Comparisons on Reinforcing Bar Corrosion Models of RC Structures in Marine Environment

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Abstract
In marine environment, the physical and mechanical performances of reinforced concrete members will vary distinctly with time, which reduces the safety of the structures. The main factors influencing the durability of reinforced concrete members include the deterioration of concrete, the reduction of reinforcing bar strength, the decrease of reinforcing bar area, the weakening of the bond strength between reinforcing bar and concrete, as well as the concrete material performance, the ratio of water to cement, the thickness of concrete cover and the chloride concentration, etc. The reinforcing bar corrosion model is the key problem in the durability calculation of reinforced concrete structures. Various reinforcing bar corrosion models are presented for the durability calculation of reinforced concrete structures and the values of the parameters in the models determined by different literatures are quite scattered. The calculating results from different models are evidently discrepant. In this paper, the various models for calculating the durability of reinforced concrete structures are compared and the effects of the model parameters on calculating results are investigated through an example. Some conclusions are remarked. It is useful for the further development of durability design theory and durability calculation method of reinforced concrete structures in marine environment.

1 Introduction
The reinforced concrete structures are widely used in port and coastal engineering of China. In marine environment, reinforcing bars corrodes easily due to the long-term ingress of chloride ions. Harbor wharfs, especially high piled wharfs, damage seriously even in design life cycle. In the 60s last century, the Ministry of Communication P.R.C investigated 27 ports in south and east China. The result shows that 74% of the reinforced concrete structures are damaged because of reinforcing bar corrosion. In 1980, the Fourth Harbor Engineering Bureau of the Ministry of Communication investigated 18 wharfs, 80% of which are seriously damaged in south China and some of the damage appears in 5~10 years[1]. In
1996, the Fourth Harbor Engineering Bureau of the Ministry of Communication investigated 20 berths in port C and port E completed after 1986 in south China. The result shows that most longitudinal and transversal reinforcements in port E are corroded within 5~10 years and a lot of corrosion cracks are generated 5~6 years after completion. 25 berths of high piled wharf in Tianjin port were constructed during 1958 to 1985 whose shoreline is 6000 meters long. Some wharfs damaged because of corrosion within 5~6 years. The durability of reinforced concrete structures in marine environment becomes an important research topic recently.

Reinforcement corrosion is the prime reason to the durability damage of reinforced concrete structures. There are two kind of mechanism of reinforcement corrosion namely the concrete carbonization and the chloride ingress. As for reinforced concrete structure in marine environment, the chloride ingress is more predominant. Many factors influence the chloride ingress such as the chloride concentration in marine environment, the cement type, the ratio of water to cement and the thickness of concrete cover, etc. Three types of chloride ingress models, namely the diffusion model with constant diffusion coefficient, the diffusion model with variable diffusion coefficient and flow model for cracked concrete, have been presented. Also three types of reinforcement corrosion models, which are the corrosion model with constant corrosion velocity, the corrosion model with decreasing velocity and the corrosion model with increasing velocity (for cracked concrete), were proposed. Not only the results from various models are different, but also the values of the parameters in the models determined by different literatures are quite scattered, some of which can only be described qualitatively. It is necessary to conduct further investigations on the durability calculation models of reinforced concrete structures and the determination of the model parameters.

In this paper, various models for the durability calculation of reinforced concrete structures are compared and the effects of the model parameters on calculating results are investigated through an example. Some conclusions are remarked. It is useful for the further development of durability design theory and the durability calculation method of reinforced concrete structures in marine environment.

2 Chloride Diffusion Model

2-1 Chloride Diffusion Model with Constant Diffusion Coefficient

Numerous detection results show that the chloride transport can be considered as a linear diffusion process. Based on the Fick's Second Law, the model as follows can be used to predict chloride concentration in concrete:

\[
\frac{\partial C}{\partial t} = D(t) \frac{\partial^2 C}{\partial x^2}
\]  

(1)

With the initial condition: \(C(x,0)=C_0\) and the boundary condition: \(C(0,t)=C_s\); \(C(\infty, t)=C_0\), the solution to Eq.(1) is
\[ C_x = C_0 + (C_s - C_0)[1 - \text{erf}(\frac{x}{2\sqrt{Dt}})] \] (2)

Where, \( C \) is the chloride content (percentage of concrete weight); \( x \) is the distance from calculated point to concrete surface; \( C_x \) is the chloride content at \( x \), \( C_s \) is the chloride content on concrete surface; \( C_0 \) is the initial chloride content in concrete, generally equals to 0; \( D \) is the effective diffusion coefficient of chloride, taking as a constant; \( \text{erf}(\cdot) \) is Guass error function

\[ \text{erf}(x) = \frac{2}{\sqrt{\pi}} \int_0^x e^{-z^2} dz. \]

**2-2 Chloride Diffusion Model with Variable Diffusion Coefficient**

According to literature[7], the chloride diffusion coefficient is not a constant, but varies with time and follows the exponential rule:

\[ D(t) = D_0 \left( \frac{t_0}{t} \right)^{\alpha} \] (3)

Where \( D_0 \) is the chloride diffusion coefficient obtained from structure exposed to chloride environment for a reference time \( t_0 \); \( \alpha \) is related to the ratio of water to cement, the type and amount of mineral additives.

