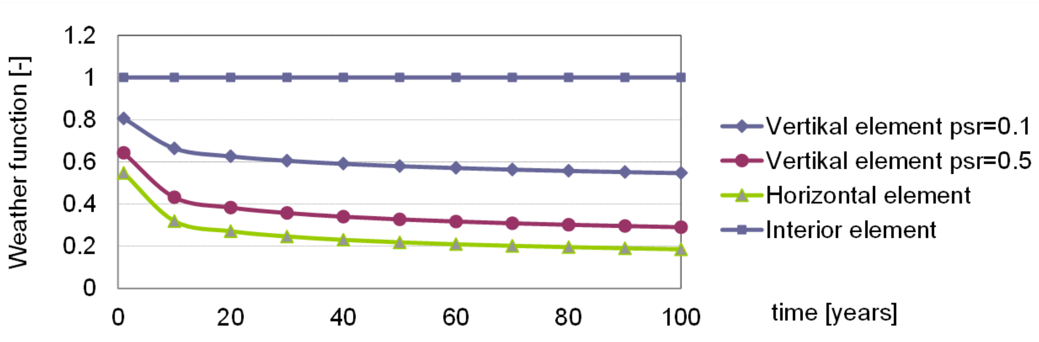

Figure 9 - Weather function for different structural elements

to precipitations which delay the carbonation process. Carbonation is far more important for vertical structural elements such as columns, walls, chimneys, etc. where the value of the weather function, and therefore the depth of carbonation, may be even 3 times greater than for horizontal element, Figure 9.

6. APPLICATION IN PRACTICE

Two questions are commonly raised in practice with regard to the service life of reinforced concrete structures. First, during the design of new structure, the question about the quality of concrete in terms of its resistance to certain deterioration mechanisms (herein carbonation) and the need to quantify durability performance and to calculate expected service life. Second, for existing structures exposed to environmental impact during the service life, there is a need to assess the capability of structure to continuously resist to certain deterioration mechanism and a need to quantify the rest of service life. In line with that is a request for proposal of measures and repairs needed to provide planned or extended service life.

Several case studies that have been done by the author's team during the past ten years will be presented here. The purpose of these studies is to illustrate the service life design and to point to the challenges and options that the designer have to make during this process. Names of clients and projects will be left out with premeditation. Software VaP was used for all probabilistic calculations.

CASE STUDY: Design of a new structure (1)

Samples of concrete designed for bridge application have been made and sent to the laboratory for durability analysis. Two samples (cylinders, D=100 mm) had been previously tested in compression and results of compressive strength testing are given in Table 6. Compressive strengths of cube with dimensions of 15 cm ($f_{c,15}$) and cylinder Ø15x30 cm (fcm) were calculated by multiplying the strength of test cylinder (f_c) with adequate coefficients.

After exposure to accelerated carbonation in a proper environment (2% CO_2, 65% RH, 20°C) for 28 days [4], average carbonation depth of Sample 1 was 8.04 mm and of Sample 2 was 5.71 mm. In order to quantify the measured values and estimate the carbonation resistance of tested concretes, the inverse effective carbonation resistance, R_{ACC}^{-1}, is calculated (eq. 12) and the following results were obtained: 36.64×10^{-11} (m²/s)/(kg/m³) and 18.48×10^{-11} (m²/s)/(kg/m³).

For qualitative assessment of concrete in terms of carbonation, these values can be compared with the recommended values, Table 3. It can be noted that the inverse effective carbonation resistance of this concretes is similar like expected values for concrete with w/c ≥ 0.55. Although there was no data regarding the real w/c applied in mixture, this estimation of w/c seems reliable having in mind the results of compressive test, Table 5. Thus, with w/c ≥ 0.55 only moderate quality in terms of carbonation resistance can be expected.

Inverse carbonation resistance is in literature presented also in another measurement units, so the results obtained and given above can be transformed in 11554.8 (mm²/year)/(kg/m³) and 5827.9 (mm²/year)/(kg/m³). These data can be used to choose the proper concrete cover in function of exposure class [17], Table 7.

