Deterioration rate diagnosis to global climatic change: The case of Dnipro Dam in Ukraine

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Deterioration rate diagnosis to global climatic change: The case of Dnipro Dam in Ukraine

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ABSTRACT: Global climate change and increase of CO_2 emissions have dramatically influenced the deterioration processes in concrete structures worldwide. Thus, increased rate of carbonisation-induced corrosion for long-operated concrete structures has become one of the most critical durability issues in recent years. Although significant research has been conducted on the development of comprehensive approaches for evaluating the carbonisation effect on remaining resource, the application of existing models to particular case studies and bridging the gap between theory and practice is still underexplored, which is the main motivation for this study. Existing carbonisation-induced numerical models are investigated and applied to this case study. Results demonstrate the criticality of structural deterioration in conditions of global climatic change and this highlights the need to account for environmental changes in design and practice.

1 INTRODUCTION

In recent decades, an alarming change of climate was witnessed, the most important signs of which are relentless temperature increase and enhanced carbon dioxide emissions. According to Mauna Loa Observatory (MLO) measurements, the atmospheric CO_2 emissions increased by 100 ppm during last 60 years, which induced an increase in global air temperature by about 1° during last century (Bastidas-Arteaga et al, 2022; Talukdar et al, 2012). Recent patterns in climate change, including temperature and pollutant concentrations increment, changes in relative humidity and precipitation are widely recognized as one of the most pressing global challenges of our time. The consequences of these intertwined phenomena extend far beyond mere scientific curiosity; they encompass a complex web of ecological, economic, and societal ramifications. Within the scope of various influences, the phenomenon of global warming, characterized by an elevated CO_2 concentration, and the subsequent changes in temperature and relative humidity under specific circumstances, have the potential to expedite carbonation-induced corrosion. This, in turn, has the capacity to exert an impact on the longevity and operational capability of reinforced concrete (RC) structures (Saura-Gómez, et al, 2022; Coronelli, 2007; Anand et al, 2021; Lahdensivu et al, 2019). Given the predominant role of RC as construction type used in infrastructural assets and substantial direct and indirect expenses associated with their corrosion, even a minor acceleration of the corrosion process due to climate change can lead to substantial annual rises in maintenance and repair costs (Stewart et al, 2012). Considering that decreasing service life s in the context of a changing climate is among the most important research issues of interest, the development of approach to assess the level of deterioration of RC assets has reached its topicality during recent decades

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(Ekolu, 2020; Inam et al, 2021). Most of existing studies in the literature provide the description of carbonization models, considering various influential factors. However, there is still insignificant amount of research, which would provide comprehensive illustration of application of such models on particular case studies- e.g. RC assets. Such demonstrative case study would provide valuable representation of the critical impact of structural deterioration in conditions of global climatic change and emphasize necessity to account environmental trends in design practice.

2 CLIMATE CHANGE AND CARBONATION-INDUCED CORROSION MODEL

Climate changes, reflected in various environmental factors (CO_2 concentration, temperature, humidity) have considerable impact on rate of deterioration especially in urban areas. In particular, corrosion damages in RC massive structures, operated during long time is a common problem for efficient exploitation of infrastructural systems. Initiation and propagation of corrosion damages, thus, could differ depending on structural type, loading schemes and aggressive environment, but in most cases include common stages: corrosion initiation, crack initiation and crack propagation. In this work damage due to carbonation induced corrosion will be considered, as one of the most common reasons of aged RC structures deterioration or even failure (Saura-Gómez, et al, 2022; Coronelli, 2007; Anand et al, 2021; Lahdensivu et al, 2019).

The point of carbonation -induced corrosion initiation (T_{init}) could be considered as the time when the carbonation depth reaches the surface of the rebar. Models for evaluation of carbonation depth, presented in the literature are based on Fick`s 1st low, and, according to Yoon et al. (2007) could be described by following Equation 1:

$$X_C = \sqrt{\frac{2D_{CO_2}}{\alpha} C_{CO_2} t} \tag{1}$$

where X_c = carbonation depth; C_{CO2} =atmospheric CO_2 concentration; D_{CO2} =CO_2 diffusion coefficient concentration. Further modification of this model was assumed by Stewart et al (2010) and Peng & Stewart (2014), by considering the time (t) dependent change in diffusion coefficient due to temperature ($f_T(t)$) and humidity ($f_{RH}(t)$) changes, as well as increased CO_2 levels in urban environment (k_{urb}), which could be expressed by Equations 2-3:

