



ESTIMATION OF RESIDUAL SERVICE LIFE OF CORRODING CONCRETE STRUCTURES: A REVIEW

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I. Introduction

Reinforced concrete is one of the important materials used in almost all types of civil engineering construction from highways to oil production platforms [1]. Most of the design studies in reinforced concrete (RC) literature assume that the durability of RC structures can be taken for granted [2,3]. But when the RC Structures are built and exposed to an environment, they undergo process of deterioration. The degree of deterioration depends on the aggressiveness of the environment to which a structure is exposed. Many structures particularly bridges and other offshore structures constructed in hostile environments reported suffer serious corrosion damages during their designed service life itself.

According to 1989 reports [4,5] in North America, a large number of bridges are constructed 50 years ago suffered extensive deterioration. According to another report in the year 1991, in USA, forty per cent of 5, 76,665 bridges on Federal Aid Systems were structurally deficient or functionally obsolete. Out of these deficient bridges, about forty per cent is 92,266 bridges are seriously affected by corrosion of reinforcing steel. In Canada [5] forty per cent of the bridges are constructed 40 years ago and their rehabilitation cost was estimated at \$10 billion. The rehabilitation of bridges in USA, during the year 1996 was reported as [6] \$90 billion. In OECD countries [5] the estimated rehabilitation and replacement costs for 800,000 bridges was \$500-900 per sq.m. during the year 1999.

As the nation's infrastructure ages (7), predicting their residual life has become increasingly important. The residual life estimation of RC structures not only help to assess the useful service life, but also helps to take appropriate decisions whether to repair or rehabilitate to extend the service life further or to demolish the structure at suitable time to prevent catastrophic failure which otherwise could lead to loss of valuable lives. In this paper some of the models developed to predict the service life of corroded concrete structures are discussed.

II. Corrosion damage

Deterioration of RC structure is caused by various factors such as corrosion of steel reinforcements, freeze-thaw cycles, mechanical loads, initial damages resulting from poor design, detailing and inadequate inspection and maintenance practices [8]. Among all these factors, the irreversible damage caused by the corrosion of steel reinforcements is considered most serious problem. By virtue of high alkalinity of concrete ($\text{pH} > 12$), and also by the passive oxide layer formed on the steel surfaces, the embedded steel reinforcements are protected in concrete. When the structure is exposed to the environment, the aggressive agents such as chloride ions, carbon-di-oxide (CO_2) and moisture diffuse into concrete through the interconnected pore system from the atmosphere. The diffused chloride ions in concrete partly combine with the hydrated cement paste of concrete and the rest will remain in concrete as free chloride. Once the free chlorides accumulate at the surfaces of steel reinforcements to the threshold level, break down of the passive oxide layer of Steel reinforcement is taking place (depassivation) and subsequently corrosion starts. The carbon-di-oxide also diffuses into concrete through the pores from the atmospheric air, reach with the hydrated cement paste and reduce the alkalinity of the concrete (carbonation). When the carbonation reaches the reinforcement, the pH of pore solution in contact with the reinforcement is considerably reduced to initiate corrosion of steel. The moisture diffusion in concrete helps for the mobility of aggressive agents and provides electrolytic contacts between anodic and cathodic sites along the embedded steel, and accelerates the corrosion process.

During the process of corrosion, metal oxides are formed around the steel reinforcements and they may be expressed as [9] $m \text{Fe}(\text{OH})_2 \cdot n \text{F}(\text{OH})_3 \cdot p \text{H}_2\text{O}$; where the values of m, n and p vary considerably depending upon the pH of the pore solution, moisture level and oxygen supply in concrete. These rust products have different densities and volume expansions (Fig. 1).

As the corrosion is in progress, the rust products also grow and exert pressure on surrounding concrete. Some quantity of the rust products fill the voids around the steel/concrete interface or migrate away from this interface [9] and the remaining will build up pressure on concrete. When this pressure exceeds the tensile strength of concrete, the concrete cracks (Figs.2-4) and subsequently delamination and spalling of concrete takes place. This in turn results in reduction in cross section of both steel as well as concrete. Therefore, structurally the strength and stiffness are reduced and the ductility of the member increased considerably and this leads to ultimate failure of a structure considering this serious problem, many researchers put their efforts to establish various physical and empirical models to assess the residual life of the structures.

III. Definition of Service Life

The definition of service life has technical, economical and legal implications as responsibilities are involved [10]. Weyers [11] describes the end-of-function service life as "the condition which defines the end-of-functional service life may be a reduction in structural capacity or safety. The second CEB/RILEM International workshop on the durability of concrete structures, described a number of definitions of service life as follows [12].