From Eq.(1) and (3), it yields:

\[ \frac{\partial C(x,t)}{\partial t} = D_0 \left( \frac{t_0}{t} \right)^{\alpha} \frac{\partial^2 C(x,t)}{\partial x^2} \] (4)

The solution to eq.(4) is:

\[ C(x,t) = C_0 + (C_s - C_0) \left[ 1 - \text{erf}(\frac{x}{2\sqrt{T}}) \right] \] (5)

in which

\[ T = \int_0^t D_0 \left( \frac{t_0}{t} \right)^{\alpha} dt \] (6)

It is proposed that the change of inner microstructure does no longer take place when the hydration in concrete is almost finished and the diffusion coefficient tends to be a stable value. Literature [7] assumes that the chloride diffusion coefficient varies in the first 10 years and is constant after 10 years.
When the diffusion coefficient varies with time in the first 10 years,

\[
C(x, t) = C_0 + (C_s - C_0) \left[ 1 - \text{erf} \left( \frac{x}{2 \sqrt{D_0 t_0^{\alpha} t^{1-\alpha}}} \right) \right]
\]  

(7)

When the diffusion coefficient reach a constant after 10 years

\[
C(x, t) = C_0 + (C_s - C_0) \left[ 1 - \text{erf} \left( \frac{x}{2 \sqrt{D_0 \left( \frac{t_0}{10} \right)^{\alpha} \left( \frac{10}{1-\alpha} + (t-10) \right)}} \right) \right]
\]

(8)

2-3 Flow Model

The cracked reinforced concrete is referred to the structure with crack width larger than 0.1mm (the initial crack is induced by load). It is presented by Li\textsuperscript{[9]} that the concrete is heterogeneous porous material, and the chloride ingress in the concrete is from a combination of various mechanisms. As for RC structures under the service loads which induce cracks in concrete, the diffusion becomes less dominant. The chloride penetration in the cracked concrete structures is much more complicated than Fick’s Second Law. Therefore, it is needed to develop new methods to predict the initiation time of reinforcing bar corrosion.

Li proposed that the chloride ingress in cracked concrete is very complicated because of its uncertainty and variation with time. So the ingression is considered as a stochastic process. As for concrete with macro crack (crack width greater than 0.1 mm):

\[
\mu_c(t) = C_0 \exp(at) \]  

(9)

\[
V_c(t) = b \cdot t + 0.1433 \]  

(10)

where, \( \mu_c(t) \) and \( V_c(t) \) are the mean and the variation coefficient of chloride content on reinforcing bar surface, respectively. \( V_c(t) \) is from 0.15 to 0.3. \( C_0 \) is the average of initial chloride concentration on reinforcement surface, which is related to concrete composition (cement type, the water-cement ratio and so on), \( a \) is a coefficient denoting the chloride ingress velocity, which is difficult to determine because of influence of natural properties of concrete and external environment. Generally, it can be obtained by regression analysis of experimental data or observed data. \( b \) is related to variation coefficient, \( t \) is actual time in day.
3 Reinforcing Bar Corrosion Model

Considering the difference of reinforcing bar corrosion velocity before and after cracking of concrete cover, a modified Tuutti model, which is based on the typical corrosion model proposed by Tuutti(1982), is presented, as shown in Fig.1. In the model, the corrosion process of reinforcing bar is divided into three periods, $T_i$, $T_s$, and $T_f$. $T_i$ is the corrosion induction period, in which the chloride concentration on reinforcing bar surface is lower than the threshold and the corrosion of reinforcing bar doesn’t begin. $T_s$ is propagation period of corrosion, from the beginning of reinforcing bar corrosion to the cracking of concrete cover. In this period, the volume of rust products expands and expansive force occurs in concrete cover. $T_f$ is the corrosion development period, in which the reinforcing bar corrodes rapidly and the corrosion crack width becomes larger even concrete scaling occurs. This period is from the beginning of concrete cracking to the serviceability limit state when the crack width arrival to 1 mm. If the reinforcing bar corrosion was not considered in design, the bearing failure of the structure will probably occurs in the second period when the reinforcement begins to corrode.

![Figure 1 Reinforcement corrosion model](image)

3-1 Initiation Time of Reinforcement Corrosion $t_i$

Reinforcing bar begins to corrode when the carbonation depth in concrete or the chloride concentration on reinforcement surface reach the threshold values. In marine environment, the chloride ingress is the dominating cause inducing corrosion. For this reason, the initiation time is considered as the time when the chloride concentration on reinforcement surface rises to a threshold value.

The initiation time of reinforcement corrosion $t_i$ is determined by Eq.(2), or Eq.(7), or Eq.(8) when $x$ and $C(x, t)$ in the equations are substituted by $c$ and $C_{cr}$ ($c$ is the thickness of concrete cover and $C_{cr}$ is the threshold of chloride concentration).