$$X_C(t) \approx \sqrt{\frac{2 f_T(t) f_{RH}(t) D_{CO_2}(t)}{\alpha} k_{urb} \int_{to}^{t} C_{CO_2}(t) dt \left(\frac{1}{t - t_0}\right)^{n_m}} \tag{2}$$

$$D_{CO_2}(t) = D_1 (t - t_0)^{-n_d} \tag{3}$$

where D_1 =diffusion coefficient after one year of operation of structure, which is assumed to increase in rate of age factor n_d; t_0 =time of beginning of exploitation; n_m = age factor for microclimatic conditions; k_{urb} =urban environment factor.

Increase of diffusion rate at high temperatures could be described according to Arrhenius Law:

$$f_T(t) = \exp\left(\frac{E}{R}\left(\frac{1}{293} - \frac{1}{273 + T(t)}\right)\right) \tag{4}$$

In which the reference temperature of 20°C is considered; E= activation energy of the diffusion process; R =gas constant; $T(t)$=temperature change within the time (t) range.

According to Model Code for Service Life Design (fib, 2006), the impact of humidity changes ($RH(t)$) on the carbonation rate is taken into account by Equation 5:

$$f_{RH}(t) = \begin{cases} 0 & RH(t) \leq 25\% \\ \left[\frac{1-(RH(t)/100)^{f_e}}{1-(RH_{ref}/100)^{f_e}}\right]^{g_e} & RH(t) > 25\% \end{cases} \tag{5}$$

where RH_{ref}= the reference humidity value (which in most cases is 65%); f_e and g_e=constant coefficients, equal to 5 and 2.5, respectively. Equation (5) reflects general trends of increase of diffusion rate up to RH=65%, while 100% humidity corresponds to f_{RH}=0. It is considered, the if RH is lower than the bottom level (25 %) carbonation would not begin due to small amount of water.

The degree of hydration (α) can be defined via Equation 6:

$$\alpha = 0.75 C_e C_{CaO} \alpha_H \frac{M_{CO_2}}{M_{CaO}} \tag{6}$$

where C_e=cement content; C_{CaO}=content of CaO in Portland cement; M_{CaO} and M_{CO2}= molar mass of CaO and CO_2, respectively; α_H= degree of hydration after more than 400 days, depending on water-cement ratio of concrete (w/c):

$$\alpha_H = 1 - e^{-3.38 w/c} \tag{7}$$

It could be assumed, that the stage of 1st crack initiation corresponds to time, when the 1st crack of 0.05 mm width is formed (T_{1st}), which is further followed by crack propagation and development, until the limit crack width is reached. (Stewart et al (2010))

Thus, further propagation of crack could be defined with the use of following model, which is obtained by reformulation of Stewart et al (2010):

$$\omega = \frac{i_{corr\_20}(T - T_{init}) k_c ME(r_{crack}) r_{crack}}{0.0114 k_R} \tag{8}$$

where i_{corr_20}=corrosion current density at reference temperature (T_{ref}=20°C); ME(r_{crack})=error of the model, used for evaluation of crack propagation, which considers variabilities between model prediction and experimental data (assumed values of mean and COV are 1.04 and 0.09, respectively (Stewart et al (2010)). Also, ω=crack width; k_c=confinement factor, which is important for rebars, allocated at edges of RC structures, due to increased rate of crack growth at zones with insufficient concrete confinement. Other parameters in model are defined by Equations 9-11.

$$\psi_{cp} = \frac{Cov}{\varnothing f_t} \in [0.1\ldots1] \tag{9}$$

$$r_{crack} = 0.0008 e^{-1.7\psi_{cp}} \tag{10}$$

$$k_r \approx 0.95 \left[\exp\left(-\frac{0.3 i_{corr(\exp)}}{i_{corr\_20}}\right) - \frac{i_{corr(\exp)}}{2500 i_{corr\_20}} + 0.3\right] \tag{11}$$

where f_t=tensile strength of the concrete; $\varnothing$=diameter of the rebar; Cov=cover of concrete; r_{crack}=the rate of crack development, which is exponentially dependent on depends on the Cov, f_t, $\varnothing$. In order to consider accelerated corrosion rate ($i_{corr\_20(exp)}$), required for enhancing crack development rate, loading rate correction factor k_r is introduced.

