ESSENTIAL PARAMETERS FOR STRENGTH-BASED SERVICE LIFE MODELLING 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 [1]. Also, in South Africa and many other parts of the world, a lot of money is spent annually on repairs and rehabilitation of existing concrete structures. The dilemma that has been argued by researchers is the definition of serviceability limit state for evaluation of the service life of the concrete structure. The difficulty is compounded by a myriad of factors such as ageing, global warming, natural disaster, lack of adequate impermeability of concrete, steel corrosion, and internal material reactions [2]. Li et al. [3] suggested that most of the failures in reinforced concrete can be attributed to the emphasis on using factors of safety related to structural strength design with only limited consideration being given to serviceability.

The most pronounced form of concrete deterioration in Africa is carbonation-induced corrosion and it occurs when carbon dioxide (CO_2) in the air diffuses into concrete pores saturated with calcium hydroxide. A chemical reaction takes place under some influences leading to formation of an acidic medium, reducing the pH value from over 12.6 to below 10. Consequently, calcium carbonate is formed and in the process, the protective oxide film around the steel reinforcement breaks down marking an important condition for initiation of the corrosion process and its propagation to the level at which expansive pressure is built up due to the corrosion products. Eventually the concrete cover develops cracking, delamination and spalling. Several investigations into parameters and design variables that influence reinforcement corrosion have been developed into proposed models, as listed in Table 1.

The models illustrate the corrosion process and or attempt to quantify service life. Most of these models have been developed basing on short-term accelerated laboratory 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 uncertainties associated with these parameters which are associated to: (i) heterogeneity of concrete, (ii) variability in cover depth, which also hinges on quality control and workmanship during construction, (iii) variability in air CO_2 concentration, relative humidity and temperature of the environment [4]. These factors eventually lead to considerable uncertainty in service life prediction.

In developing a service life model, it is essential to first examine the accuracy and suitability of corrosion model that will account for essential governing parameters relevant to the local operating environment. In addition, a useful model should be simple but not simplistic, practically easy to execute and should utilize readily available or easily obtainable data. It thus becomes important to consider accelerated test results in conjunction with long - term data from field tests or from existing structures. It is also appropriate to apply stochastic methods of service life evaluation which consider statistically treated variability along with sensitivity analyses of the influential parameters in order to evaluate the impact of their uncertainties on service life [4-7].

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.

Table 1 List of models and parameters in concrete carbonation

			ENVIRONMENTAL			MATERIAL					TIME DEPENDANT		
Models	Exposure	CO2	Diffusion coeff.	RH	Temp.	w/c	Cement type	a/c	Cement content	Curing	Strength	Time	Geometry
Tuutti	X	X	X		X	X						X	
Papadakis et al	X	X	X			X			X	X		X	
Parrot		X	X									X	
Liang et al		X	X									X	X
Khunthongkeaw			X									X	
Hakkinen	X						X				X	X	
LeSage de Fontenay	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	
Matsuzawa et al		X			X	X						X	
Nagatakis et al	X					X	X		X	X	X	X	
Vesikari						X	X			X	X		

2. TUUTTI'S CONCEPT MODEL FOR THE SERVICE LIFE OF CONCRETE STRUCTURES

The different mathematical or empirical models for reinforcement corrosion, as proposed by various authors (see Table 1) are based on the concept model initially introduced by Tuutti [8], where service life can be idealized as two-phased stages, namely the initiation time followed by propagation time of corrosion before an unacceptable corrosion damage level is reached, as shown in Figure 1.

The time-based process can be written as

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

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. It can be seen from Figure 1 that the initiation period of corrosion primarily represents the time for CO_2 diffusion into concrete to reach the level of steel reinforcement. This time period is in turn influenced by several factors especially the cover depth, concrete quality determined by the water/cement ratio or strength, curing regime, and cement type.; the environmental conditions.