3-2 Concrete Cracking Time $t_c$

When the reinforcement corrosion begins, the reinforcement corrosion will propagate in concrete and produces expansive rust which occupies a much larger volume than the original reinforcement and thereby generates expansion force which increases with rust products increasing. Eventually the force exceeds the tensile capacity of concrete cover and then crack exists. So the calculation of the cracking time will associate with the corrosion velocity and the corrosion amount.
3-2-1 Corrosion Velocity Model of Reinforcement

Three models are proposed for the corrosion current density. One believes that the corrosion current density is constant, second considers that the corrosion current density decreases with time, and the third assumes that the corrosion current density increases with time.

(1) Model with constant corrosion velocity (Niu Ditao model)

According to numerical experiment and site observation results, Niu Ditao believes that the corrosion velocity of reinforcement are two different constants before and after concrete cracking. the corrosion velocity after cracking is evidently larger than that before cracking. He presented the empirical formula in atmospheric environment about corrosion velocity before and after cracking:

\[
\lambda_{c1} = 46k_c e^{0.04T} (RH - 0.45)^2 c^{-1.36} f_{cu}^{-1.83}
\]

(11)

\[
\lambda_{c2} = \begin{cases} 
2.5\lambda_{c1} & (\lambda_{c1} > 0.008) \\
4.0\lambda_{c1} - 187.5\lambda_{c1}^2 & (\lambda_{c1} \leq 0.008)
\end{cases}
\]

(12)

in which, \(\lambda_{c1}\) and \(\lambda_{c2}\) are the corrosion velocity before and after cracking, respectively, in mm/a; \(k_c\) is the modifying coefficient of reinforcement position, 1.6 for the corner reinforcement and 1.0 for the middle reinforcement; \(k_{ce}\) is modifying coefficient of environmental condition, 3.0~4.0 for outdoors in humid area, 1.0~1.5 for indoors in humid area, 2.5~3.5 for outdoors in dry area, 1.0 for indoors in dry area; RH is ambient humidity (%); c is the thickness of concrete cover in mm; \(f_{cu}\) is the cubic compressive strength of concrete; T is ambient temperature.

In Niu Ditao’s model, the corrosion velocities are assumed to be two constants before or after concrete cracking, but the two values are discrete and have a skip.

(2) Model of corrosion velocity decreasing with time (Kim model)

Kim believes that the corrosion velocity is related to the obtaining of oxygen and water on reinforcement surface and it should be a function of the concrete quality and the thickness of concrete cover\(^{[11]}\). It is expected that the formation of rust products on the reinforcement surface will prevent the diffusion of the iron ions away from the reinforcement surface. Therefore the corrosion velocity will reduce with time. According to the experimental result by Liu\(^{[13]}\), a corrosion model can be derived as follows:

\[
i_{corr}(t) = 0.85 i_{corr}(1)t^{-0.29}
\]

(13)

\[
i_{corr}(1) = \frac{37.8(1-w/c)^{-1.64}}{c}
\]

(14)

where, \(i_{corr}(1)\) is the initial velocity of corrosion in \(\mu A/cm^2\); w/c is the ratio of water to cement; c is the thickness of concrete cover; t is the corrosion time (a).
In practical application, the equation is converted to:

\[ \lambda \approx 0.0116i_{\text{cor}}(t) \ (\text{mm} / \text{a}) \]  \hspace{1cm} (15)

The variation coefficient of \( i_{\text{corr}} \) is between 0.14 and 0.33.

(3) Model of corrosion velocity increasing with time (for the concrete with cracks)[12]

Based on a large number of experiments on reinforced concrete bending members with cracks induced by service loads in simulated marine environment, Li[12] suggested that the corrosion velocity increases with time and gives the empirical formula after corresponding curve fitting:

\[ i_{\text{cor}}(t) = 0.3683 \ln(t) + 1.1305 \]  \hspace{1cm} (16)

(4) Corrosion model proposed herein

Kim model of corrosion velocity decreasing with time is based on the statistical analysis of many experimental data in chloride ingress environment. However, the data are obtained by Weyers from experiment on un-cracked concrete members. So Kim model can be only suitable for un-cracked concrete members in marine environment. Niu Ditao model with constant corrosion velocity is obtained in atmospheric environment and on the assumption that there is only the erosion of carbon oxygen. So Niu Ditao model is suitable for cracked and un-cracked concrete structures in atmospheric environment. Chun Qing Li model is based on a large number of experiments on the reinforced concrete members with cracks induced by service loads in simulated marine environment. So it isn’t suitable for un-cracked concrete structures.

According to the analysis above, it is assumed that in marine environment, Kim model of corrosion velocity decreasing with time is rational before concrete cracking, and Chun Qing Li model of corrosion velocity increasing with time is more suitable after cracking.

3-2-2 Quantity of Corrosion Products

Assume that the diffusion of iron ions is inverse proportional to thickness of oxidation layer, so generation rate of corrosion products decreases gradually with the increasing of corrosion layer thickness:

\[ \frac{dW_{\text{rust}}}{dt} = \frac{k_p}{W_{\text{rust}}} \]  \hspace{1cm} (17)

Where, \( W_{\text{rust}} \) is the quantity of corrosion products in mg/mm. \( k_p = 0.105(1/\alpha)\pi D_i_{\text{cor}} \) is a coefficient related to the rate of metal loss. \( \alpha \) is related to the type of corrosion products, which equals to 0.523 when corrosion products are regarded as \( F_e(OH)_3 \) and 0.622 as \( F_e(OH)_2 \). As the corrosion products are in between \( F_e(OH)_3 \) and \( F_e(OH)_2 \), \( \alpha \) is generally taken as 0.57.