The second sample gave lower value of $R_{ACC,0}^{-1}$ and 30 mm concrete cover for XC2 looks applicable, while 40 mm would necessary if conclusion is based on the first sample with 11554.8 (mm²/year)/(kg/m³). Obviously, if this approach is used it is necessary to provide more samples to get more reliable conclusion with regard to the concrete cover. This approach belongs to the "deemed to

Table 6 - Results of compressive strength testing

Sample label	Compressive strength f_c [MPa]	Compressive strength $f_{c,15}$ [MPa]	Compressive strength f_{cm} [MPa]
1	30,5	32.7	25.9
2	34.1	36.6	29.0

Table 7 - Average values of $R^{-1}_{ACC,0}$ ((mm²/year)/(kg/m³)) for different exposure classes and concrete covers (c_{min}) [17]

Exposure class	c_{min} (mm)						
	10	15	20	25	30	35	40
XC1	< 2100	< 5200	< 9500	< 15000	< 21500	< 29500	< 38500
XC2	< 650	< 1700	< 3200	< 5150	< 7500	< 10200	< 13500
XC3	< 400	< 1150	< 2200	< 3600	< 5200	< 7200	< 9500
XC4	< 430	< 1250	< 2350	< 3800	< 5600	< 7600	< 10000

satisfy rules" branch for verification of limit state, Figure 2, as tabulated values were derived from a full probabilistic method used and service life of 50 years.

But, what if there is no equipment to perform neither accelerated carbonation test nor data about the used type of cement and w/c? Based on experimental data obtained during an extensive campaign carried out on cast-in-place uncracked concretes of in-field exposed existing reinforced concrete structures from a highway infrastructure, Guiglia&Taliano [18] propose analytical relationship between inverse effective carbonation resistance ($R_{NAC,0}^{-1}$) and mean value of compressive strength:

$$R_{NAC,0}^{-1} = 10^7 \cdot f_{cm}^{-2.1} \tag{16}$$

Accordingly, based on the values of fcm (Table 5) and eq.(7), carbonation depth ($x_c(t)$) and consequently service life can be calculated. But, attention should be taken as eq.(16) was derived by linear regression of collected in-situ results on structures that have been exposed to certain environmental conditions (CO_2, RH). The reliability of utilization, i.e. extrapolation of the same relation for all other exposure conditions is doubtful.

Beside this approach, calculated values of inverse accelerated carbonation resistance were used to perform full probabilistic method for service life design. Limit state depassivation of reinforcement due to carbonation was tested (eq. (2)) and carbonation depth in function of time calculated based on eq. (9). The client provided measurements of CO_2 concentrations at the site so the mean value of $8.7 \cdot 10^{-4}$ [kg/m³] and standard deviation of $0.5 \cdot 10^{-4}$ [kg/m³] were calculated. According to the Client's statement, concrete was cured 7 days after casting which meant that execution parameter is equal to one. For weather function value 1 is adopted, since the surface of element that was designed was vertical and sheltered from rain. This would give the maximum value of the carbonation depth, and all surfaces unsheltered from rain would have smaller carbonation depths. Other parameters were taken as recommended and explained in previous sections.

For the adopted data set and with varying concrete cover and service life, using the First Order Reliability Method (FORM) analysis the failure probability (p_f) and reliability index (β) were calculated, as shown in Table 8.

For a given pair of input data - concrete cover depth (a) and time of exposure to carbonation (t), probability of failure (p_f) and reliability index were calculated (β). Service life of 100 years is demanded for bridges as well as reliability index of at least 1.3. Obviously, concrete cover of 60 mm would be

Table 8 - Result of probabilistic calculation – case study 1

SAMPLE 1			SAMPLE 2		
Parameter	pf	β	Parameter	pf	β
a=20mm, t=1.6 years	**0.096**	**1.3**	**a=20mm, t=3 years**	**0.094**	**1.32**
a=20mm, t=10 years	0.395	0.27	a=20mm, t=10 years	0.227	0.75
a=30mm, t=9 years	**0.093**	**1.32**	**a=30mm, t=10 years**	**0.042**	**1.73**
a=30mm, t=10 years	0.108	1.23	a=30mm, t=18 years	0.097	1.3
a=40mm, t=10 years	0.014	2.2	a=40mm, t=10 years	0.003	2.72
a=40mm, t=23 years	**0.098**	**1.29**	**a=40mm, t=45 years**	**0.1**	**1.28**
a=50mm, t=10 years	$7.0 \cdot 10^{-4}$	3.18	a=40mm, t=50 years	0.127	1.14
a=50mm, t=30 years	0.033	1.84	a=50mm, t=10 years	$1.1 \cdot 10^{-4}$	3.7
a=50mm, t=40 years	**0.083**	**1.38**	**a=50mm, t=50 years**	**0.019**	**2.07**
a=50mm, t=50 years	0.16	0.99	a=50mm, t=80 years	0.091	1.34
a=60mm, t=10 years	$1.7 \cdot 10^{-5}$	4.15	a=50mm, t=100 years	0.173	0.94
a=60mm, t=30 years	0.003	2.76	a=60mm, t=10 years	$1.4 \cdot 10^{-6}$	4.69
a=60mm, t=50 years	0.031	1.87	a=60mm, t=50 years	0.001	3.01
a=60mm, t=65 years	**0.089**	**1.35**	**a=60mm, t=100 years**	**0.035**	**1.82**