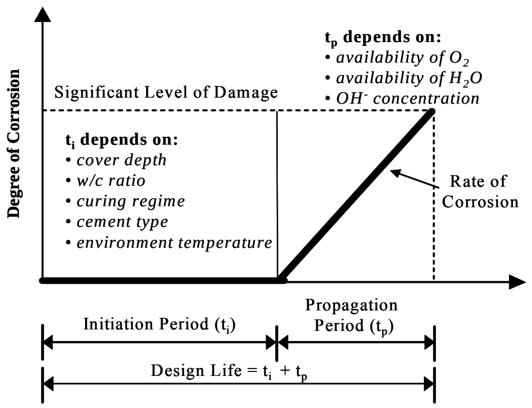

Figure 1: Degree of corrosion and design life of concrete structure [8]

Although Tuutti's model gives fundamental insights into corrosion damage in reinforced concrete it does not give sub-stages of the damage propagation, which are related to cracking, spalling and delamination of concrete cover before failure. The model also suggests no differentiation between structural response and corrosion-induced damage in the propagation stage [5]. Tutti's approach is favoured by most researchers due to it's practicality and simplicity. There are other similar models such as Tomosawa et al. [9] and Matsuzawa et al. [10] who considered temperature, humidity, and CO2 concentration in mathematical modeling of service life. The Bazant model [11] is also accurate as Tuutti's model but the former is more theoretical and applies mainly to marine structures. The Bazant model is complex and considers oxygen diffusion, electrical conductivity, cathode and anode areas. These factors are difficult to measure in practice when modeling corrosion rate.

Corrosion-induced cracking has been also been analysed in the literature and modeled as function of critical parameters such as material properties especially the tensile strength of concrete, bar spacing, ratio of concrete cover to bar diameter, operating environment and corrosion rate [3, 9, 10, 11]. On the other hand, the duration of the propagation time depends principally on corrosion rate as affected by many factors such as concrete geometry (cover depth and bar diameter), concrete quality and corrosion current [9]. While many models attempt to predict the time to cracking of specimens under accelerated corrosion, the influence of environmental conditions and the other parameters on sub-stages of damage such as spalling, delamination and structural collapse is hardly dealt with [10].

3. CORROSION INITIATION MODELS

Corrosion initiation time begins when the high pH level of the concrete reduces through formation of carbonates by oxidation process as a result of carbon dioxide diffusion into the concrete pores. This process destroys the protective oxide film around the steel, allowing the corrosion to take place. The depth of carbonation is known to be related to the square root of time as given in equation 2. Accordingly, the corrosion initiation time can be expressed as given in equation 3.

$$d = K_c t^{1/2} \qquad (2)$$

$$t_i = \left[\frac{d}{K_c}\right]^2 \qquad (3)$$

Where K_c is the carbonation rate in mm/√yr, d is the depth of carbonation in mm, and t_i is the corrosion initiation time in years. The assumptions under which the equations hold are the uniformity of pore structure and absence of surface cracks [12]. But the accuracy of the square root - time relationship in predicting the service life in practice has been questioned by some authors suggesting a linear relationship to be more suited for use in practice [12, 13]. Since CO_2 ingress into concrete is a diffusion process, theoretical models tend to adopt the Fickian's time function in describing service life while the carbonation rate, which is a constant in the equation, is dependent on concrete quality and environmental conditions [15, 17]. There are several empirical and analytical models describing the carbonation rate, each model being based on different sets of parameters and experimental designs. A range of some interesting models are reviewed in the subsequent sections.

3.1. Carbonation rate and diffusion coefficient

A model based on the assumption of Fickian process of diffusion in predicting the time to corrosion initiation was proposed by Zhang and Lounis [4], where the carbonation rate K_c is expressed as thus:

$$K_c = \left(\frac{2D_c(C_1 - C_2)}{a}\right)^{\frac{1}{2}} \qquad (4)$$

Where d is the depth of carbonation in mm, D_c is the diffusion coefficient in m^2/s for a given moisture distribution within the concrete pores, $C_1 - C_2$ is concentration difference of carbon dioxide, a represents the amount of alkaline substance in the concrete. Equation 4 was developed on the assumption of steady state ingress of carbon dioxide which means that both the diffusion coefficient and the level of alkalinity depend largely on the concrete quality [9].