Equations listed above can be used for description of crack propagation within the time frame for RC structures, subjected to carbonation-induced corrosion. In particular the stage of corrosion damage (T_{sp}) corresponds to time when cracks in concrete cover reach the limit width (w_{lim} =1 mm). After the limit width of crack is reached, the corrosion of rebar is initiated, resulting in loss of steel bar cross-section, which, according to Stewart et al (2010) can be defined as:

$$\Delta\varnothing(t) = 2 \cdot 0.0116 \int_{T_{sp}}^{t} i_{corr}(t) dt \tag{12}$$

where temperature-dependent corrosion rate ($i_{corr}(t)$) is defined by equation, which is close correlation to Arrhenius equation:

$$i_{corr}(t) = i_{corr\_20}[1 + K(T(t) - 20)] \tag{13}$$

Additionally, it is important to consider the effect of corrosion processes on material behavior. Cracking and spalling in the compressed concrete lead to more brittle behavior. Thus, enhanced transverse strains and longitudinal microcracking cause reduction of the compressive concrete, which is, according to model of Vecchio & Collins (1986) can be defined by (Coronelli & Gambarova (2004)):

$$f'_c = \frac{f_c}{1 + K\varepsilon_1/\varepsilon_{co}} \tag{14}$$

where ε_{co} =strain at the peak compressive stress f_c; ε_1 =average tensile strain:

$$\varepsilon_1 = \frac{n_{bars}\omega_{cr}}{b_o} \tag{15}$$

where b_0= initial width of the concrete cross-section; n_{bars}=number of bars in concrete cover; ω_{cr}=total crack width for a given corrosion level X_c:

$$\omega_{cr} = 2\pi(\upsilon_{rs} - 1)X_c \tag{16}$$

where ν_{rs}=ratio of volumetric expansion of the oxides.

The general scheme for the time margins of each stage of corrosion degradation, according to utilized carbonation model is given in Figure 1.

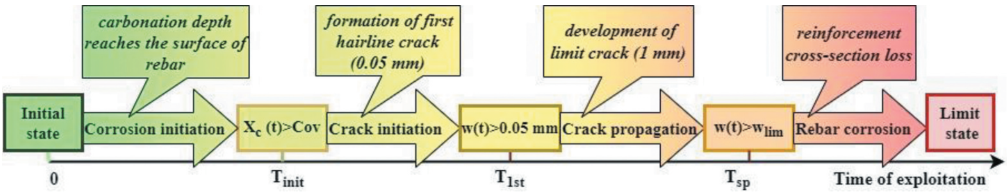


Figure 1. Main stages of degradation model.

3 CASE STUDY: RESULTS AND DISCUSSION

3.1 *Description of the case study -Dnipro HPP Dam-Bridge complex*

The Dnipro Hydroelectric Station (Dnipro HPPS) is the largest hydroelectric power station on the Dnipro River (Ukraine) and one of the most important network nodes, serving for both electric power provision for the most important industrial regions of country and enabling connection between two banks and communication within the country. As the part of transport infrastructure of Ukraine, the bridge of Dnipro HPP complex enables connection of the most important geopolitical centers and districts of the country, efficient operation of transport and logistic routes.

The Dnipro HPP consists of 49 supports that are connected from above by a monolithic reinforced concrete overpass. Its length is 760 meters, and its height is 60 meters (see Figure 2).

Dnipro HPPs complex is the oldest hydro-power station in the territory of Ukraine, which is the reason of increasing attention to its technical state and reliable operation regime. Its construction commenced in 1927, with the first hydro-unit being put into operation in 1932, while the design capacity of 560 MW was reached in 1939. Considering such long-term of operation of the

