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Essential parameters for strength-based service life modeling of reinforced concrete structures—a review

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ABSTRACT: While there are a number of carbonation-induced service life models and extensive data that has been presented in the literature, most do not capture all the necessary parameters to allow their universal application to reinforced concrete structures. Results in the literature generally show an existence of a strong fundamental relationship between carbonation and compressive strength of concrete, but hardly any model has been successful in developing a universal functional relationship for service life prediction. It is often the case that models developed on the basis of a particular data set fail to apply when treated to another data from other authors. These issues indicate the underlying complexity of attempting to determine and emerge the empirical or mathematical functions capable of adequately capturing the key influential parameters affecting observed performance. In a critical review of literature, a variety of parameters in the strength-based models are compiled for a range of potentially plausible models and then examined. Specific interest has been given to models that account or have the potential to account for complex cementitious systems, various types of climates or field exposure conditions. It is found that there is potential to introduce robustness into few selected models that seem, from the literature examination, to closely relate to service life situations and/or needs in Sub-Saharan Africa, among other regions.

1 INTRODUCTION

In the recent times, the use of concrete has grown in application to residential, industrial, commercial, innovative structures and constructions. It is pertinent to know that many concrete structures fail to achieve their expected service life due to the limited scientific knowledge on service life design. These issues are some of the major challenges confronting the modern construction industry. Consequently, billions of money is spent annually on repair and maintenance of ageing concrete infrastructure. In 1990, the National Research Council report in the U.S. gave an estimated cost of $2 to $3 trillion required over the next 20 years to repair concrete structures deteriorated by reinforcement corrosion (Hoff 1991) Also, in Sub Sahara Africa and many other parts of the world, a lot of money is spent annually on repairs and rehabilitation of existing concrete structures.

However, the most pronounced form of concrete deterioration in Africa is carbonation-induced corrosion, which normally develops into cracking, delamination and spalling of concrete cover. Several experimental investigations into parameters and design variables that influence reinforcement corrosion have been developed into various models, as listed in Table 1. However, most of these models have been developed basing on short and long-term experiments and their results often show poor agreement with or large variations from the actual real life observations. Also in practice, durability design of concrete is prescriptively done by specifying a minimum concrete cover depending on the exposure conditions, water/cement ratio, compressive strength, corrosion resistance, and diameter of the steel bars. However, there are many uncertainties associated with these parameters which make service life prediction difficult in practice. This paper reviews some potential useful models available in the literature while identifying the essential parameters, limitations and problems in order to make suggestions for an improved robust model that can be suited to application in Sub-Saharan Africa (SSA), among other regions.

2 TUUTTI'S CONCEPT MODEL

The different empirical models for reinforcement corrosion, as proposed by various authors (see Table 1) are based on the concept model initially introduced by Tuutti (1982), where service life can be idealized as two-phased stages, namely the initiation time followed by propagation time of

Table 1. List of models and parameters in concrete carbonation.

	ENVIRONMENTAL					MATERIAL				TIME DEPENDANT			
Models	Exposure	CO_2	Diffusion coefficient	RH	Temp.	w/c	Cement type	a/c	Cement content	Curing	Strength	Time	Geometry
Tuutti	X	X	X	X	X	X						X	
Papadakis et al	X	X	X	X		X		X	X	X		X	
Parrot		X	X	X								X	
Liang et al		X	X									X	X
Khunthongkeaw			X									X	
Hakkinen	X			X			X				X	X	
LeSage de Fontnay	X					X				X		X	
Loo et al		X			X	X			X	X	X		
Kokubu and Nagataki	X					X	X			X	X	X	
Bob		X		X							X	X	
Matsuzawa et al		X		X	X	X						X	
Nagatakis et al	X					X	X		X	X	X	X	
Vesikari						X	X			X	X		

corrosion before an unacceptable corrosion damage level is reached, as in equation 1 (Tuutti 1982).

$$t_l = t_i + t_p \tag{1}$$

Where t_l is the service life in years, t_i is corrosion initiation time in years, and t_p is the corrosion propagation time in years. The initiation and propagation period of corrosion is influenced by several factors which are categorized under concrete quality and properties along with environmental condition. However, Tuutti's model give fundamental insights into corrosion damage but the influence of these parameters on sub-stages of damage is hardly dealt with, this makes differentiation between structural response and corrosion-induced damage in the propagation stage difficult (Otieno et al 2011). However, the approach is favoured by most researchers due to its practicality and simplicity due to easily measured parameters.