The effects of water/cement ratio, exposure conditions (sheltered and unsheltered from rain) and time were studied by Tuutti [8] in an experiment. The carbonation rate was found to be greater in concrete sheltered from rain than in concrete under the unsheltered exposure. This effect was attributed to low solubility of carbon dioxide in water. Thus the influence of pore water in carbonation as relating to relative humidity and temperature, underscore the importance of environmental factors. The carbonation rates under sheltered and under unsheltered conditions are different, as proposed by Vesikari [9] in equations 5 and 6.

Sheltered exposure:

$$k = 26(w/c - 0.3)^2 + 1.6 \qquad (5)$$

Unsheltered exposure:

$$k = (w/c - 0.3)^2 + 0.7 \qquad (6)$$

Papadakis et al. [19] worked on an experimental investigation of the physico-chemical processes of concrete carbonation. Concretes of different mix ratios were produced and subjected to usual ranges of temperature, relative humidity and CO_2 concentration as found in practice. There was a good agreement in results from analytical expression and test results obtained from accelerated exposure. These reports suggest that the change in carbonation depth with time is a function of the chemical composition of cement, mix proportions of concrete, CO_2 concentration in the atmosphere, and the relative humidity. This model as given in equation 7 predicts carbonation depth in concrete of different binders for a constant relative humidity as a means of maintaining the degree of pore saturation.

$$d = \left(\frac{2[CO_2]^2 D_{c,CO_2}}{[CH] + 3[CSH]} t \right)^{\frac{1}{2}} \qquad (7)$$

These parameters are considered to be the controlling factors for effective diffusivity as dependant of porosity *viz:-* $D_{c,CO_2} = (1.64x10^{-6})\varepsilon_p^{1.8}[1-(RH/100)]^{2.2}$
The model can therefore be applied to indoor and outdoor exposure conditions. ε_p is the hardened cement paste porosity obtained through an expression 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. [20] 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. They reported the expressions shown in equation 8 which depicted that carbonation is greater at corners than at the flat surfaces of concrete.

$$d = 2\left(\frac{DC_0}{m_0}t\right)^{\frac{1}{2}} \quad d = \left(\frac{2DC_0}{m_0}t\right)^{\frac{1}{2}} \qquad (8)$$

i. surface ii. Corner surface

The results show the relevance of structure geometry on carbonation rate which many carbonation models do not normally consider especially during testing. It can be seen in Table 1 that most studies concentrate on measurements done at flat surfaces of concrete. Since carbonation is greater at the corners of the structural member (equation 8), it indicates that measurements based on flat concrete surfaces may not provide the worst case scenario for service life prediction. The application of this type of model has been found relevant to concrete used in nuclear reactors, silos and other structures where CO_2 concentration difference and aesthetics are issues of interest.

3.3. Carbonation rate and surface porosity

In an attempt to show the relationship between the transport mechanism and performance of concrete for in-situ assessment of concrete carbonation, Parrot [16] used the oxygen permeability of cover concrete to model the carbonation depth in concrete. The influence of moisture conditions of the cover concrete on CO_2 penetration, and reinforcement corrosion were studied in terms of relative humidity within the concrete. The empirical relationship between the parameters was proposed as shown in equation 9

$$d = \frac{aK^{0.4}t^{n}}{c^{0.5}} \quad (9)$$

Where K is the oxygen permeability at 60% RH, t is time for corrosion initiation in years; c is the alkaline content in the cement, n is the attenuation factor dependant on the environment. The experimental investigation revealed the role played by relative humidity in controlling CO_2 penetration into the concrete. The power n varies depending on the rate controlling processes such as CO_2 diffusion, pore structures, amount of lime and humidity. However, porosity is strongly related to strength and permeability. At the intermediate levels of relative humidity of 50 to 70%, CO_2 carbonation is greater than the case where the pores are saturated at 80%RH and above [21]. The application of Parrot model showed a good agreement between measured depths of carbonation at 60% RH and the calculated carbonation depths determined using the model.