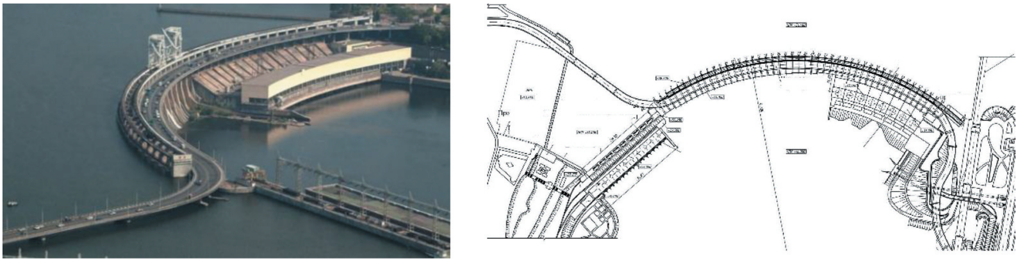
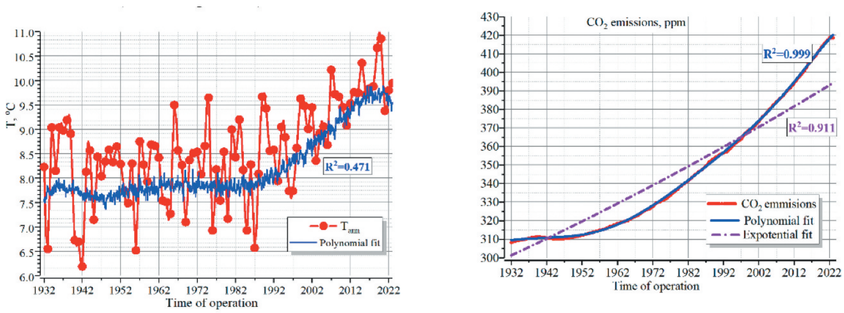


Figure 2. General view of the case study (Dnipro HPPS, Zaporizha, Ukraine).

structure in aggressive environment it is obvious, that its technical state and remaining resource requires specific attention: remote monitoring of deformations and structural state, on-site inspections of signs of deterioration, tracking damages growth. The HPP complex, in particular its bridge part is subjected to ageing, physical wear, intensive impact of humidity, etc. The strengthening measures, introduced during the last years enabled to slow down the process of destruction of structures. However, as was stated by Head of Ukrhydroenergo, based on trends of worsening of damages, the risk of destruction of the damn is increasingly reaching the emergency level (Press Service of Dnipro HPP in Politeks; Forpost). However, the ongoing military situation in Ukraine makes it impossible to conduct reconstruction works on this structure. Thus, the assessment of remaining capacity and corrosion-induced deterioration level is of prevalent importance.

3.2 *Climate changes in Ukraine and globally: Anthropological impacts*

Ukraine, as many other parts of the world has faced considerable climate changes during recent decades, which has become a growing concern, raising the public attention to necessity of adaptation measures and more precise assessment of structural deterioration parameters. In particular, a consistent increase in temperatures has been recorded, which is especially urgent for urban areas due to urban heat island effect (which is relevant for the case study in this research). Thus, according to the information of World Bank Group, temperature patterns for the examined period of time (period of exploitation of the case study, - 1932-2023) reveal the rise of annual mean average temperature by approximately 1.7 °C, while the August temperature anomalies show the increase by 2.54 °C per decade, according to NOAA National Centers for Environmental information (see Figure 3, a).



a) b)

Figure 3. Patterns of climate changes in Ukraine for studied period of time (in the area of interest-Zaporizha, Ukraine): a) annual mean average temperature; b) The increase of CO_2 concentration.

The increase of CO_2 emissions and the rise in temperature are closely interlinked aspects of climate change, as increased levels of CO_2 in the atmosphere enhance the greenhouse effect, leading to a gradual rise in global temperatures. This issue has reached its topicality during recent years, as the concentration of CO_2 has ben steadily increasing after during last century (see Figure 3, b) up to warning level, resulting in more frequent and severe heatwaves, changing precipitation patterns, and a rise in sea levels, among other effects.

3.3 *Assessment of deterioration due to carbonation induced corrosion*

Carbonation induced corrosion damage has critical impact on RC structures, subjected to long-term operation in aggressive environments, which is demonstrated with the use of case study. Deterioration model described above was used for assessment of level of corrosion damage to Dnipro HPP complex (Zaporizha, Ukraine). Values of initial parameters, used for the case study with reference sources are given in Table 1.

Table 1. Initial parameters for assessment of damage level of case study.

