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Probability Durability Assessment of Existing Concrete Structures in Carbonation Environment

Hui Gu[1], Jitao Yao[2] and Zhaobo Meng[1]

[1]Department of Construction Engineering, Liaocheng University, Liaocheng 252000, PR China, guhuiyatou@163.com(Hui Gu), mengzhaobo@lcu.edu.cn (Zhaobo Meng)

[2]School of Civil Engineering, Xi'an University of Architecture and Technology, Xian 710000, PR China, yaojitao@163.com (Jitao Yao)

Abstract. *In a particular service environment, the calculation models used to determine the durability of existing concrete structures differ from those employed in the design phase. This paper presents a methodology for assessing the probability durability of existing structures, taking into account the non-destructive testing results and the target reliability index. The initial step involves introducing a correction coefficient into the carbonation and corrosion development theory model. This coefficient is necessary to account for the correction associated with the corrosion monitoring of steel. The values of this coefficient should be determined based on the actual testing results, specifically considering the presence of reinforcing steel corrosion. Next, a probability expression is proposed using the measured results of probability characteristics and the target reliability index. This approach takes inspiration from the design-value method and the existing theory of structure reliability. It also takes into account the specific variables of concrete protective cover thickness and the compressive strength as the fundamental random variables. The residual working life can be calculated as the outcome of a durability assessment through quantitative analysis. This calculation can serve as a valuable reference for the maintenance program.*

Keywords: *Durability assessment; Design-value method; Carbonation environment; Reliability index; Residual working life*

1 Introduction

During the service life of concrete structures, the deterioration caused by environmental erosion and the behavior of materials are influenced by inherent uncertainties and random factors. The residual durability life of a concrete structure refers to the permissible period of use, determined through prediction analysis that relies on the assumption of reliability, which is considered a random variable. Durability is a distinct issue concerning the degradation of material properties, which in turn affects reliability. The process of carbonation can compromise the protective function of steels, resulting in corrosion of the steel. This corrosion can negatively impact the overall performance of the structure, potentially causing it to reach or surpass its limit state. Consequently, the structure may become unsuitable for continued use, with its ultimate practical lifespan falling short of the originally intended service life(Kvgd et al. 2014, Liu and Fang 2012). Determining the residual service life of existing structures is a prominent subject in the field of civil engineering. This analysis offers a scientific foundation for making informed decisions regarding the appropriate timing for maintenance, reinforcement, and dismantling activities (Kvgd et al. 2014, Liu and Fang 2012, Bajaj and Bhattacharjee 2021).

This paper aims to predict the residual service life of existing structures by considering the structural objective damage caused by the general atmospheric environment. The analysis was conducted by considering the temporal aspect and modifying the state equation. This was done using the measured values of carbonation depth and corrosion depth. Considering the probability features of the variable and the target reliability index of the crack expansion, using the design-value method which is put forward in ISO2394(ISO 2015), it directly established the life calculation expression, enables the quantitative prediction of the residual useful life of a structure, given a specific guarantee ratio. It utilizes the test results of service structures and the corrosion expansion life criterion. It is organized as follows: In section 2, the durability limit state equation with probabilistic lifetime prediction model are presented. In Section 3, the modified assessment expression is presented along with the corresponding test results. In Section 4, the life prediction method is presented, specifically focusing on the design-value method. Section 5 presents a case study that applies, evaluates, and compares various in-situ inspection methods.

2 Models for carbonation-induced deterioration of RC structures

Concrete carbonation can lead to the degradation of the passive state of the reinforcing steel that is embedded within the concrete. The process of concrete carbonation is a complex physicochemical phenomenon, wherein the key factors that govern it are the diffusivity of carbon dioxide (CO_2) and the reactivity of CO_2 with the concrete material (Chang and Chen 2006, Sanjuán et al. 2003). The reaction between CO_2 and calcium hydroxide may consume a great amount of $Ca(OH)_2$ which helps to maintain a high PH-value of concrete. Corrosion initiation occurs when the carbonation depth reaches a significant level and the pH value of the surrounding concrete decreases to approximately 9-10. The carbonation-induced deterioration process of reinforced concrete (RC) structures is typically categorized into three stages: corrosion initiation, crack initiation and crack propagation (Gu and Li 2020, Koelioe et al. 2014). The three stages of this theory encompass the concept of carrying capacity, which serves as a predictive measure for determining the remaining lifespan.