Parrot model may require modification before use outside the German climatic conditions may be considered. The model allows for variations in carbonation rate with changes in exposure conditions through the introduction of constant, a. The main parameters used in the model to determine the depth of carbonation are the concrete permeability, alkaline content of the cement used, and exposure time which are all functions of the concrete quality. The choice of concrete quality is in turn mostly decided basing on the exposure conditions, cover depth and notional service life as specified by standards or designers [16].

3.4. Carbonation rate and binder type

It is generally acknowledged that carbonation may be greater in concretes containing extenders. Some researchers have attempted to explain the higher carbonation to the relatively lower early strengths in material systems containing extenders. But it is also well established that incorporation of extenders such as slag improves the microstructure and permeability of concrete [22, 23]. Khunthongkeaw at al. [23] studied the carbonation depth prediction of fly ash concrete in different environment, where empirical model relating the carbonation depth in a natural environment with accelerated carbonation was proposed as shown in equation 10.

$$D_{n,t} = A * D_a * \sqrt{t} \quad (10)$$

Where D_{nt} represents the carbonation depth in a natural environment for t months, D_a is the carbonation depth (mm) in accelerated environment for one month, t is the exposure time in a natural environment in years, and A is the slope of the relationship which is a function of environmental conditions. The slope A is obtained as given in equation 11, where C is the CO_2 concentration, and RH is the relative humidity in the environment.

$$A = 0.22\frac{C^{0.56}}{RH} \quad (11)$$

Carbonation is greater in concrete containing both fly ash and slag with more carbonation occurring in slag concrete than in Portland cement (PC) concrete [24]. This effect of extenders was attributed to low portlandite content of the mixture that reduces the binding activity of carbon dioxide. However, this effect decreases as the strength of concrete increases with age following the square root law as shown in equation 12 [22, 23]. Carbonation rate is illustrated as a function of concrete compressive strength as shown in equation 13.

$$d = v * \sqrt{t} \quad (12)$$

$$v = a * f^{b} \quad (13)$$

Where d is the depth of carbonation in mm, v is the carbonation rate in mm/√year, t is time in years, f is compressive strength in N/mm^2, while a, b represent constants that depend on the binder type. In recent years, it has been established that the use of extenders improves the concrete microstructure seemingly suggesting increase in the time required for carbonation of the concrete cover. But this is not true as can be equally argued that extenders utilize the cement hydrates that maintain high alkalinity in concrete. Others believe that cement containing fly ash takes some time to harden which makes it easily susceptible to carbon dioxide attack before maturity. However, this effect can be reduced through extended curing days and using thick concrete cover [25-27].

Hakkinen's work cited in [17] extensively considered different types of binders and determined the shift factors shown in Table 1, where a, b can be seen to depend on the binder type. 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 [22]. Such an investigation becomes necessary to derive various constants applicable to binder material systems used in the local industry.

Table 1 Coefficients a and b for determination of the carbonation rate, Hakkinen cited in [17]

Binder	a	b
PC	1800	-1.7
PC+ FA 28%	360	-1.2
PC+ SF 9%	400	-1.2

PC+GGBS 70%	360	-1.2

3.5. Carbonation depth and curing

LeSage de Fontenay [28] modeled the carbonation relationship between the 28-day water cured concrete and the air-cured concrete, and reported the expressions shown in equations 14 and 15. It may be noted that for PC concrete, the effect of curing beyond 14 days on carbonation rate is negligible [29].

28-day cured:

$$d = 0.43(w/c - 0.4)(12(t-1))^{0.5} + 0.1 \quad (14)$$

Non-cured:

$$d = 0.53(w/c - 0.2)(12t)^{0.5} + 0.2 \quad (15)$$

The influence of curing presents a challenge on time and material optimization in concrete construction, which in turn are important variables in modeling the initiation time of corrosion. Furthermore, 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 concretes such as pre-stressed concrete elements as well as normal concretes subjected to different forms of curing.