Technical Service Life: The time in service until a defined unacceptable state is reached.

Functional Service Life: Time in service until the structure becomes obsolete due to change in functional requirements.

Economic Service Life: Time in service until replacement of the structure is economically more advantageous than keeping the structure in service. **Design Service Life:** It is the anticipated time in service in the anticipated environment until a defined acceptable probability that the structure will have reached an unacceptable state is exceeded, for a defined level of maintenance.

End of Life: Practically, the point in time when it is judged to no longer be economically viable to keep the structure or element 'alive'.

IV. Service Life Models

Many service life prediction models have been derived by considering the magnitude and seriousness of the corrosion problem [7]. The two major approaches pursued are the use of models based on the conceptual model described by Bazant [13] in 1979 and Tuutti [14] in 1982 (Figs. 5) in which corrosion starts after the end of initiation period followed by propagation period of active corrosion. The initiation period involves the transportation of chloride ions to the reinforcement level to threshold concentration to depassivate the steel. The rate of corrosion in propagation period is controlled by rate of oxygen diffusion to the cathode, resistivity of pore solution and the temperature. It is also assumed that the initiation period (induction period) is much longer than the propagation period. Another approach is carbonation of concrete, which follows square root time function [15]. In the recent years, residual life prediction using time order concept is also appearing based on probability theories. When more than one degradation processes are involved, simultaneously with different time order, then the time order concept will be helpful. This will give minimum residual life and represent a conservative approach.

Service life model given by Morinaga [61] indicate the degradation process by chlorides and CO₂. It also indicates that chloride damage to a structure is more serious than by carbonation. Lounis (81 suggested a model in 2000, a slight modified version of Tuutti's (Fig.6). He also suggested a model (Fig.7) to illustrate the impact of rehabilitation on service life of structure. Following are some of the numerical models established by various researchers to quantify the residual life of structures based on deterioration by corrosion.

In 1976, Clear Kenneth (17) suggested a life model in FHWA report, by considering the primary variables such as cover thickness, water-cement ratio (w/c ratio) and chloride level in ppm is

$$\text{Life} = \frac{129}{\frac{w}{c}} \frac{(\text{Cover})^{1.22}}{\text{Cl}^{0.42}} \quad (1)$$

Bazant [18] presented the following theoretical equation for estimating the time to corrosion cracking of concrete.

$$t_{\text{corr}} = \rho_{\text{corr}} \frac{D \Delta D}{S_{\text{jr}}} \quad (2)$$

where

t_{corr} = time to corrosion cracking

ρ_{corr} = function of mass density of steel rust

D = original diameter of reinforcing bar

ΔD = increase in bar diameter

S = spacing of bar and

j_r = rate of rust production per unit length of bar

The increase in bar diameter (ΔD) is function of cover thickness (L), concrete tensile strength (f_t), bar diameter (D) and bar hole flexibility (δ_{pp}). By assuming that the bar spacing is greater than 6D, the increase in bar diameter is given by

$$\Delta D = 2 f_t \frac{L}{D} \delta_{pp} \quad (3)$$

$$\text{and } \delta_{pp} = \left[\frac{D(1+\phi_{cr})}{E} \right] (1+\gamma) + D^2 \left[\frac{2}{S^2} + \frac{1}{4L(L+D)} \right] \quad (4)$$

where

ϕ_{cr} = creep coefficient of concrete

γ = poisson's ratio of concrete and

E = elastic modulus of concrete.

During the year 1979, Bazant [13] suggested another simplified physical model to estimate service life of a structure by considering corrosion rate of reinforcing steel, and is given by

$$L_f = \frac{1.72 \left(\frac{f_t'}{E} \right) f \left(\frac{C_t}{D} \right) D}{J_r} \quad (5)$$

where

L_f = service life (years)

J_r = instantaneous corrosion rate

f_t' = tensile strength of concrete

E = elastic modulus of concrete

C_t = thickness of cover concrete

D = original diameter of rebar

In the year 1988, Morinaga [19] suggested an empirical relation to determine the service life of concrete buildings based on corrosion rate and is given by

$$t = \frac{Q_{cr}}{q} \quad (6)$$

where

$$Q_{cr} = 0.59 \left[1 + \left(\frac{2c}{d} \right) \right]^{0.85} d$$

$$q = \frac{q_1 q_2}{q_2}$$

$$q_1 = [-0.5 - 7.45N + 44.1 (w/c)^2 + 66.64N (w/c)^2 \cdot d/c^2]$$

$$q_2 = 2.54 - 0.5T - 6.76H - 22.43O - 0.97N + 0.14TH + 0.5TO + 0.01TN + 59.63HO + 3.3HN + 7.18ON$$

$$q_2' = 0.55435 + 1.402N$$

where

t = Life of a structure or structural member due to chloride induced corrosion (year)