In the context of concrete structures exposed to atmospheric conditions, the corrosion cracked life criterion refers to the point at which the structure has reached its serviceability limit state due to the expansion of cracks in the protective layer caused by steel corrosion (Niu 2003). For some buildings with high requirements in use or no cracks in surface, the corrosion expansion life is the end of the expectancy life as the basis for a normal life prediction. This study employed the theoretical model of the concrete cracked life theory as proposed by Niu (2003). The primary objective was to establish the limit state equation of durability and subsequently determine the theoretical time for crack propagation.

2.1 The initial corrosion time

Regardless of the residual carbonization (Niu 2003), the initial corrosion time is determined with the depth of carbonization reaching the rebar surface. If we take into account the residual carbonization, it is possible that the depth of carbonation, which initiates corrosion, did not reach or surpass the surface of the steel bar. The carbonation depth of concrete structures in a typical atmospheric environment is expected to increase over time (Niu 2003):

$$C_t = K_c k_1 \left(57.94 * m_c / f_{cu} - 0.76\right) \cdot \sqrt{t} \tag{1}$$

Where, K_c is the uncertainty factor of the calculation model for the carbonation depth; k_1 is the condition coefficient for carbide development calculated with $k_1 = K_e k_j k_{CO_2} k_p k_s$; K_e is the environmental factor calculated with $K_e = 2.56\sqrt[4]{T}\left(1-RH\right)RH$; T is the annual average relative temperature; RH is the annual average relative humidity; k_j is the angle correction coefficient, taken as 1.4 for angle, and 1.0 for others; k_{CO_2} is concentration effect coefficient of carbon dioxide calculated as $k_{CO_2} = \left(C_{CO_2}/0.03\right)^{0.5}$; C_{CO_2} is carbon dioxide concentration; k_p is the casting surface correction coefficient; k_s is the working stress influence coefficient; f_{cu} is the average value of the cube compressive strength of concrete; m_c is the coefficient of mean value, calculated by the ratio of the average value and standard value for the concrete cube compressive strength; t is the structural design life.

The limit state equation is $g_1 = c - C_t = 0$, taking into account the carbonation depth that has affected the surface of the steel bar and initiated corrosion, while disregarding the impact of any remaining carbonation. The variable c represents the precise measurement of the cover thickness. The duration of the initial corrosion period is:

$$t_1 = \left[c\big/\left(K_c k_1 \left(57.94 m_c / f_{cu} - 0.76\right)\right)\right]^2 \tag{2}$$

If considering the effect of residual carbonization, the limit state equation is $g_1 = c - x_0 - C_t = 0$. Where, x_0 is the carbide residue expressed as $x_0 = k_0\left(c-5\right)\left(\ln\left(f_{cu}/m_c\right) - 2.3\right)$, in which, k_0 is calculated with $k_0 = 4.86\left(-RH^2 + 1.5RH - 0.45\right)$. The steel initial corrosion time can be expressive as:

$$t_1 = \left[\left(c - k_0\left(c-5\right)\left(\ln f_{cu}/m_c - 2.3\right)\right)\big/\left(K_c k_1\left(57.94 m_c / f_{cu} - 0.76\right)\right)\right]^2 \tag{3}$$

2.2 The initial corrosion cracking time

The failure of a structure due to corrosion is determined by the corrosive cracking limit state, which is characterized by the extent of corrosion depth δ_t (expressed as a linear function of time (t-t_1)) reached the critical corrosion depth δ_{cr} (assumed when corrosion crack width came to $\omega = 0.1mm$ for easy to notice). Given the abundance of accelerated corrosion test data and engineering test results (Niu 2003), δ_t and δ_{cr}, correlation with the material properties and environment parameters, can be respectively described as:

$$\delta_t = k_2 c^{-1.36} f_{cu}^{-1.83}\left(t - t_1\right) \tag{4}$$

$$\delta_{cr} = k_{crs}\left(a_1 c/d + a_2 f_{cu} + a_3\right) \tag{5}$$

where, k_2 is the corrosion condition coefficient calculated as $k_2 = 46 k_{cr} k_{ce} e^{0.04T}\left(RH - 0.45\right)^{\frac{2}{3}}$; k_{cr} is the position correction coefficient for steel, taken as 1.6 for angle, and 1.0 for others; k_{ce} is the small environment correction coefficient, taken values of 1.0~1.5 when indoor and outdoor climates, 3.0~4.0 when in damp areas, and 2.5~3.5 when in drier areas; k_{crs} is the reinforced location factor, taken as 1.0 for angle, and 1.35 for others; *d* is the diameter of bar; a_1, a_2, a_3 is the parameters considering tendon types, taken respective values of 0.012, 0.00084, 0.022 when plain round bar, 0.008, 0.00055, 0.022 when deformed bars, and 0.026, 0.0025, 0.068 when stirrup or steel mesh reinforcement.