3 CORROSION INITIATION MODELS

Corrosion initiation time begins when the high pH level of the concrete reduces through formation of carbonates by oxidation process. The process destroys the protective oxide film around the steel for the corrosion to take place. However, the corrosion initiation time can be expressed as thus (equation 2). However, this is a diffusion process and many theoretical models tend to adopt the Fickian's time function, where each model is based on different set of parameters and experimental design. A range of some interesting models are reviewed in the subsequent sections.

$$t_i = \left[\frac{d}{K_c}\right]^2 \tag{2}$$

Where K_c = carbonation rate in mm/√yr, d = depth of carbonation in mm.

3.1 *Carbonation rate and diffusion coefficient*

Papadakis et al (1991) worked on an experimental investigation of the physico-chemical processes of concrete carbonation. The report suggests that the change in carbonation depth with time is a function of the chemical composition of cement, mix proportions, CO_2 concentration and the relative humidity. However, the model predicts carbonation depth in concrete of different binders (equation 3). The controlling parameters are exposure time and effective diffusivity as dependant of porosity and relative humidity (equation 4)

$$d = \left(\frac{2[CO_2]^2 D_{e,CO_2}}{[CH]+3[CSH]} t\right)^{\frac{1}{2}} \tag{3}$$

$$D_{e,CO_2} = (1.64x10^{-6})\varepsilon_p^{1.8}\left[1-(RH/100)\right]^{2.2} \tag{4}$$

D_{e,CO_2} is the effective diffusivity and ε_p is the hardened cement paste porosity obtained through an expression

$$\varepsilon_p = \varepsilon \frac{1+(w/c)(\rho_c/\rho_w)(a/c)(\rho_c/\rho_a)}{1+(w/c)(\rho_c/\rho_w)}$$

Where ε is the total porosity of the concrete, a/c is the aggregate/cement ratio, and ρ_c/ρ_w is the ratio of concrete density and water density, while ρ_c/ρ_a expresses the ratio of concrete density and aggregate density.

3.2 *Carbonation depth and concrete geometry*

Liang et al (2002) worked on mathematical modeling of concrete carbonation using Fick's first and second laws of linear diffusion equations to express the three dimensional equation of conservation of mass for prediction of carbonation depth at corners and general surfaces of concrete structures. However, based on the model it could be rightly said that carbonation is greater at corners than at the flat surfaces of concrete (equation 5). This shows the relevance of structure geometry in carbonation model and as an indication that measurement based on flat concrete surface may not provide the worst scenario for service life prediction.

$$\underbrace{d = 2\left(\frac{DC_0}{m_0}t\right)^{\frac{1}{2}}}_{\text{i. surface}} \quad \underbrace{d = \left(\frac{2DC_0}{m_0}t\right)^{\frac{1}{2}}}_{\text{ii Corner surface}} \qquad (5)$$

Where C_0 is the CO_2 concentration at the outer edge of concrete, m_0 is the unit volume (kg/m^3) of absorbed CO_2 and D is the diffusion coefficient.

3.3 *Carbonation rate and binder type*

Carbonation is greater in concrete containing cement extender. This is attributed to low portlandite content of the mixture which reduces the binding activity of carbon dioxide. However, this effect decreases as the strength of concrete increases with age following the square root law (equation 6 & 7) (Hakkinen 1993b, Loo et al. 1994). Hakkinen 1993b work extensively considering different types of binders and determining the shift factors but the application of the shift factors in modeling may not be generalized since mineralogical compositions of extenders vary from one country or source to another. This becomes necessary to derive various constants applicable to binder material systems used in the region.

$$d = K_c * \sqrt{t} \qquad (6)$$

$$K_c = a * f^b \qquad (7)$$

Where f is the compressive strength in N/mm^2, a represent constant and b is the shift factor.