Integrating Eq.(17), the quantity of corrosion products is given by:
The threshold quantity of corrosion products inducing concrete cover cracking can be calculated by:

\[
W_{\text{crit}} = \rho_{\text{rust}} \left\{ \pi \left[ \frac{Cf_t}{E_{\text{ef}}} \left( \frac{b^2 + a^2}{b^2 - a^2} + \nu_c \right) + d_0 \right] D + \frac{W_u}{\rho_{\text{st}}} \right\}
\]

where, \( \rho_{\text{rust}} \) is the density of rust, equals to 3600 kg/m\(^3\); \( \rho_{\text{st}} \) is the density of reinforcement, equals to 7850 kg/m\(^3\); \( C \) is the thickness of concrete cover; \( f_t \) is the tensile strength of concrete; \( E_{\text{ef}} \) is effective elastic modulus of concrete and is given by \( E_{\text{ef}} = E_c / (1 + \varphi_{\text{cr}}) \), in which \( E_c \) is the elastic modulus of concrete and \( \varphi_{\text{cr}} \) is the creep coefficient of concrete, 2.2 for high-early cement and 2.0 for ordinary cement; \( \nu_c \) is Poisson ratio of concrete, generally equals to 0.17; \( a = (D + 2d_0)/2 \); \( b = C + (D + 2d_0)/2 \); \( d_0 \) is the interval thickness between reinforcement and upper interface of concrete, usually equals to 12.5 \( \mu \text{m} \); \( W_u \) is the mass or corroded reinforcing bar, \( W_u = \alpha W_{\text{crit}} \).

Moreover, the Durability Criterion for Concrete Structures (being revised) gives the expression of corrosion thickness of reinforcing bar induced by expansion fracture\(^{[14]}\).

For smooth reinforcing bar:

\[
\delta_{\text{cr}} = k_1 (0.012c/d + 0.00084f_{\text{cu}} + 0.22)
\]

For deformed reinforcing bar:

\[
\delta_{\text{cr}} = k_1 \cdot (0.008c/d + 0.00055f_{\text{cu}} + 0.022)
\]

For hoop reinforcement:

\[
\delta_{\text{cr}} = k_1 \cdot (0.026c/d + 0.0025f_{\text{cu}} + 0.068)
\]

where, \( k_1 \) is modified coefficient of reinforcement position, 1.00 for corner reinforcement and 1.33 for side middle reinforcement.

3-2-4 Calculation of Concrete Cover Cracking Time \( t_c \)

The cracking time of concrete cover can be determined by Eq.(18) when corrosion amount in Eq.(18) equals to the threshold value computed from Eq.(19) or Eq.(20), (21), and (22).

3-3 Time of Concrete Crack Width to 1 mm \( t_l \)

The effect of longitudinal cracks along reinforcement on corrosion is more
predominant than of transverse cracks. Nevertheless, the longitudinal crack width limits haven’t been specified at present in Design Code for Concrete Structures. According to the Criterion for Dangerous Building, the service life will end when the longitudinal crack width arrives to 1 mm.

According to the collected numerous experimental results and site investigations. Durability Criterion for Concrete Structures (being revised) gives the relation between the corrosion thickness of reinforcement and crack width

For corner smooth reinforcing bar:

\[
\delta_w = 0.07w + 0.012c / d + 0.00084f_{cu} + 0.08 \quad (w \geq 0.33\text{mm})
\]

\[
\delta_w = 0.35w + 0.012c / d + 0.00084f_{cu} - 0.013 \quad (w < 0.33\text{mm})
\]

(23)

For non-corner smooth reinforcing bar:

\[
\delta_w = 0.69w + 0.026c / d + 0.0025f_{cu} + 0.074 \quad (w \geq 0.34\text{mm})
\]

\[
\delta_w = 1.0w + 0.026c / d + 0.0025f_{cu} - 0.032 \quad (w < 0.34\text{mm})
\]

(24)

For corner deformed reinforcing bar:

\[
\delta_w = 0.086w + 0.008c / d + 0.00055f_{cu} + 0.015 \quad (w \geq 0.10\text{mm})
\]

\[
\delta_w = 0.35w + 0.008c / d + 0.00055f_{cu} - 0.013 \quad (w < 0.10\text{mm})
\]

(25)

where, \(\delta_w\) is the corrosion thickness of reinforcing bar; \(c\) is the thickness of concrete cover; \(d\) is the initial diameter of reinforcement; \(f_{cu}\) is the compress strength of concrete in MPa; \(w\) is crack width in mm.

On the assumption that reinforcing bar corrodes at a uniform velocity, the relationship between the corrosion thickness of reinforcement and time can be given as follows:

\[
\delta = 0.0116\int_0^t i_{corr}(1)dt + \int_0^t \lambda_{c2}dt
\]

(26)

where

\[
i_{corr}(1) = \frac{37.8(1 - w / c)^{1.64}}{c}
\]

When the crack width \(w\) arrives to 1 mm, the corresponding corrosion thickness \(\delta_w\) can be calculated by Eq.(23), (24) or (25). Let \(\delta\) in Eq.(26) equals to \(\delta_w\), the time concrete crack width to 1 mm \(t_l\) can be determined.