3.6. Carbonation rate and exposure conditions

As already mentioned, exposure condition is a major factor in determining carbonation in reinforced concrete. It is also one 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 from rain (or (ii) unsheltered from rain (structure subject to alternate wetting and dry ing). These conditions are defined in [30], while others have proposed definitions such as shown in Table 2. (The effects of the exposure condition to concrete carbonation in SSA have not been (extensively that investigated compared to studies done in other parts of the world such as Europe, Asia and America. Kokubu and Nagataki model given in Table 3 [31] 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.

3.7. Carbonation rate and relative humidity and temperature

Matsuzawa et al. [10] investigated the effect of relative humidity and temperature on carbonation rate under accelerated test conditions where CO_2 was fixed at 5%. The relative humidity was varied at 30, 45, 60 and 80% RH while temperatures 20ºC or 60ºC were used. The 60ºC temperature was considered so as to depict conditions in a nuclear power plant structure.

The carbonation coefficient was modeled as shown in equations 16 and 17 with or without the temperature parameter. The experimental results were found to follow the general trend that carbonation rate is higher with increase in temperature of curing.

$$K = H(100 - H)(140 - H)/192000 \quad (16)$$

$$K = -0.001478H^2 + 0.144H + 0.0351T - 1.9853 \quad (17)$$

3.8. 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 literature [25, 28, 31,33], 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 [33]. A case in point is the 20 year long - term study by Kokubu and Nagataki [31] (see Table 3), whose findings are similar to the results of the accelerated carbonation study of Loo et al. [25] where carbonation depth is modeled as $\alpha[(w/c) - \beta]$, where α and β are constants for specific exposure conditions. 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. [34] as one of the compositional parameters that influence the concrete carbonation depth in an extensive model shown in equation 18.

Table 2 Classification of exposure condition in Afri-

EXPOSURE	DESCRIPTION
Mild	Structures protected from harsh conditions except for a brief period of exposure to normal weather conditions during construction
Moderate	Concrete structures buried in soil, structures submerged in water, structures sheltered from rains, salt spray and heavy winds, structures exposed to dry winds
Severe	Structures exposed to spray or abrasive action of sea water, alternate wetting and drying, structures exposed to corrosive fumes in industrial areas

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Table 3: Model equations [31]

EXPOSURE CONDITION	CURING PERIODS IN WATER (DAYS)	MODEL EQUATION Depth of carbonation (mm)	
	1	$[0.12(w/c)-4.9]\sqrt{t}$	$12.2-1.72\sqrt{f_{28}}\sqrt{t}$
Unsheltered	7, 28, 91	$[0.10(w/c)-4.4]\sqrt{t}$	$9.90-1.44\sqrt{f_{28}}\sqrt{t}$
	7	$[0.19(w/c)-6.2]\sqrt{t}$	$19.3-2.54\sqrt{f_{28}}\sqrt{t}$
Sheltered	91	$[0.14(w/c)-4.5]\sqrt{t}$	$14.6-1.91\sqrt{f_{28}}\sqrt{t}$

$$d = 350\left(\frac{\rho_c}{\rho_w}\right)\frac{w/c-0.3}{1+(\rho_c/\rho_w)w/c}\left(1-\frac{RH)}{100}\right) \left\{\left[1+\left(\frac{\rho_c}{\rho_w}\right)\frac{w}{c}+\frac{\rho_c}{\rho_w}\left(\frac{a}{c}\right)\right]yCO_2t\right\}^{0.5} \quad (18)$$

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.

The parametric studies by Papadakis et al. model [32] suggests water/cement ratio as the main determinant that dictates the performance of the concrete under carbonation. The limitations of this model for use in practice are associated with the complexity in evaluating the CO_2 diffusion as well as the difficulty in analysis of w/c, along with involvement of too many parameters. Making use of concrete 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 [25].