3.4 *Carbonation depth and curing*

LeSage de Fontenay (1985) modeled the carbonation relationship between the 28-day water cured concrete and the air-cured concrete (equations 8 and 9). However, the effect of curing beyond 14 days on carbonation rate has been reported to be negligible (Dhir et al. 1989; Kokubu & Nagatakis 1989; Calavera et al. 2011 and Nagatakis et al. 1986). This presents a challenge on time and material optimization in concrete construction, which in turn an important variable in modeling the initiation time of corrosion. However, there is little or no literature on the effect of different types of curing on carbonation depth for purposes of assessing service life. Such understanding would be relevant to precast concrete.

$$d = 0.43(w/c - 0.4)\,(12(t-1)^{0.5} + 0.1 \quad \text{28-day cured} \quad (8)$$

$$d = 0.53(w/c - 0.2)(12t)^{0.5} + 0.2 \quad \text{Non-cured} \quad (9)$$

3.5 *Carbonation rate and exposure conditions*

As already mentioned, exposure condition is a major factor in determining carbonation in reinforced concrete. It is also a reason why codes and specifications classify the exposure conditions into mild, moderate and severe. Under any of the exposure classes, a structural member can then be categorized into: (i) sheltered (ii) unsheltered from rain. These conditions are defined in SABS 0100-1: 1992, while others have proposed definitions. The effects of the exposure condition to concrete carbonation in SSA have not been extensively investigated compared to studies done in other parts of the world such as Europe, Asia and America. Kokubu and Nagataki model shows that carbonation of indoor exposure is greater and lies between 2 to 4 times that of an outdoor exposure, irrespective of curing effect, water/cement ratio, binder type, cement content and age. Moreover experimental results of most researchers have been found to follow the general trend that carbonation rate is higher with increase in temperature of curing and between humidity of 50–65% similar to many other reports.

3.6 *Carbonation rate and mix proportion*

The w/c is fundamental in mix design of concrete and can be expected to have a strong effect on carbonation. Several models have been proposed basing on the impact of w/c on carbonation depth. From the various literatures (Loo et al 1994; LeSage de Fontena 1985; Kokubu & Nagatakis 1989; Parrot 1987), it is recognized that carbonation strongly increases with increase in the w/c along with reduction in cement content, irrespective of the exposure conditions and curing duration (see Table 3). From some of the models considered in this paper, it can be seen that most equations may be reasonably consistent in their predictions (Parrot 1987). A case in point is the 20 year long—term study by Kokubu & Nagataki (1989), whose findings are similar to the results of the accelerated carbonation study of Loo et al (1994) where carbonation depth is modeled as $\alpha[(w/c)-\beta]$, where α and β are constants for specific exposure condition.

However, the correlation between these constants is not strong enough for model acceptability due to factors such as batching, absorption properties of aggregates, compaction and form work types etc. The effect of aggregate/cement was considered by Papadikis et al (1992) as one of the compositional parameters that influence the concrete carbonation depth in an extensive model shown in equation 10. The limitations in practice are the complexity in evaluating the CO_2 diffusion as well as the involvement of too many parameters. However, the use of 28 day compressive strength is a possible way of overcoming these practical difficulties. Strength is more convenient to measure, a good indicator of concrete quality, relates well with CO_2 diffusivity and capillary porosity which are all influential factors on carbonation rate (Loo et al. 1994).

$$d = 350\left(\frac{\rho_c}{\rho_w}\right)\frac{w/c-0.3}{1+(\rho_c/\rho_w)w/c}\left(1-\frac{RH}{100}\right) \left\{\left[1+\left(\frac{\rho_c}{\rho_w}\right)\frac{w}{c}+\frac{\rho_c}{\rho_w}\left(\frac{a}{c}\right)\right]yCO_2 t\right\}^{0.5} \qquad (10)$$

Where ρ_c and ρ_w are the unit weights of concrete and water, w/c and *a/c* are the water/cement and aggregate/cement ratios respectively.