4 Effect of Reinforcement Corrosion on Structural Properties

The effects of reinforcement corrosion on structural properties include the decrease of reinforcing bar area, the reduction of reinforcing bar yield strength and the weakening of the bond strength between reinforcement and concrete, as shown in Fig.2.
4-1 Sectional Dimension of Reinforcement Varying with Time

The uniform corrosion of reinforcing bar is assumed and the diameter of reinforcing bar after \( t \) years is

\[
d(t) = d_0 - 0.0232 \int_{i_{yp}}^{t} i_{corr}(t) dt
\]  
(27)

The loss rate of sectional area is

\[
\rho_s = \frac{d_0^2 - d^2(t)}{d_0^2}
\]  
(28)

4-2 Variation of Corroded Reinforcement Strength

Engineering practice and many experiments shows that the mechanical property of reinforcement changes with corrosion proceeds and the strength and ductility of reinforcement decrease. The deterioration of corroded reinforcement strength is connected with corrosion level. The reduction rate of yield strength and tensile strength increases as loss rate of sectional area increases because of the stress concentration induced by micro pitting corrosion at the outside edge of reinforcement section. When the reinforcement is critically eroded, the change will take place in stress-strain curve in which there is no yield point and yield strength approaches to the tensile strength.

The actual yield strength \( f_y \) and the actual ultimate tensile strength \( f_u \) is expressed as follows:

\[
f_y = k_y f_{y,0}
\]  
(29)

\[
f_u = k_u f_{u,0}
\]  
(30)
Where, \(f_{y,0} \cdot f_{u,0}\) are the yield strength and ultimate tensile strength of un-corroded reinforcement, respectively; \(k_y, k_u\) are the coefficients denoting the reduction of yield strength and ultimate tensile strength induced by reinforcement corrosion, respectively.

It needs to be noted that actual yield strength and ultimate strength are respectively defined as the ratios of the actual yield load and the ultimate load to the actual area of corroded reinforcement, while the nominal yield strength and ultimate strength are the ratios of actual yield load and ultimate load to the nominal sectional area. The reduction coefficient of nominal yield strength and ultimate tensile strength are

\[
k'_y = k_y (1 - \eta_s)
\]

\[
k'_u = k_u (1 - \eta_s)
\]

The reduction coefficient of nominal strength of corroded reinforcement is shown in table 1.

<table>
<thead>
<tr>
<th>Origin</th>
<th>Nominal yield strength (k_y)</th>
<th>Nominal ultimate strength (k_u)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HuiYunling(1997)</td>
<td>(k_y = 0.985 - 1.028\eta_s)</td>
<td>(k_u = 0.986 - 1.103\eta_s)</td>
</tr>
<tr>
<td>Niu Ditao(2002)</td>
<td>(k_y = 1 - 1.077\eta_s)</td>
<td>(k_u = 1 - 0.805\eta_s)</td>
</tr>
<tr>
<td>Shen Dejian</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforcement I</td>
<td>(k_y = 1 - 1.54\eta_s)</td>
<td>(k_u = 1 - 1.49\eta_s)</td>
</tr>
<tr>
<td>Reinforcement II</td>
<td>(k_y = 1 - 1.33\eta_s)</td>
<td>(k_u = 1 - 1.51\eta_s)</td>
</tr>
</tbody>
</table>

**4-3 Cooperation Coefficient between Reinforcement and Concrete**

Excellent bond properties are a prerequisite for reinforcement and concrete working together. The cooperation coefficient is an important factor reflecting the influence of the bond properties deterioration on bearing capacity of bending members. Assume the bearing capacity of un-corroded reinforcement concrete members to be \(R_0\), the bearing capacity of corroded members is

\[
R = k_s R_0
\]

where, \(k_s\) is the cooperation coefficient.

There is no identical result of rule about the cooperation coefficient varying with the amount of corrosion products or expansion crack width. In literature [17], the cooperation coefficient is relevant to crack width and the corresponding
computation mode is given. Based on this theory and the relationship between the corrosion depth and crack width, Niu Ditao\cite{10} presents a formula to calculate the coefficient using the corrosion depth

$$k_{si} = \begin{cases} 1 & \delta_{si}(t) \leq \delta_{crj} \\ 1 - 0.85 \cdot [\delta_{crj} - \delta_{cri}] & \delta_{cri} < \delta_{si}(t) \leq 0.3 \\ 0.745 + 0.7 \cdot \delta_{cri}(t) & \delta_{si}(t) > 0.30 \end{cases}$$ (34)

where $\delta_{si}(t)$ is the corrosion depth of $i$th reinforcement at time $t$; $\delta_{cri}$ is the corrosion depth at the time when the $i$th concrete cover cracks.