3.9. 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. [25] proposed the following three different models based on experimental work,
$K_a = A(B-f)$, $K_a = e^{a(1-bf)}$, $K_a = \alpha f^{0.5} - \beta$
Where A, B, å, b, α and β are coefficients. The strengths at 28 days and strengths at the age of testing were compared. The strong correlation found shows that either of the two can be used in an corrosion initiation model. However, better correlation is possible when the square root of compressive strength is used [31]. Hakkinen [22] in an experimental work shows the existence of an inverse relationship between the carbonation rate and compressive strength as $v = a * f^{-b}$, 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 [9] proposed a model where the carbonation rate is a square function of strength as shown in equation 19. The function contrasts with Kokubu and Nagatakis model [31] shown in equation 20, 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. For concrete of known w/c, compressive strength can be obtained through the relation $f_{28} = 74 - 64w/c$.

$$K = 0.0063(54.5 - f_{28})^2 + 1.6 \tag{19}$$

$$K = A - B\sqrt{f_{28}} \tag{20}$$

The constants A and B in the Kokubu and Nagatakis model accounts for the time response of the cementitious system, and are expected to adjust along with replacement of PC content with extenders. As already seen, carbonation generally shows a good linear correlation with the square root of compressive strength and can be used to predict long term carbonation depth but most of the strength-based models are generalized and inadequate under certain limiting conditions such as varied exposure environments.

3.10. Carbonation prediction models at multivariate level of parameters

Loo et al [25] 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 as shown equation 21.

$$K_a = \alpha f_{28}^{a}\, C_0^{b} e^{cT} t_{wc}^{d} + \beta \tag{21}$$

Where K_a is the carbonation coefficient in mm/year$^{0.5}$, f_{28} is the 28-day compressive strength in N/mm^2, C_0 is carbon dioxide concentration, t_{wc} is curing duration in days, e^{cT} accounts for the effect of temperature, α and a are constants depending on the type of binder, and b, c, d are shift factors. The study gives insight into correlation between carbonation coefficient and the key parameters influencing the rate of carbonation such as w/c, cement content, 28-day compressive strength, porosity, curing regime, and temperature. There was strong correlation between test results and predictions based on 28-day compressive strengths. However, the effect of cement content was negligible in comparison but the inclusion of tests such as sorptivity and water absorption slightly improved the correlation of results. Also, the use of various binders was not considered which shows the model limitation in applicability.

The inverse relationship between the depth of carbonation and strength has also been modeled numerically as shown in equation 22. It considers the parameters representing the CO_2 binding capacity, surface concentration of CO_2, and environmental condition as proposed by C.Bob [36] cited in Badea et al. [37].

$$d = \frac{150ckd}{f_c}\sqrt{t} \tag{22}$$

Where c is the parameter representing the CO_2 binding capacity, d is the coefficient of surface concentration of carbon dioxide, f_c is compressive strength of the concrete in N/mm^2, while k represents the relative humidity which forms the determinant parameter when correlated with others.

The equation 23 proposed by Nagatakis [38] examined various parameters that influence the carbonation rate of fly ash concrete such as type of cement, environment and curing conditions, water/cement ratio and micro-structure of concrete. Long-term tests of 20 years were conducted to investigate the effects of various multivariate factors on the carbonation depth.

$$d = AB[\alpha(w/c) - \beta]t^{\gamma} \tag{23}$$

Where d is the carbonation depth, A is the correcting factor, B is the factor for initial curing period in water and α, β are the factors for fly ash, w/c is the water /cement ratio, t is age in years and γ is the factor for environmental conditions. The model allows for different types of locally available fly ash while the effect of the environment was accounted for through the time function parameter, γ. The model is somehow similar to the one proposed in Japan by De Sitter (1984) cited in [9, 39], and given in equation 24. Its application is limited to concrete of water/cement ratio of less than 0.6.

$$k = \frac{46w/c - 17.6}{2.7} R * K \tag{24}$$

R is the influence of cement and K is the influence of exposure condition. These coefficients change depending on extenders and severity of exposure conditions. It is also worth noting that early days of curing have a greater effect on carbonation depth than curing done at a later age.