needed to reach the target. For thinner concrete cover, targeted reliability would be reached for shorter period of time (bolded values in the table). These results served as an input parameters for the durability design of bridge structure.

CASE STUDY: Design of a new structure (2)

In this case, client sent three samples for accelerated carbonation testing and assessment of concrete resistance to carbonation. There was no data with regard to the concrete mixture or compressive strength. Based on a measurements of carbonatin depth (xc) on three samples, an inverse accelerated carbonation resisstance was calculated by means of eq.(12) to 5.07x10^{-11} (m^2/s)/(kg/m^3), i.e. 1600 (m^2/s)/(kg/m^3). CO_2 concentration was taken as recommended value from fib MC (average: $8.2\cdot10^{-4}$ [kg/m^3], st.dev:$1.0\cdot10^{-4}$) while other parameters were like in previous case. For this set of data, varying the concrete cover depth (20÷50 mm) and curing period (3 days and 7 days), FORM analysis was performed with the target reliability index β=1.3. The same limit state as in previous case study was assumed. Calculated values of service life in function of curing time and concrete cover are presented in Table 8.

For the same curing conditions, the time needed for carbonation to passivise reinforcement, i.e. for carbonation of the full concrete cover and making the conditions for corrosion, increases with the increase of concrete cover thickness. If at least 50 years of service life is demanded, it can be fulfilled with 30 mm of concrete cover and 7 days of curing or 35 mm concrete cover with 3 days of curing.

In both presented cases, $R_{ACC,0}^{-1}$ was obtained through the experimental session on samples exposed to accelerated carbonation under 2% of CO_2. Different standards define different percentage of CO_2 during accelerated test – 2% [4], 3% [19], 4% [20] and in general, the range is even greater, from 1% to 50% [21]. The fact that coefficient K (eq.(6)) does not depend on CO_2 concentration makes an important assumption that allows the use of accelerated carbonation tests with different CO_2 concentrations and enables comparison of carbonation depths ($x_{c,1}$, $x_{c,2}$) at different CO_2 concentrations ($[CO_2]_1$, $[CO_2]_2$) and exposure times (t_1, t_2), for one concrete type.

Therefore, in case that carbonation depths were measured after exposure to CO_2 that differs from 2% or the duration of test differs from 28 days, it is possible to calculate carbonation depth needed for R_{ACC}^{-1} calculation starting from eq. (2):

$$\frac{c,1}{c,2} = \sqrt{\frac{[CO\ ]_1}{[CO_2]}} \cdot \sqrt{\frac{1}{t_2}} \tag{17}$$

This expression can be used when CO_2 concentrations differ for relatively small amounts. If they are quite different the expression will not be reliable as there is a change in kinetic of carbonation process [22].

CASE STUDY: Assessment of remaining service life of existing structure (3)

In case of existing structures, it is usual to get concrete cores for analysis and the question is to assess the remaining service life of the structure exposed to carbonation. Beside cores drilled from the structure, the duration of exposure to deterioration mechanism, i.e. how old is the structure is a very valuable data.

Measurement of carbonation depths, Figure 10, were conducted in accordance with the standard EN 14630: 2006 [23]. For the structure exposed to natural carbonation for 26 years, measured values for columns and slabs (uper and lower side) are given in Table 10. Base on this values and eq. (5), carbonation coefficient (k_c) was calculated for each depth.