It should be mentioned that the relationship between the bond strength and the corrosion concluded in literature suggests that when the amount of corrosion products is small, the bond strength will increase as the amount of corrosion increases. Before expansion crack appears, the bond strength of corroded reinforcement is higher than that of un-corroded one, but after that, the bond strength will decrease as the corroded amount increases. However, the rise tendency of the bond strength isn’t implied in the formula presented above. Because the experimental results in literature shows that the rise of bond strength is of less importance to structural bearing capacity, so the cooperation coefficient is regarded to be invariant before cracking.

5 Analysis of Factors During Simulating Corrosion Process

5-1 Factors Influencing Structural Durability

There are many mathematical models to analysis the durability of reinforced concrete structures and various values of the parameters in these models are suggested. Main factors influencing structural durability are illustrated in Fig.3. The concrete strength, the reinforcement strength, the sectional dimension of reinforcement and the bond properties between reinforcement and concrete are all decrease with time and effect the durability of reinforced concrete structure.

As for compressive members, the variation of concrete strength with time has great effect on the structural bearing capacity. While as for tensile and bending members, the variation is not so important and the main factors influencing bearing capacity of these members are reinforcement strength, sectional dimension of reinforcement and the reduction of bond strength between reinforcement and concrete, all of which are induced by reinforcement corrosion.

5-2 Effect of Reinforcement Corrosion Model on Durability Calculation

The reduction of reinforcement strength, sectional dimension of reinforcing bar and bond strength between reinforcement and concrete is mainly caused by the corrosion of reinforcement. There are several key times including the initial corrosion time, the concrete cracking time, the time when crack width is great enough to influence the regular service (up to 1 mm) and the time when structure doesn’t satisfy the demands of bearing capacity.
5-2-1 Effect of Chloride Ingress Model

The initial time of corrosion, which is determined by the chloride ingress model, is the time when chloride concentration on reinforcement surface rises to the threshold value and reinforcement begin to corrode. As mentioned above, there are three chloride ingress models namely: the diffusion model with constant diffusion coefficient, the diffusion model with variable diffusion coefficient and the flow model for cracked concrete. Fig.4 shows the influence of different chloride ingress model on the initial corrosion time (i.e. chloride concentration on reinforcement surface rises to threshold) and Fig.5 shows the influence of the water-cement ratio and the thickness of concrete over on initial corrosion time.

Figure 3 Main factors influencing durability of reinforced concrete
5-2-2 Effect of Corrosion Velocity Model

The corrosion velocity model determines the concrete cracking time and the time when the crack width is great enough to influence the regular service and the time when structure doesn’t satisfy the demands of bearing capacity. That means that the times are determined by the amount of corrosion. The main corrosion velocity models are: the model with constant corrosion velocity (Niu Ditao model), the model with decreasing corrosion velocity (Kim model), the model with increasing corrosion velocity and the model with comprehensive corrosion velocity. Fig.6 shows the variation of corrosion velocity with time and Fig.7 shows the change curves of corrosion amount in different corrosion velocity model.

Table 2 gives the initial time of corrosion, the cracking time of concrete cover, the time of crack width reaching 1 mm and the relevant corrosion amounts for different corrosion velocity models.
Table 2 Controlling times of corrosion and relevant corrosion amounts

<table>
<thead>
<tr>
<th>Corrosion model</th>
<th>Initial time of corrosion (a)</th>
<th>Cracking of concrete cover Amount (%)</th>
<th>Crack width reaching 1 mm Amount (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Niu model</td>
<td>24.8</td>
<td>256.71</td>
<td>488.71</td>
</tr>
<tr>
<td>Li model</td>
<td>24.8</td>
<td>28.19</td>
<td>36.48</td>
</tr>
<tr>
<td>Kim model</td>
<td>24.8</td>
<td>27.59</td>
<td>41.09</td>
</tr>
<tr>
<td>Combined model</td>
<td>24.8</td>
<td>27.59</td>
<td>35.98</td>
</tr>
</tbody>
</table>

5-2-3 Effect of Relevant Parameters on the Durability Calculation

The main parameters influencing the calculation of reinforcement concrete durability includes: the chloride concentration on concrete surface, the threshold value of chloride concentration, the diffusion coefficient of chloride ions, the thickness of concrete cover and the ratio of water-cement, as well as strength properties of concrete.

(1) Effect of chloride concentration on concrete surface

Chloride diffusion is induced by the concentration gradient, and the amount of chloride ions diffused into concrete rises as the concentration on concrete surface grows. Aging, environmental conditions (such as the chloride concentration in seawater and the location of structure), material properties of concrete and absorbability to chloride are all contributed to the chloride concentration on concrete surface.

There are many research results on the chloride concentration of concrete surface. The Scientific Research Institute of the Fourth Harbor Engineering Bureau of the Ministry of Communication\(^{[18]}\), Nan Jing Hydraulic Research Institute\(^{[19]}\) etc. obtained the average chloride concentration on concrete surface in marine environment from experiment on site, given in Table 3.