The Hakkinen model given in equations 25 and 26 [17], evaluates the carbonation rate considering multivariate factors of material properties consisting of the characteristic compressive strength of concrete, porosity in terms of air content, and exposure conditions.

$$K_c = c_{env} c_{air} a(f_{cm} + 8)^b \quad (25)$$

$$t_i = \frac{d^2}{(c_{env} c_{air} a(f_{cm} + 8)^b)^2} \quad (26)$$

Where c_{env} is the environmental coefficient (humidity and temperature), c_{air} is the air content coefficient, f_{cm} is the characteristic compressive strength of concrete in N/mm^2. The coefficents a and b depend on the cement combination type. The model classifies the exposure into two categories of the sheltered and unsheltered exposure from rain (see Table 1). Siemes et al. [39] gave details of exposure conditions along with their coefficients relevant to the European environment. Consideration of air content may not be relevant to the SSA climates due to absence of severe winter conditions in the sub-continent. Liang et al. [20] combines the controlling factors and uses a statistical method to modeled carbonation depth of concrete as given in equation 27.

$$d = \zeta_1 \zeta_2 \zeta_3 \zeta_4 (\mu_A + \beta\sigma_A)\sqrt{t} \quad (27)$$

Where ζ_1 is the concrete material coefficient, ζ_2 is the cover thickness coefficient, ζ_3 is the coefficient influenced by the environment, ζ_4 is the structural damage coefficient. With this approach sensitivity analysis of each parameter can be evaluated using numerical methods.

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 mechanism of corrosion attack is illustrated in Figure 2. Corrosion causes premature deterioration of reinforced concrete structures [3] as determined by the rate at which reinforcement corrosion proceeds [27]. The damage induced by the expansion of corrosion products increases the stress around the reinforcement which causes cracking and spalling of the concrete cover, and forms the second phase of Tuutti's model. However, Tuutti's model is a generalized concept and does not identify the sub-stages of corrosion damage specifically cracking, spalling, and delamination [5].

Relatively, the propagation time is a short period compared to the initiation time and is one reason that some models do not incorporate the propagation phase into their service life estimation. The time taken to attain corrosion-induced damage (that is, the corrosion propagation time) [8, 16, 40] is defined by the corrosion rate. This phase is largely controlled by availability of oxygen, temperature, anode and cathode area, resistivity of concrete [9, 27].

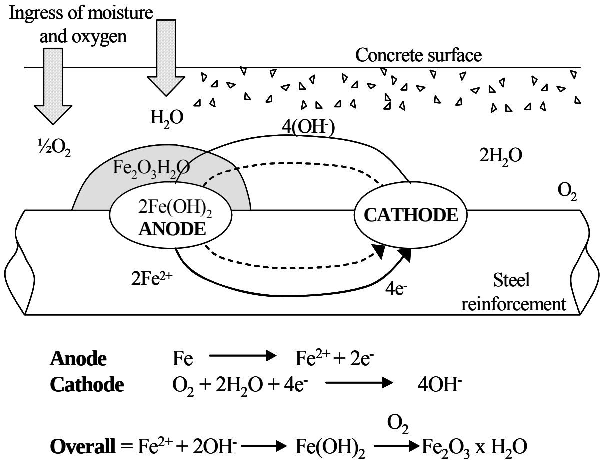

Figure 2: The steel corrosion process

An experimental model was proposed [9] where the rate of corrosion is influenced by the flow of hydroxyl ions from the cathode to anode. The resistance to the flow of ions is determined by the characteristics of the concrete material and the anodic surface area, as illustrated in Figure 2 and equation 28

$$r = k_r \frac{I}{A} \quad (28)$$

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 [8] ascertained that corrosion rate is dependent mostly on environmental factors such as temperature and relative humidity rather than material properties. Corrosion rate is greater when pores are partially saturated and becomes zero at less than 40% RH [13]. The effect of temperature cannot be overemphasized in corrosion reactions. The corrosion rate is bound to increase at elevated temperature as molecules become more chemically agitated. Tuutti [8] investigated the effect of temperature from 0 to 20°C, and found that corrosion increased ten times due to the temperature rise. 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. [41] pointed out that increase in temperature may lead to higher evaporation of the pore water with the immediate effect on concrete resistivity and reducing the corrosion rate. It is also evident that many authors rarely incorporate temperature effect into their models. It would appear that most corrosion models consider the temperature parameter to be less important.