Q_{cr} = Amount of corrosion which causes cover concrete cracking ($\times 10^{-8}$ N/mm²)

q = Corrosion rate of steel ($\times 10^{-8}$ N/mm² year)

c = Cover thickness (mm)

d = Diameter of rebar (mm)

q_1 = Corrosion rate in concrete containing chloride content at the reference condition (temperature of 15°C, relative humidity of 69% and oxygen concentration of 20%) ($\times 10^{-8}$ N/mm² year)

q_2 = Corrosion rate in concrete containing chloride content at the condition of the structure or structural member ($\times 10^{-8}$ N/mm² year)

q_2' = Corrosion rate of the structure or structural member under reference condition ($\times 10^{-8}$ N/mm² year)

N = Sodium chloride (NaCl) by mass of rinsing water (%) ($N = 165 \times Cl/w$)

Cl = Chloride content in concrete (kg/m³)

W = Water to cement ratio (%/100)

T = Temperature (°C)

H = Humidity [$H = (RH-45)/100$]

RH = Relative humidity (%)

O = Oxygen concentration (%/100)

In this method, the corrosion rate has to be calculated every year, and cumulative amount of corrosion till n^{th} year is calculated by summing up (Q_n). The life is assessed by comparing Q_n with Q_{cr} . When Q_n exceeds Q_{cr} , crack due to corrosion of rebar is considered to be caused and the n^{th} year is regarded as the life.

Vesikari [20] suggested a model in the year 1988 to predict the service life of chloride contaminated concrete structure and is given by

$$t = k_c \cdot k_e \cdot c^2 + k_a \cdot c \quad (7)$$

where

t = Service life (in years)

c = Cover thickness

k_c = quality coefficient of concrete

k_e = coefficient of environment

k_a = coefficient of active corrosion

The values of 7.56, 0.85 and 4.0 were used for k_c , k_e and k_a respectively in the report.

Andrade et al. [21] suggested a model to compute residual diameter of corroding steel rebar in the year 1989. Based on the percentage reduction in cross sectional area of the steel rebar, and corrosion current density, the residual life of a structure is estimated, as can be seen from fig. 10

$$\varphi(t) = \varphi_i - 0.023 I_{\text{corr}} \cdot t \quad (8)$$

where

$\varphi(t)$ = residual diameter of rebar at time, t (mm)

φ_i = original diameter of rebar (mm)

I_{corr} = corrosion rate ($\mu\text{m}/\text{year}$)

A factor of 0.023 is used to translate ($\mu\text{A}/\text{cm}^2$) to ($\mu\text{m}/\text{year}$). Based on the combination of laboratory, outdoor exposure and field studies, Clear [17] suggested the following relationship (which assume constant corrosion rates with time) between corrosion rates (I_{corr}) and remaining service life.

1. I_{corr} less than $0.5 \mu\text{A}/\text{cm}^2$ - No corrosion damage is expected.
2. I_{corr} between 0.5 and $2.7 \mu\text{A}/\text{cm}^2$ - Corrosion damage is possible in the range of 10 to 15 years.
3. I_{corr} between 2.7 and $27 \mu\text{A}/\text{cm}^2$ - Corrosion damage expected in 2 to 10 years.
4. I_{corr} in excess of $27 \mu\text{A}/\text{cm}^2$ - Corrosion damage expected in 2 years or less.

These criteria are in agreement with the predictions given by Andrade [10].

Clifton [22] arrived a model in the year 1991 with time order concept, considering the degradation processes. The time order approach provides a technical basis for extrapolation [22]. It is assumed that the degradation depends on the environment, geometry of the structure, and properties of concrete. The time to failure is empirically related to the amount of degradation, (A_d) and is given by

$$A_d = K_d \cdot t_y^n \quad (9)$$

and $t_{yf} = (A_d / K_d)^{1/n}$

where

A_d = amount of accumulative deterioration at time t_y (years)

K_d = degradation constant

N = time order

t_{yf} = time to failure

The time order constant, n depends on rate controlling process; and can be obtained by a theoretical analysis of rate controlling processes by modeling degradation processes and empirically from accelerated degradation tests. Table 1 shows the values of n obtained from models.