Table 3 The chloride content on surface (% , wt. Concrete)

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Environment</th>
<th>Splash</th>
<th>Water level change</th>
<th>Under water</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fourth Harbor Engineering Bureau</td>
<td>W/C</td>
<td>0.55</td>
<td>0.45</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Chloride content</td>
<td>0.423</td>
<td>0.404</td>
<td>0.329</td>
</tr>
<tr>
<td>Nanjing Hydraulic Research Institute</td>
<td>W/C</td>
<td>0.55</td>
<td>0.53</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>Chloride content</td>
<td>0.513</td>
<td>0.443</td>
<td>0.436</td>
</tr>
<tr>
<td>Environment</td>
<td>Splash</td>
<td>Wave fog zone</td>
<td>Atmospheric zone</td>
<td></td>
</tr>
<tr>
<td>---------------------------------</td>
<td>--------</td>
<td>---------------</td>
<td>-----------------</td>
<td></td>
</tr>
<tr>
<td>Portland cement concrete</td>
<td>0.75%</td>
<td>0.5%</td>
<td>0.25%</td>
<td></td>
</tr>
<tr>
<td>Additive cement concrete</td>
<td>0.9%</td>
<td>0.6%</td>
<td>0.3%</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Region</th>
<th>Annual development speed of chloride concentration on surface</th>
<th>Maximum value of chloride concentration on surface (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Under water</td>
<td>Temporal</td>
<td>0.80</td>
</tr>
<tr>
<td>Splash</td>
<td>0.10</td>
<td>1.00</td>
</tr>
<tr>
<td>Less than 800m from coast</td>
<td>0.04</td>
<td>0.60</td>
</tr>
<tr>
<td>Less than 1500m from coast</td>
<td>0.02</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Based on the investigation of chloride concentration on concrete surface in marine splash zone in British, Bamforth obtains that the chloride concentration is 0.3%~0.7% of concrete weight and gives the values as shown in Table 4.

Thomas and Bemtz, who use Life-365 software to predict the service life of reinforcement concrete exposed in chloride environment, suggest that the values of chloride concentration can be determined from Table 5 if there is no reliable data.

As for marine atmospheric zone, on the basis of on-site detecting on 1158 bridges in Tasmania of Australia, McGee proposed that chloride concentration on concrete surface in offshore atmospheric zone is a function of distance from building to coast.

**2. Effect of chloride concentration threshold**

Chloride concentration is influenced by many factors such as C₃A content, alkali content, sulfate content, temperature, the additive amount of fly ash in concrete and the type of reinforcement etc. Table 6 shows the threshold of chloride concentration based on the investigation of harbor engineering in China and the exposure experiment of harbor engineering in south of China.

<table>
<thead>
<tr>
<th>Splash zone</th>
<th>Water level change zone</th>
<th>Submerged zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.154~0.221</td>
<td>0.250~0.379</td>
<td>0.298~0.483</td>
</tr>
</tbody>
</table>

Many researchers also give their results on the threshold of chloride concentration, as shown in table 7.
(3) Effect of chloride diffusion coefficient

The diffusion coefficient, which is related to concrete composition, the amount and feature of inner hole and hydrated degree, is an important index to reflect concrete durability. There are three main viewpoints on diffusion coefficient. The first theory regards diffusion coefficient as a constant. The second view believes that the diffusion coefficient varies with time, but does not vary with x. The third theory believes that the coefficient varies with x and t. According to exposure experiments, diffusion coefficient of chloride ions varies with time. However, few literatures show that the diffusion coefficient varies with x.

On the basis of experimental results, Mangat suggests that the index parameter $\alpha$ in the model with variable diffusion coefficient (see equation (3)) is related to the ratio of water to cement

$$\alpha = 3 \left(0.55 - \frac{W}{C}\right)$$

(36)

Literature presents the formula to calculate the chloride diffusion coefficient

$$D(t) = D_0 \left(\frac{t_0}{t}\right)^\alpha k_c k_e$$

(37)

where, $k_c$ is curing coefficient. If curing time is less than 28 days, it is needed to multiply $k_c$. $k_e$ is a environmental coefficient which is determined from table 8 considering different marine environment and different gelled materials; $\alpha$ is a time-dependant constant of diffusion coefficient related to gelled material and environmental condition as given in table 9.

It is suggested in American Life-365 prediction software that $\alpha$ is related to mix proportion of concrete, the type and amount of additives, and environmental condition etc.

$$\alpha = 0.2 + 0.4(F / 50 + K / 70)$$

(38)

where, F is the percentage of fly ash; K is the percentage of slag; the maximum value of $\alpha$ is 0.6.

Table 7 Threshold of chloride content (percentage of concrete weight)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Splash</td>
<td>Water level change</td>
<td>Submerged</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Threshold (%)</td>
<td>0.15</td>
<td>0.15</td>
<td>0.20</td>
<td>0.20~</td>
<td>0.20~</td>
<td>0.154~</td>
<td>0.298~</td>
</tr>
<tr>
<td></td>
<td>0.40</td>
<td>0.221</td>
<td>0.379</td>
<td>0.483</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 8 Environmental coefficient $k_e$

<table>
<thead>
<tr>
<th>Gelled material</th>
<th>Portland cement</th>
<th>Slag</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Submerged zone</td>
<td>Tidal zone</td>
</tr>
<tr>
<td>$k_e$</td>
<td>1.32</td>
<td>0.92</td>
</tr>
</tbody>
</table>