The corrosive cracking limit state equation, denoted as $g_2 = \delta_{cr} - \delta_t = 0$, provides a representation of the corrosion cracking initiation time *t*:

$$t = t_1 + \left[k_{crs} \left(a_1 c/d + a_2 f_{cu} + a_3 \right) c^{1.36} f_{cu}^{1.83} \big/ k_2 \right] \tag{6}$$

3 The modified assessment expression with test results

The observed results frequently deviate from the theoretical calculation values due to discrepancies between the calculated approximation and the level of construction quality. This paper presents modifications made to the calculation formulas based on actual test results, taking into account the prevention of steel rust and corrosion.

3.1 No corrosion when assessment

If no corrosion is detected after t_0 years of use prior to assessment, it indicates that the time elapsed is earlier than the initial corrosion occurrence. The variable x represents the test carbonation depth, which refers to the measured depth of carbonation. This depth is determined after a specific period of time, denoted as t_0 years. To take the difference between the actual environmental effects and the theoretical assumptions into account, the carbonation depth correction factor K_1 is introduced to replace the uncertainty coefficient K_C in the calculation model, calculated with $K_1 = x/C_{t_0}$, where C_{t_0} is the calculated carbonation depth after t_0 year with Equation (1). In this regard, the uncertainty associated with the calculation mode is indirectly taken into account through the utilization of the correction factor K_1. The employing of the uncertainty coefficient of the calculation model K_C has been discontinued. The initial corrosion time t_1 was obtained by considering the presence or absence of residual carbonization:

Not considering the residual carbonization

$$t_1 = \left[c \big/ \left(K_1 k_1 \left(57.94\, m_c / f_{cu} - 0.76 \right) \right) \right]^2 \tag{7}$$

Considering the residual carbonization

$$t_1 = \left[\left(c - k_0 \left(c - 5 \right) \left(\ln f_{cu} / m_c - 2.3 \right) \right) \big/ \left(K_1 k_1 \left(57.94\, m_c / f_{cu} - 0.76 \right) \right) \right]^2 \tag{8}$$

According to Equations (1) and (4), the carbonization rate can be described as a square root function with respect to time, while the corrosion rate can be described as a straightforward function of time. The limit state equation corrected as $g_2 = \delta_{cr} - K_1^2 \delta_t$, and the corrosion cracking initiation time *t* corrected as:

$$t = t_1 + \left[k_{crs} \left(a_1 c/d + a_2 f_{cu} + a_3 \right) c^{1.36} f_{cu}^{1.83} \big/ K_1^2 k_2 \right] \tag{9}$$

3.2 With corrosion when assessment

After t_0 years used before assessment, steel corrosion depth has been identified as y. With introducing the corrosion correction factor $K_2 = y/\delta_{t_0}$, in which, δ_{t_0} is the calculation value with Equation (4) after t_0 years. The corrosion cracking initiation time t is corrected as:

$$t = t_0 + \left[k_{crs}\left(a_1 c/d + a_2 f_{cu} + a_3\right) - y\right] c^{1.36} f_{cu}^{1.83} \Big/ K_2 k_2 \tag{10}$$

4 Probability Durability Assessment

The statistical data obtained from the tested project of service structures can be utilized to make inferences about the residual working life. This includes analyzing the mean values and variation coefficients of various parameters such as the measured protective layer thickness, concrete strength, carbonation depth and the steel corrosion depth. The design-value method makes use of statistical results to directly calculate the residual service life with a specific guarantee rate. The system exhibits standard characteristics, possesses adaptability, and can be easily acquired by the appropriate personnel.

4.1 The basic variables

The design-value method (ISO2394 2015) is based on the stochastic processes combination theory and JC method (Yao and Gu 2018). Corresponding to the limit state, the structure is reliability used when $g\left(X_1, X_2, \cdots X_n\right) \geq 0$. If the design checking point of the basic variable $X_i\left(X_1, X_2, \cdots X_n\right)$ shown as $p^* = \left(x_1^*, x_2^*, \cdots x_n^*\right)$ and the sensitivity coefficient a_i^* determined in the calculation process of the reliability index with FORM (First Order Reliability Methods) (ISO2394 2015), they are satisfied with the follow expression:

$$F_{X_i}\left(x_i^*\right) = \Phi\left(-a_i^* \beta\right) \quad i = 1, 2, \cdots, n \tag{11}$$

The design values can be defined as $x_i^* = F_{X_i}^{-1}\left[\Phi\left(-a_i^* \beta\right)\right]$, according to the probability distribution of the basic variable (ISO2394 2015).