Parameter	Value	Units	Parameter	Value	Units	Reference
Structural parameters of the case study						
w/c	0.62	-	C_e	265	kg/m^3	Nilender (1937)
Cov	20	mm	Ø	25	mm	Nilender (1937)
f_t	2.46	MPa	f_c	21.57	MPa	Nilender (1937)
n_{bars}/b_o	5×10^{-2}	num/mm width	ε_{co}	1.6×10^{-3}	-	Nilender (1937)
$C_{CO2}(t)$	f(t)	ppm				SeaLevel.info
T(t)	f(t)	°C				climateknowledgeportal.worldbank.org
RH(t)	f(t)	%				power.larc.nasa.gov
Parameters of carbonation -induced corrosion model						
D_1	6×10^{-4}	cm^2/s	n_d	0.246	-	Peng & Stewart, 2016
n_m	0.12	-	k_{site}	1.140	-	Stewart et al, 2010
k_{urb}	1	-	$ME(r_{crack})$	1.04	-	Peng & Stewart, 2016
k_c	1.3	-	$i_{corr-20}$ ($i_{corr-exp}$)	2.586 (100)	μA/cm^2	Peng & Stewart, 2016
g_e	5	-	f_e	2.5	-	fib (2006)
E	40	kJ/mol	R	8.314	kJ/molK	
CaO		0.65				
M_{CO2}	44	g/mol	M_{CaO}	56	g/mol	
v_{rs}	2	-	K	0.1	-	Coronelli & Gambarova (2004)
T_{ref}	20	°C	RH_{ref}	0.65	%	Peng & Stewart, 2016

Parameters and coefficients of empirical model of corrosion propagation were taken from the studied literature (see Section 2). In order to consider structural parameters of the particular case study, archive materials of experimental testing of the concrete samples from Dnipro HPP in 1930 were thoroughly studied.

Thus, with the use of equations and assumptions, described in previous section, the corrosion initiation and propagation in the particular case study was assessed and its effect on remaining capacity was evaluated.

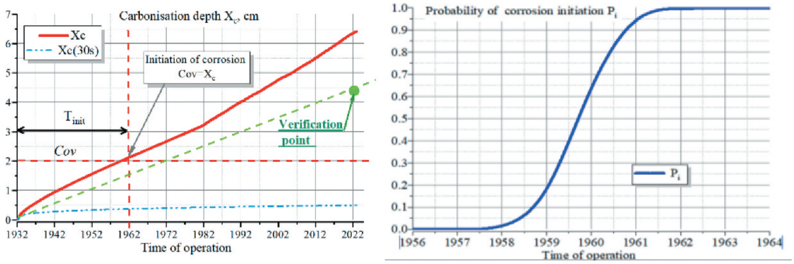
The cumulative probability of corrosion initiation (*Pi*) at time *t* is defined regarding the rapidity of carbonization depth propagation, defined by Equation (2):

$$P_i(t) = \Pr[Cov - X_c(t) < 0] \tag{17}$$

It is assumed, that the corrosion damage of reinforcement is initiated after the time when concrete cover severely cracks with limit width and its cumulative probability (P_s) at time t is:

$$P_s(t) = \Pr\left[t > T_{sp}\right] \tag{18}$$

It is important to emphasize, that the construction of this asset was performed with accordance to design regulations of early 1930s`, which do not meet the requirements of nowadays in terms of both sustainability, transport capability and resilience. In addition, design practices of that time did not consider climate changes and cumulative effect of carbonization induced corrosion. This becomes obvious from the values of concrete cover (20 mm), which does not ensure requirements of sufficient protection of rebar from corrosion (see Figure 4).

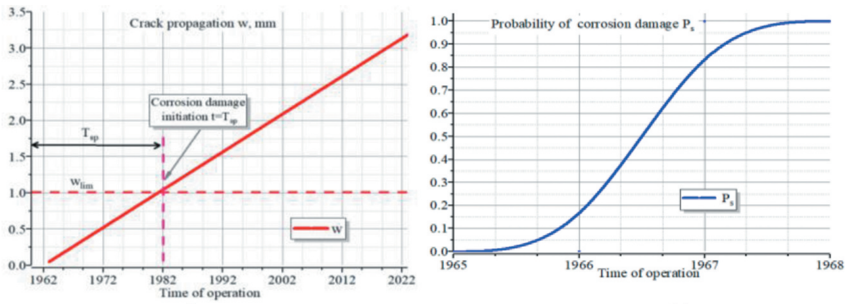


a) b)

Figure 4. Results of assessment of corrosion initiation stage for the case study: a) carbonization depth; b) probability of carbonization induced corrosion initiation.