Obviously, the quality of concrete in terms of carbonation resistance depends of the structural member from which the sample was taken. Samples taken from the upper surface of slab have for the level of magnitude lower carbonation coefficient compared to samples taken from the bottom side and consequently better carbonation resistance. The upper side of slab have been exposed to precipitations, which significantly slowed down the carbonation. For the assumed service life of

Table 9 - Result of probabilistic calculation – case study 2

Concrete cover	t_c [curing days]	β	t [years]
a=20 mm	3 days	1.34	12
a=30 mm	3 days	1.31	45
a=35 mm	3 days	1.31	70
a=40 mm	3 days	1.33	100
a=50 mm	3 days	1.3	180
a=20 mm	7 days	1.3	20
a=25 mm	7 days	1.36	40
a=30 mm	7 days	1.31	73
a=40 mm	7 days	1.3	165
a=50 mm	7 days	1.31	290

Figure 10. Measurement of carbonation front depth

the next 50 years, the depth of carbonation front can be calculated (eq.(5)) and it is still far away from passivaton layer of reinforcement:

$$x_c\left(t=50years\right)=0.9+0.18\cdot\sqrt{50}=2.2\quad mm \quad (18)$$

On the other hand, on the bottom surface of slab extensive carbonation was noticed and measured. There was a measured concrete cover of 25 mm. It means that the time needed for carbonation of the whole concrete cover would be:

$$t\left(x_c=25mm\right)=\left(\frac{25-17.4}{3.41}\right)^2=5.0\quad years \qquad (19)$$

Therefore, after only 5 years of further exposure to CO_2 it is expected that condition for corrosion will be fulfilled. In the next 50 years estimated carbonation depth would be:

$$x_c\left(t=50years\right)=17.4+3.41\cdot\sqrt{50}=41.5\quad mm \quad (20)$$

which is much deeper than the position of reinforcing bars, i.e. thickness of concrete cover. Thus, propagation of corrosion is expected and the structure would reach limit state – cracking or even spalling of concrete, Figure 1.

Calculation peformed for columns showed that existing 30 mm of concrete cover will be carbonated in the next 55.9 years which is even above the targeted service life of 50 years:

$$t\left(x_c=30mm\right)=\left(\frac{30-12.2}{2.38}\right)^2=55.9\quad years \quad (21)$$

7. CONCLUSION

Methodology of service life design proposed by fib and briefly presented in this paper offers a powerful tool for analysis of structures in terms of planning durability or assessment of existing structures with regard of durability. Models for two main deterioration mechanisms - carbonation and chloride ingress that lead to reinforcement corrosion as the most severe threat to reinforced concrete structures have been developed, all parameters quantified and recommended values given. Their influence was analyzed taking into account local environmental conditions, construction materials and practice. Although powerful, probabilistic approach can be too complicated for engineers and daily based calculations. All parameters should be used as stochastic values, hence databases are necessary to be provided. It is not a problem for parameters that describe environmental impact, but there is a problem to provide a database with regard to the duration of curing, which significantly affects calculation of service life. Especially important is to fill a database which correlates inverse carbonation resistance in accelerated condition with cement type and water to cement ratio. Apart from this, obstacle in application can be a software needed for such probabilistic calculations. They are commercially available but basic training for their use is necessary. On the other hand, the use of semi probabilistic approach, i.e. partial factor method, although seems reasonable and similar to design for resistance, is compromised with the need for calibration of partial factors (to reach the target reliability) for each case study, i.e. they are still not defined for proposed models

Table 10 - Measured carbonation depths and calculated carbonation coefficient – case study 3

Sample	Depth x_c [mm]	k_c [mm/year$^{0.5}$]	Element type	$x_{c,\,mean}$ [mm]	$k_{c,\,mean}$ [mm/year$^{0.5}$]
S81	7.0	1.37	Column	12.2	2.38
S63	17.3	3.39			
48	1.8	0.35	Upper surface of slab	0.9	0.18
51	0.8	0.16			
67	0.9	0.18			
1* upper s.	0.5	0.10			
2* upper s.	0.5	0.10			
3* upper s.	1.0	0.20			
1* bottom s.	9.1	1.78	Bottom surface of slab	17.4	3.41
2* bottom s.	20.5	4.02			
3* bottom s.	22.5	4.41			

nor for certain parameters. Therefore, the application of "deemed to satisfy" design approach where the durability indicators are quantified for different duration of service life and represent a demand which have to be proved by testing in the phase of structural design, seems to be a reliable and optimal solution for engineering practice.