The calculation result of the corrosion cracking time is influenced by various factors, as indicated by the expression provided above. The thickness of the protective layer and the compressive strength of the concrete are crucial factors that significantly impact the prediction results, as per engineering experience. For simplify, it only takes these two factors as the random variables, considering their measured statistical characteristics and ignoring the randomness of others. According to the statistics results (Yao and Gu 2018), the protective layer thickness and concrete cubes compressive strength have normal distribution shown as $N\left(u_i, \sigma_i^2\right)$. With the above design-value method, the design value of variable can be expressive as:

$$x_i^* = \mu_i - a_i^* \beta \sigma_i = \mu_i\left(1 - a_i^* \beta \delta_i\right) \tag{12}$$

The following formula incorporates the optional reliability index and the actual test results of the service structure, including the mean values and variance coefficients of the variables. The sensitivity coefficient a_i^* here reflects the sensitivity of the reliability index with basic random variables. A good level of precision and reliability can be achieved in the control of structural durability analysis by making a reasonable selection of the sensitivity coefficient value. In the literature (Yao and Gu 2018) and (Yao and Cheng 2016), it determined the sensitivity respectively as $a_X^* = \pm 0.85$ and $a_X^* = \pm 0.35$ of the main control and the non-master control variables respectively, analyzed with the whole single-control type of the optimal reliability control mode, and ensuring a high control accuracy degree of reliability. Based on the grey relation theory proposed by Yao and Cheng in 2016, this study examines the influence level and sensitivity of each factor. Based on the above analysis, it takes the sensitivity coefficient of the concrete cover thickness and the concrete compressive strength respectively as $a_c^* = 0.85$ and $a_{f_{cu}}^* = 0.35$.

4.2 Calculation of the residual working life

The FORM (First Order Reliability Method) is a reliable calculation method that simplifies the limit state equation into a linear function. The coordinate origin is defined as the average value of the random variable. The checking point is determined as the point closest to the coordinate origin. At the checkpoint, the discrepancy between the tangent plane and the original surface is minimized, resulting in the highest level of accuracy. The method demonstrates a high level of effectiveness, as evidenced by its ability to produce a unique and reliable indicator result across various equivalent functions.

Considering a limit state of the component reliability analysis as $Z = R(t) - S(t)$, where $R(t)$ is the resistance of the component and $S(t)$ is the corresponding load affection at the same section. Both of them are function of time. The dynamic reliability index β of the structure decreases with the service time increasing, and the structure fails when it below a certain limit.

The working life, as determined by the design-value method, refers to the actual service time of a structure in its specific service environment. The actual checking point value calculated by taking the test results of the basic variables into the Equation(11). The selection of the target reliability index and its corresponding failure probability, in relation to the serviceability limit state, can be made by referring to Table 1 in consultation with the owners.

Table 1. Target Reliability Indices for Serviceability Limit States (50 years)

Relative cost of safety measure	Probabilistic model code	ISO2394 and ISO13822	EN1990	GB50153
High	$\beta = 1.3(P_f \approx 10^{-1})$	$\beta = 1.5(P_f \approx 7\times10^{-2})$	$\beta = 0(P_f \approx 5\times10^{-1})$	$\beta = 0 \sim 1.5(P_f \approx 5\times10^{-2})$
Normal	$\beta = 1.7(P_f \approx 5\times10^{-2})$		$\beta = 1.3(P_f \approx 10^{-1})$	
Low	$\beta = 2.3(P_f \approx 10^{-2})$		$\beta = 2.3(P_f \approx 10^{-2})$	

Then the residual service life based on the actual test results can be calculated with follows equations:

Steel has no corrosion when assessed without considering the residual carbonization:

$$
\begin{aligned}
t = & \left[k_{crs} \left(a_1 \mu_c \left(1 - a_c^* \beta \delta_c\right) \middle/ d + a_2 \mu_{f_{cu}} \left(1 - a_{f_{cu}}^* \beta \delta_{f_{cu}}\right) + a_3 \right) \right] \cdot \left[\mu_c \left(1 - a_c^* \beta \delta_c\right) \right]^{1.36} \left[\mu_{f_{cu}} \left(1 - a_{f_{cu}}^* \beta \delta_{f_{cu}}\right) \right]^{1.83} / \left(K_1^2 k_2\right) \\
& + \left[\mu_c \left(1 - a_c^* \beta \delta_c\right) \middle/ \left[K_1 k_1 \left(57.94 m_c \middle/ \mu_{f_{cu}} \left(1 - a_{f_{cu}}^* \beta \delta_{f_{cu}}\right) \right) - 0.76 \right] \right]^2 - t_0
\end{aligned}
\tag{13}
$$