Seki et al. [23] predicted service life of reinforced concrete structures, considering the end – of – life on formation of longitudinal cracking and spalling of cover concrete.

$$t_{sp} = k_t (L/d \cdot \gamma) \quad (10)$$

where

t_{sp} = time required for longitudinal cracking and spalling

k_t = Coefficient (≈ 0.8)

L = Concrete cover

γ = Corrosion rate ($\gamma = \gamma_{ab}$ or $\gamma = \gamma_{cl}$)

γ_{ab} = Corrosion rate in absence of chloride

γ_{cl} = Corrosion rate in presence of chloride

d = Design diameter of rebar

Once spalling of cover concrete occurred, neither chloride nor carbonation can influence corrosion process but only atmospheric humidity may influence corrosion process. In such case the corrosion rate is estimated as

$$\gamma = 0.005RH - 0.29 \text{ (mm/year)} \quad (11)$$

where

RH is the relative humidity (%)

Morinaga [24] has developed an empirical model in 1990 to compute steady state corrosion period to predict service life of structures based on amount of corrosion products causing expansion cracking of cover concrete.

$$Q_{cr} = 0.602 \left[1 + \frac{2C_{cr}}{D} \right]^{0.85} D \quad (12)$$

$$\text{and} \quad t_{cor} = \frac{Q_{cr}}{J_r} \quad (13)$$

where

Q_{cr} = amount of corrosion product around the steel rebar when the cover concrete cracks ($\times 10^{-4} \text{ gm}/\text{cm}^2$)

C_r = thickness of cover concrete

D = original diameter of steel rebar

t_{cor} = steady state corrosion period ; and

J_r = instantaneous corrosion rate per unit surface area of rebar

Wang et Al. [25] developed a model in 1993, based on finite element analysis to compute the thickness of corrosion product (Δ) to cause cover concrete to crack . The expansion coefficient of rust products (γ) in terms of cube strength of concrete is given as

$$\gamma = \frac{\Delta}{H} = 0.33 \left(\frac{D}{C_r} \right)^{0.565} f_{cr}^{1.436} \quad (14)$$

Further, the value of H can be used to estimate the time necessary to develop longitudinal cover cracking due to corrosion expansion. The steady state corrosion period is computed as

$$t_{cor} = \frac{H}{P_r} \quad (15)$$

where

γ = expansion of coefficient of rust products

Δ = thickness of corrosion products

H = depth of rebar penetration due to corrosion

t_{cor} = steady state corrosion period

P_r = penetration rate of rebar due to corrosion

D = original diameter of rebar

C_r = thickness of cover concrete

f_{cu} = cube compressive strength of concrete

The rate of $\left(\frac{\Delta}{H} \right)$ found to be nearly constant and equal to 7.6 and thus the model suggested can be used only in conjunction with the finite element model requiring the determination of Δ [1].

Alonso et al. [26] had suggested a model based on the Tuutti's suggestion that when a structural element start to show visible cracks of 0.1 to 0.2 mm width , is the final state or end of service life.

$$t = (t_o + t_p) = \frac{x^2}{K_c^2} + \frac{P_{lim}}{I_{corr}^n} \quad (16)$$

where

t = service life

t_o = initiation period

t_p = propagation period

K_c = carbonation factor

x = cover thickness

P_{lim} = limit attack penetration

I_{corr} = corrosion rate ; and

n = attenuation factor

As the corrosion rate and carbonation coefficient will vary with climatic conditions and relative humidity, the concept of "wetness period" is introduced instead of simply considering an I_{corr} value when the relative humidity is 80%. Accordingly, the modified equation for service life is suggested as

$$t = \frac{x^2}{K_c^2(1-\omega)^2} + \frac{P_{lim}}{\omega I_{corr}^n} \quad (17)$$

Where ω is the percentage of the year in which the relative humidity is more than 80%

Another model suggested by Alonso et al. [26] estimates the service life as

$$t = t_i + t_p = K_c \sqrt{t} + \frac{PL}{CR} \quad (18)$$

where

t = service life (years)

t_i = corrosion initiation time (years)

t_p = corrosion propagation time (years)

K_c = carbonation coefficient

PL = penetration limit (mm)

CR = corrosion rate ($\mu\text{m}/\text{year}$)

In 1998, Youpingliu et al. [9] suggested the service life of structure as given below by considering concrete cover cracking is the end of life.