3.7 *Carbonation rate and concrete strength*

The correlation between carbonation coefficient and strength in many of the prediction models has been reviewed by many authors and several equations have been proposed from experimental works, basing on accelerated or long-term tests. Expectedly, there is consistency between strength-based and w/c—based models. Loo et al. (1994) proposed the following three different models based on experimental work, $K_a = A(B-f)$, $K_a = e^{a(1-bf)}$, $K_a = \frac{\alpha}{\sqrt{f}} - \beta$, where A, B, a, b, α and β are coefficients. However, there is good correlation when the square root of compressive strength is used (Kokubu and Nagatakis 1989 & Loo et al. 1994). Hakkinen (1993 part 2) in an experimental work shows the existence of an inverse relationship between the carbonation rate and compressive strength as $v = af^{-b}$, where a and b are dependent on the binder type. The mathematical function shows universality of the model for application to different cementitious systems. Vesikari (1988) proposed a model where the carbonation rate is a square function of strength as shown in equation 11. The function contrasts with Kokubu & Nagatakis (1989) model shown in equation 12, where the carbonation rate is expressed as a square root function of strength. While both models do not account for exposure time to CO_2, they seem to be intended for use, irrespective of the binder type.

$$K = 0.0063(54.5 - f_{28})^2 + 1.6 \qquad (11)$$

$$K = A - B\sqrt{f_{28}} \qquad (12)$$

The constants A and B in equation 12 account for the time response of the cementitious system which are expected to adjust along with replacement of PC with extenders. However, carbonation generally shows a good linear correlation with the square root of compressive strength and can be used to predict long term carbonation depth.

3.8 *Carbonation prediction models at multivariate level of parameters*

Loo et al (1994) proposed a prediction model from accelerated experimental investigation. Due to the complex nature of the interaction of the various parameters, the model attempts to combine the controlling factors at multivariate level (equation 13) It gives insight into correlation between carbonation coefficient and the key parameters However, the application is limited PC concrete. Moreover, the model is similar to RILEM 14 model which evaluates the carbonation depth considering multivariate controlling factors (equation 14), this yield better result compare with using few parameters.

$$K_c \alpha f_{28}^a C_o^b e^{cT} t_{wc}^d + \beta \qquad (13)$$

$$K_c = C_{env} C_{air} a(f_{cm} + 8)^b \qquad (14)$$

Where t_{wc} accounts for curing duration in days, e^{cT} accounts for the effect of temperature, α and β are the constants and a, b, c and d are the shift factors. C_{env} represent the environmental coefficient, C_{air} is the air content coefficient while f_{cm} is the characteristic compressive strength of concrete in N/mm^2.

4 CORROSION PROPAGATION

Corrosion propagation starts when the passive film is destroyed as a result of reduction in the pH of concrete due to carbonation. The propagation is determined by the rate at which reinforcement corrosion proceeds. The damage induced by the expansion of corrosion products includes cracking, delamination and spalling of the concrete cover, which form the second phase of Tuutti's model. Tuutti's model is a generalized concept which does not identify the sub-stages of corrosion damage (Otieno 2011). However, propagation time is a short period compared to the initiation time and is one of the reasons that some models do not incorporate the propagation phase into their service life. However, this phase is largely controlled by availability of oxygen, temperature, anode and cathode area and resistivity of concrete (equation 15).

$$r = k_r \frac{I}{A} \tag{15}$$

Where r is the corrosion rate in cm/sec, I is the corrosion current in coulomb/sec, k_r is interdependency coefficient of corrosion rate which may be insignificant, and A is the surface area affected by the corrosion in cm^2.