Steel has corrosion when assessed:

$$t=\left[k_{crs}\left(a_1\mu_c\left(1-a_c^*\beta\delta_c\right)\big/d+a_2\mu_{f_{cu}}\left(1-a_{f_{cu}}^*\beta\delta_{f_{cu}}\right)+a_3\right)-y\right]\cdot\left[\mu_c\left(1-a_c^*\beta\delta_c\right)\right]^{1.36}\left[\mu_{f_{cu}}\left(1-a_{f_{cu}}^*\beta\delta_{f_{cu}}\right)\right]^{1.83}/\left(K_2k_2\right) \quad (14)$$

5 Examples

To validate the accuracy of this approach and assess its versatility and convenience compared to existing life prediction methods, an example from literature (Yao and Niu, 2009) was employed. This involved performing calculations, conducting comparisons, and analyzing the results to effectively illustrate the efficacy of this method.

The reinforced concrete column selected here is described with the section size of $450mm\times450mm$ in the general atmospheric environment, with the Ⅱ grade, and with 16mm diameter ribbed bar as longitudinal reinforcement. The design concrete protection layer thickness is $u_c=50mm$, $\delta_c=0.3$, and the design concrete strength is $u_{f_{cu}}=30MPa$, $\delta_{f_{cu}}=0.2$, with the designed service life of $t=50a$. In actual used of the structure, the environment's annual average temperature is 12ºC, and the annual average relative humidity is 60%. After 13 years used, the measured carbonation depth is $x=32mm$, and the measured protective layer thickness is $u_c=45mm$ and $\delta_c=0.2$, the test concrete strength is $u_{f_{cu}}=30MPa$ and $\delta_{f_{cu}}=0.25$. The target working life will predicted with the corrosion cracking life criterion, choosing the reliability index of $\beta=1.28$, and without taking the carbonization residuals into account.

Based on the aforementioned test results, it can be concluded that the carbonation depth is found to be lower than the thickness of the protective layer. Consequently, the steel substrate is susceptible to corrosion. The Monte Carlo method was employed in a literature study conducted by (Yao and Niu 2009). The purpose of the study was to evaluate corrosion crack prediction life and cumulative failure probability. This was achieved by simulating the protective layer thickness and cube strength using a normal distribution. The simulation was performed 5000 times. With coordinated the intersection of 10% failure probability and the cumulative frequency, the calculated time is 42.5 years. It is to say the predicted residual working life of durability is 29.5 years with a target reliability index of $\beta=1.28$. This approach requires the utilization of the Monte Carlo computer simulation program, the generation of life frequency histograms, and the creation of cumulative failure probability curves.

With the method mentioned in this paper, the carbonation correction coefficient $K_1=1.409$ is calculated first, and other basic parameters are determined the same as literature (Yao and Niu 2009). Based on the assessment of the fundamental variables and Equation (11), the residual working life can be determined as 28.7 years through direct calculation. This calculation takes into account the corrosion cracking criterion and the reliability index of $\beta=1.28$. The proposed method demonstrates a reduction in computational complexity and enhanced flexibility for durability life prediction compared to the Monte-Carlo method described in Liu H, (Yao and Niu 2009). However, it is worth noting that the proposed method yields a result that is one year less than the aforementioned literature. Nonetheless, these improvements make the proposed method more user-friendly and accessible.

6 Conclusions

This paper presents a method for determining the theoretical corrosion time using the limit state equation and incorporating the corrosion cracking life criterion. The approach relies on the fundamental principles of reliability calculation theory, specifically the JC method and the design-value method. The tool has the capability to directly present the statistical characteristics of the fundamental variables and the selected reliability index for the corrosion life criterion. This assessment provides insights into the actual durability reduction experienced by the structure during its use.

The probability assessment method presented here allows for the determination of the residual working life of the existing concrete structures. This method relies on the analysis of test results and takes into account the actual degradation rate. The calculation method employed in this evaluation process offers a high degree of flexibility, convenience, and ease of mastery for the evaluation workers. The calculated result is sufficiently accurate to serve as a rational theoretical foundation for the selection of maintenance and reinforcement schemes by the owners.

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