$$t_{cr} = \frac{W_{crit}^2}{2K_p} \quad (19)$$

$$W_{crit} = \rho_{rust} \left\{ \pi \left[\frac{c f_t}{E_{ef}} \right] \left(\frac{a^2 + b^2}{b^2 - a^2} + \gamma_c \right) + d_o \right\} D + \frac{W_{st}}{\rho_{st}} \quad (20)$$

$$K_p = 0.098 (1/\alpha) \pi D i_{cor} \quad (21)$$

where

t_{cr} = time to corrosion cracking of concrete cover

W_{crit} = critical amount of corrosion product

k_p = rate of rust formation

ρ_{rust} = density of corrosion product

f_t = tensile strength of cover concrete

D = diameter of steel rebar

d_o = thickness of pore band around steel / concrete interface

E_{ef} = effective elastic modulus of concrete

$$[E_{ef} = E_c / (1 + \phi_{cr})]$$

E_c = elastic modulus of concrete

ϕ_{cr} = creep coefficient of concrete

$a = (D + 2d_o) / 2$

$b = c + (D + 2d_o) / 2$

c = cover thickness

$\alpha = 0.523$ when corrosion products are considered as $Fe(OH)_3$; and 0.622 when corrosion products are considered as $Fe(OH)_2$

ρ_{st} = density of steel rebar

γ_c = Poisson's ratio of concrete

I_{cor} = the annual mean corrosion rate (mA/ft²)

$$(1 \text{ mA/ft}^2 = 1.07 \text{ } \mu\text{A/cm}^2)$$

Khatri *et al.* [27] considered diffusion of chloride ions into concrete according to Fick's second law and suggested an equation for service life as given below. It can be seen that the service life is inversely proportional to the diffusion coefficient.

$$t = \frac{x^2}{4Da} \left\{ \text{erf}^{-1} \left[\frac{C_t(X_1t) - C_s}{C_s} \right] \right\}^{-2} \quad (22)$$

where

t = service life (in seconds)

X = cover thickness (mm)

Da = chloride diffusion coefficient (m²/s)

$C_t(X_1t)$ = chloride concentration at cover depth (% by mass of concrete)

C_s = surface chloride concentration at time, t (% by mass of concrete)

erf^{-1} = inverse error function

These are some of the models reported by various researchers, on predicting the residual service life of a structure. Each model will have its own merits and drawbacks. Although it is impossible to predict the service life very accurately, these existing physical and empirical models will be certainly very helpful to predict the life to some extent so that any repair or rehabilitation can be attended well in advance to save the structure and the valuable lives from the failure.

V. Discussion

❖ In almost all the models, corrosion damage caused by chloride diffusion and carbonation of concrete are considered seriously and accordingly the corrosion rate of steel rebars are incorporated in the models. Therefore the measurement of corrosion rate has got a significant role to play [28]. Corrosion rate of embedded steel in concrete will vary with climatic conditions particularly the relative humidity and temperature are the major factors determine the corrosion rate. Therefore, a single corrosion rate at any one particular climatic condition will not give reliable prediction. Also the corrosion rate varies with the geometry of the structure, properties of concrete and on specific degradation process. It is suggested that when the corrosion rate is to be measured, a time period has to be selected to represent the corrosion rate as an annual average value for estimating service life.

❖ Although these approaches provide reasonable value, large number of uncertainties is associated with. For example, the threshold free chloride level to initiate corrosion of steel in concrete is not fully and precisely understood. The other

Uncertainties are such as surface chloride concentration, chloride diffusion constant and variability of loadings on structure. The surface chloride concentration on concrete will vary time to time. As the concrete is of heterorganic material, the chloride diffusion rate may not be uniform over a surface.

❖ In some models, the initiation period is considered longer duration than the propagation period. This may not be same for all structures and at all environments. Therefore, the initiation and propagation periods can be modeled as random variables, which could be defined by their statistical distribution.

❖ Once corrosion of rebar commenced the corrosion products form around the steel rods. At initial stages, these rust products will diffuse into the pores and voids around the bar and block. This in turn hinders oxygen supply to the cathodes. This may have some effect on corrosion rate measurements.

❖ Models based on the assumption that the rust products around the steel cause corrosion cracking of cover concrete by expansion is considered appropriate as long as the structure is unsaturated. When a structure

is saturated, the corrosion will be stopped by oxygen starvation. A highly saturated structure can corrode rapidly without the signs of cracking, as the ferrous ions will stay in the solution without forming a solid rust product around the rebar.

❖ During corrosion propagation period, the effect of fatigue and bond deterioration by corrosion should be considered in the models.

❖ The effect of steel stress is not considered in most of the models. During propagation period, the reinforcements gradually loses its cross section and the steel stress level is increased and a stage may be reached that yielding of steel can take place. This is particularly important in under- reinforced concrete structures.

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