ACKNOWLEDGEMENTS

Presented analysis and calculations for practical applications are results of team effort. Contribution of prof. Snežana Marinković and assist. prof. Vedran Carević is greatly appreciated.

REFERENCES

[1] BAB 87, Knjiga 2 Prilozi, Građevinska Knjiga, 1991, pp.3-111.

[2] SRPS EN 1992-1-1:2015 Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings

[3] SRPS EN 206:2017 Concrete - Specification, performance, production and conformity

[4] fib Task Group 5.6:Model Code for Service Life Design, International Federation for Structural Concrete fib, Lausanne, Switzerland, 2006, pp. 116

[5] fib Model Code for Concrete Structures, fib, Lausanne, 2010, pp. 317

[6] Sascha Lay, Peter Schießl, Instructions on methodology and application of models for the prediction of the residual service life for classified environmental loads and types of structures in Europe, 2003, p. 169

[7] French Civil Engineering Association: Concrete design for a given structure life, State of the Art and Guide for the implementation of a predictive performance approach based upon durability indicators, 2007, p. 240

[8] K. Tuutti, Corrosion of steel in concrete, Swedish Cement and Concrete Reasearch Institute, Stockholm, Sweden, 1982.

[9] Carević V., Ignjatović I. Influence of loading cracks on the carbonation resistance of RC elements. Construction and Building Materials. 2019; 227.

[10] O. Trocónis de Rincón, Duracon Collaboration, Durability of concrete structures: DURACON, an iberoamerican project. Preliminary results, Building and Environment 41, 2006, p. 952-962

[11] J. Khuthongkeaw, S. Tangtermsirikul, T. Leelawat: A study on carbonation depth prediction for fly ash concrete, Construction and Building Materials 20, 2006, p. 744-753

[12] M. Luković, I. Ignjatović: Analysis of the parameters in the model of carbonation process proposed by new Model Code, Proceedings of the XXV Congress and International Symposium, DIMK Srbije, pp. 341-350, Tara, October 19-21, 2011.

[13] E. Rozière, A. Loukili, F, Cussigh: A performance based approach for durability of concrete exposed to carbonation, Construction and Building Materials 23, 2009, p. 190-199

[14] K. Sisomphon, L. Franke: Carbonation rates of concretes containing high volume of pozzolanic materials, Cement and Concrete Research 37, 2007, p. 1647-1653

[15] Vagelis G. Papadakis, Effect of supplementary cementing materials on concrete resistance against carbonation and chloride ingress, Cement and Concrete Research 30, Denmark, 2000, p. 291-299

[16] Carević V., Ignjatović I. Evaluation of Concrete Cover Depth for Green Concretes Exposed to Carbonation. Structural Concrete. 2020. https://doi.org/10.1002/suco.202000086

[17] Vedran Carević, Ivan Ignjatović. Limit values of accelerated carbonation resistance to meet EC2 durability requirements. Building materials and structures. Vol.65(1), p.1-6

[18] Matteo Guiglia, Maurizio Taliano. Comparison of carbonation depths measured on in-field exposed existing r.c. structures with predictions made using fib-Model Code 2010, Cement and Concrete Composites, 2013, Vol. 38, p.92-108

[19] ISO 1920-12, Tessting of concrete - Part 12: Determination of the carbonation resistance of concrete - Accelerated carbonation method, (2015) 14.

[20] PrCEN/TS 12390-12, Testing hardened concrete - Part 12: Determination of the potential carbonation resistance of concrete: Accelerated carbonation method, (2010) 15.

[21] T.A. Harrison, G. Khanna, S. Kandasami, M.D. Newlands, M.R. Jones, Experience of using the prTS 12390-12 accelerated carbonation test to assess the relative performance of concrete, Mag. Concr. Res. 64 (2012) 737–747. doi:10.1680/macr.11.00162.

[22] V. Carević, I. Ignjatović, J. Dragaš. Model for practical carbonation depth prediction for high volume fly ash concrete and recycled aggregate concrete. Construction and Building Materials, 2019, 213, 194-208.

[23] EN 14630:2006: Products and systems for the protection and repair of concrete structures. Test methods. Determination of carbonation depth in hardened concrete by the phenolphthalein method