Tuutti (1982) ascertained that corrosion rate is dependent mostly on environmental factors rather than material properties and greater when pores are partially saturated and becomes zero at less than 40%RH (Balafas and Burgoyne 2010). However, corrosion rate is bound to increase at elevated temperature as molecules become more agitated. This was investigated by Tuutti (1982) who increased temperature from 0 to 20°C, and found that corrosion increased ten times. In SSA, temperatures can be as high as 30°C in certain regions and seasons, which could imply higher corrosion rates. However, Andrade et al. (2009 & 2006) pointed out that increase in temperature may lead to higher evaporation of the pore water with the immediate effect on concrete resistivity. It is also evident that many authors rarely incorporate temperature effect into their models considering it to less important.

Rashid et al. (2010) investigated the effect of concrete strength on corrosion resistance and found that the latter increases along with increase in strength. Hence, concretes of higher strengths may generally provide better resistance to reinforcement corrosion thereby lengthen the propagation time, hence the service life (Andrade et al 2006). The equation 16 proposed by Alonso & Andrade (1993), gives the propagation period of the corrosion process as a function of change in bar radius in direct relation to corrosion rate.

$$t_p = \frac{\Delta R_{max}}{r} \tag{16}$$

Where t_p is the propagation time, ΔR_{max} is the maximum loss of radius of the steel bar, and r represents rate of corrosion in mm/year.

However, rate of corrosion is determined by the environmental factors and concrete material properties such as steel diameter, concrete cover, and geometry of concrete surface and factors such as exposure time and cover thickness play no significant role once corrosion has commenced (Liang 2005). A model describing corrosion propagation cracking time is given in equation 17 (RILEM 14)

$$t_p = \frac{0.08C}{Dr} \tag{17}$$

Where D is the bar diameter in mm, and C represents cover thickness in mm.

It can be seen that C/D is directly proportional to propagation time which affect the time to surface cracking. However, in related work of Zhou et al. (2005), cracking initiation was not affected by changing the ratio C/D but a slight increase in compressive strength of concrete significantly improved crack initiation. However, the mechanical effects of steel corrosion such as spalling and delamination, and their impacts on serviceability and safety of the reinforced concrete structure is still a subject of research interest. However, there are uncertainties associated with these parameters which make quantification of propagation stage of service life difficult, although this is smaller compare to corrosion initiation time. In the literature, several methods of analysis which include finite element, boundary element, differential method, and Monte Carlo simulation have been used to examine the impact of various parameters on service life. Analysis suggests that limiting criteria for the failure modes were considered as functions of design variables of the concrete cover. Cracking and spalling were considered functions of cover/bar diameter ratio while delamination mainly depends on the bar spacing/diameter ratio (Zhou et al. 2005).

5 PARAMETER VARIABILITY IN MODEL DEVELOPMENT

In the past, deterministic approaches have been applied to evaluate the service life of reinforced concrete structure. But this approach is severely inadequate as it is unable to evaluate the risk of failure associated with the random nature and uncertainties of variables (Ekolu 2010). The probability-based method that takes into consideration the uncertainties inherent in the random variables for service life in order to predict their effect is more appropriate. However, stochastic approach will be made use of in an investigation into a robust, practically effective service life model for conditions relevant to SSA.

6 CONCLUSION

In conclusion, it is evident that the relationship between the main service life parameters can be universally expressed mathematically to depict the deterioration process in some form of theoretical or empirical model such as the plethora of proposals found in the literature. Most of the service life models are based only on the initiation time to corrosion and give no consideration to corrosion propagation. It has been argued that ignoring the propagation stage may incorporate conservativeness and a safety margin into the model. Another reason favoring the argument is the relatively longer corrosion initiation period compared to the subsequent propagation time which typically lies between 5 to 10 years.

Furthermore, the advent of new binder material systems coupled with new construction technologies rules out suitability of most of the models discussed in the foregone. Extensive coverage of binders and exposure conditions as found in Sub Sahara Africa, and incorporation of the effects of new construction materials and technologies are necessary requirements for an effective service life model. However, this may be difficult, because of high scatter in service life that may result. The execution of such a model should take into account the randomness of the parameters due to their complex interactions. In search of a robust, practically effective service life model, the foregone considerations call for application of the stochastic method along with multivariate analyses.

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