Rashid et al. [42] 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

[41]. More importantly, concretes made using pozzolans such as silica fume, metakaolin, and slag usually show low corrosion rates. Their effectiveness may partly be attributed to the resulting refined denser microstructure and low concentration of hydroxide ions in the pore solution of the concretes containing extenders. Through this influence, extenders can lengthen the propagation time and hence the service life of the concrete structure.

The equation 29 proposed by Alonso and Andrade [43], gives the propagation period of the corrosion process as a function of change in bar radius in direct relation to corrosion rate.

$$t_p = \frac{\Delta R_{max}}{r} \tag{29}$$

Where t_p is propagation time, ΔR_{max} represents the maximum loss of radius of the steel bar, and r is the rate of corrosion. From the equation 27, it can be seen that propagation time depends on rate of corrosion which in turn is determined by the environmental factors and concrete resistivity [8]. Electrochemically, the rate of corrosion is controlled by corrosion current and area of anodes (see equation 26). But corrosion rate is also influenced by physical factors such as steel diameter, concrete cover, geometry of concrete surface. Also, the exposure time and cover thickness no longer play a significant role once corrosion has commenced [20, 27]. A model describing corrosion propagation in terms of time to surface cracking is given in equation 28.

$$t_p = \frac{0.08C}{Dr} \tag{30}$$

Where t_p is the propagation time of corrosion in years, r is the rate of corrosion in mm/year, D is the bar diameter in mm, and C is cover thickness in mm. It can be seen that C/D is directly proportional to propagation time which delay the time to surface cracking [3]. In related work by Zhou et al. [14], cracking initiation was not affected by changing the ratio C/D but a slight increase in uniaxial compressive strength of concrete significantly improved crack initiation.

Despite the improvement in understanding of the propagation phase of corrosion, 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. The difficulty in quantifying the propagation stage is attributed to the uncertainties of the influences of the several associated factors from the onset of steel depassivation, especially the corrosion rate, geometry of corrosion build up, and types of corrosion products [14]. In the literature, several methods of analyses including the finite element, boundary element, differential method, and monte carlo simulation have been used to examine the impact of various parameters on service life. In [14], finite element modeling was applied to corrosion-induced cracking, spalling and delamination of reinforced concrete bridge decks. In their analysis, the limiting criteria for the three 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 [14].

5. PARAMETER VARIABILITY IN MODEL DEVELOPMENT

It is well established that the variables influencing steel corrosion in concrete have uncertainties associated with them, which realistically makes service life a stochastic quantity. The variation in material properties could arise from the quality of concrete due to variations in the mix proportion, different level of compaction, the hydration characteristics of cement and changes in temperature, use of extenders, concrete cover depth. Variations do also occur in the service environment (night and day) due to climatic changes [44]. All these uncertainties compound into variation in service life.

In the past, deterministic approaches have been applied to evaluate the service life of reinforced concrete structures. But this approach is severely inadequate as it is unable to evaluate the risk of failure associated with the random nature and uncertainties of the variables [7]. The probability - based method that takes into consideration the uncertainties inherent in the random variables for service life in order to predict their effects, is more appropriate. It is this stochastic approach that the authors of this article intend to use in an investigation into a robust, practically effective service life model for conditions relevant to SSA.

6. CONCLUSION

This paper has been able to identify various parameters considered by different authors in modeling of the service life of concrete structures for carbonation-induced reinforcement corrosion. 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.

Most of the research studies are laboratory-based experiments where carbonation and corrosion are accelerated in the interest of time. In studies that have included parallel field investigations along with accelerated tests, results typically show strong correlation between the latter and the long-term experimental results. Validation of laboratory results with extensive site specific data on corrosion initiation and propagation is important for wider acceptability and development of empirical models. This aspect presents challenges and at the same time offers opportunities for future research in service life modeling. Existing aged structures can also be used to validate laboratory data or model predictions.

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 Africa, and incorporation of the effects of new construction materials and technologies are necessary requirements for an effective service life model. Execution of such a model should take into account the randomness of the parameters due to complex interactions of variables. 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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