Considering the results, illustrated in Figure 4, it can be assumed, that the corrosion initiation point was reached after 20 years of operation of structure (1932-1950), which was further followed by crack initiation stage, the indicator of which is formation of 0.05 mm hairline crack. Graph X_c(30s`) on Figure 4 demonstrates the results of similar calculations (Equation 2), if climate effects are neglected, which reveals underestimation of deterioration impact on structural safety.

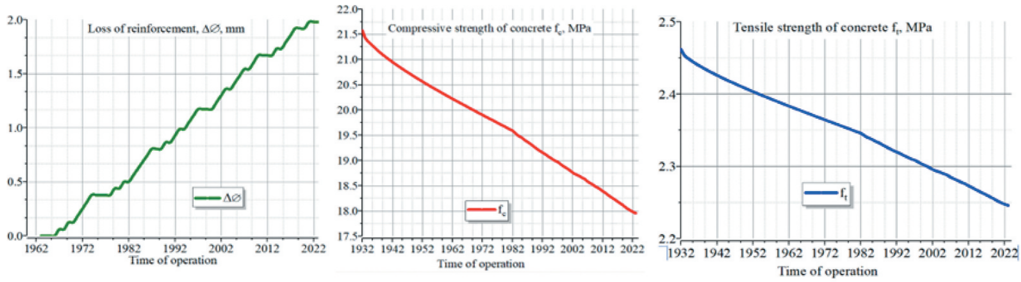
Application of crack propagation model to the Dam-Bridge structure has shown, that after the crack initiation (about 30 years of operation, -1932-1963), cracking width was increasing at enhanced rates and reached the limit value of 1 mm, evoking the beginning of corrosion damage to reinforcement (see Figure 5). Time-dependent corrosion loss or diameter of rebar according to Equation (12) follows the linear trend and for studied concrete structure could reach the value of about ΔØ=2 mm (see Figure 6,a). Finally, the approach for assessment of corrosion-induced strength decrease of concrete in long-term operated structures was applied to Dnipro HPP complex and results are demonstrated by Figure 6, b-c.



a) b)

Figure 5. Results of assessment of crack propagation and corresponding probability of corrosion damage for the case study: a) crack development; b) probability of initiation of corrosion damage.

Results, obtained with the use of proposed empirical model in general correspond to on-site observations, described in several technical reports (Onyshchenko, et al, (2021)). In particular,



a) b) c)

Figure 6. Time dependent degradation factors in corroded RC structure (case study): a)the rate of corrosion loss of the rebar diameter in the case study; b) decrease of concrete compressive strength; b) decrease of concrete tensile strength.

worsened condition of structural components was observed during visual and instrumental inspection by Galat, et al (2022), which manifested itself in increased number of cracks, areas of corrosion of concrete and reinforcement in comparison with previous inspections (2008, 2017 2021). Carbonation depth, assessed with the use of chemical reaction between phenolphthalein and the alkaline environment of concrete, reached the values up to 45 mm (Galat, et al (2022)), which is slightly lower than assumptions from empirical model (see verification point on Figure 4). Similarly, crack widths, measured during inspections were 1- 20 mm, thus the calculated values are within the range. However, although the assumptions of empirical model us this study overestimate the rate of corrosion propagation, it could still provide reliable qualitive supposition. Similar observations were reported in reports of governmental authorities, published in social media (Politeks; Forpost). Further adaptation and refinement of empirical model could be performed with the use of laboratory tests on concrete samples with particular concrete properties and particular conditions (Blikharskyy et al, 2021).

4 CONCLUSIONS

This paper employed a time-dependent analysis for interpreting the deterioration processes in RC structures due to carbonization-induced corrosion, which is becoming critical due to exacerbated environmental conditions as a result of climate change and increasing CO_2 in the atmosphere. Empirical models for the assessment of carbonisation-induced corrosion are introduced and applied to a real case study of an important asset of transport infrastructure in Ukraine. Results revealed the paramount role of climate changes and increased CO_2 emissions, witnessed recent decades, on rate of ageing and decrease of service life of RC structures, which is an important factor worldwide. To attain the desired service life for concrete structures it is essential to account for the environmental stressors in design practice, incorporating appropriate concrete cover with a margin of safety to mitigate the risk of carbonation-induced corrosion. Simultaneously, a more methodical approach, involving the analysis of concrete structures in real-world conditions across different environments, is necessary to address this matter effectively.

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