Table 9 Values of $\alpha$

<table>
<thead>
<tr>
<th>Marine environment</th>
<th>Portland cement</th>
<th>Fly ash</th>
<th>Slag</th>
<th>Silica powder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Submerged zone</td>
<td>0.30</td>
<td>0.69</td>
<td>0.71</td>
<td>0.62</td>
</tr>
<tr>
<td>Tidal and splash zone</td>
<td>0.37</td>
<td>0.93</td>
<td>0.60</td>
<td>0.39</td>
</tr>
<tr>
<td>Atmospheric zone</td>
<td>0.65</td>
<td>0.66</td>
<td>0.85</td>
<td>0.79</td>
</tr>
</tbody>
</table>

Table 10 Chloride diffusion coefficient of Zhan Jiang port exposure test for 3a, 5a and 10a

<table>
<thead>
<tr>
<th>Exposure zone</th>
<th>Splash zone</th>
<th>Water level change zone</th>
<th>Submerged zone</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.55</td>
<td>0.45</td>
<td>0.40</td>
</tr>
<tr>
<td>Ratio of water to cement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3a</td>
<td>6.42</td>
<td>4.17</td>
<td>4.17</td>
</tr>
<tr>
<td></td>
<td>13.31</td>
<td>9.05</td>
<td>7.18</td>
</tr>
<tr>
<td></td>
<td>33.40</td>
<td>17.77</td>
<td></td>
</tr>
<tr>
<td>5a</td>
<td>5.83</td>
<td>2.73</td>
<td>2.44</td>
</tr>
<tr>
<td></td>
<td>11.80</td>
<td>8.58</td>
<td>5.97</td>
</tr>
<tr>
<td></td>
<td>19.17</td>
<td>19.60</td>
<td></td>
</tr>
<tr>
<td>10a</td>
<td>6.00</td>
<td>2.56</td>
<td>1.98</td>
</tr>
<tr>
<td></td>
<td>21.33</td>
<td>15.74</td>
<td>7.34</td>
</tr>
<tr>
<td></td>
<td>27.22</td>
<td>49.60</td>
<td></td>
</tr>
</tbody>
</table>

The diffusion coefficient $D_0$ at service time $t_0$ is calculated by the relationship between diffusion coefficient of concrete curing for 28 days and the ratio of water to cement in American Life-365 software

$$D_0 = 10^{-12.06 + 2.49f / C}$$  \hspace{1cm} (39)

Wang Shengnian, et al\cite{2} (the Fourth Harbor Engineering Institute of the Ministry of Communication) give the chloride diffusion coefficient of Zhanjiang port exposure test for 3a, 5a and 10a, as shown in table 10.

Fig. 8 gives the variation of the chloride concentration with time for different values of chloride diffusion coefficients.
(4) Effect of concrete cover thickness

Concrete cover prevents reinforcement from corrosion. The thicker the concrete cover is, the time needed to reach reinforcement surface for external erosion medium such as chloride ions, oxygen and water etc. is longer, and the more durable the structure is.

However, for extremely thick concrete cover, the contraction stress and the temperature stress are not in control of reinforcement and cracks occur easily in fact. Because of cracks, protective action of concrete cover will sharply reduce, and even no longer operate when the crack width is greater than 0.1mm. Generally, thickness of concrete cover is not allowed to exceed 80~100mm and the specific dimension should be determined by structural design.

Quality Control Specification for Concrete in Marine Traffic Engineering JTJ269-96 promulgated in 1996 and Construction Code for Concrete in Marine Traffic Engineering JTJ268-96 specified that the minimum thickness of concrete cover is 65 mm, the maximum water-cement ratio should be less than 0.4 and fly ash, slag powder and silicon powder are permitted to add into concrete.

Table 11 and table 12 present the minimum thickness of concrete cover for reinforced concrete in marine environment and for pre-stressed reinforced concrete, respectively.

<table>
<thead>
<tr>
<th>Location</th>
<th>Atmospheric zone</th>
<th>Splash zone</th>
<th>Water level change zone</th>
<th>Submerged zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>North China</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>30</td>
</tr>
<tr>
<td>South China</td>
<td>50</td>
<td>65</td>
<td>30</td>
<td>30</td>
</tr>
</tbody>
</table>
Fig. 9 gives the variation of reinforcement corrosion rate with time for different thickness of concrete cover. Table 13 gives the initial time of corrosion, the cracking time of concrete cover, the time of crack width reaching 1 mm and the relevant corrosion amounts for different thickness of concrete cover.

Table 12 Minimum thickness of concrete cover for pre-stressed reinforced concrete (mm)

<table>
<thead>
<tr>
<th>Location</th>
<th>Atmospheric zone</th>
<th>Splash zone</th>
<th>Water level change zone</th>
<th>Submerged zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of cover</td>
<td>75</td>
<td>90</td>
<td>75</td>
<td>75</td>
</tr>
</tbody>
</table>

Table 13 Controlling times of corrosion and relevant corrosion amounts

<table>
<thead>
<tr>
<th>Thickness of concrete cover</th>
<th>Initial time of corrosion (a)</th>
<th>Cracking of concrete cover</th>
<th>Crack width reaching 1 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time (a)</td>
<td>Amount $\rho$ s(%)</td>
<td>Time (a)</td>
</tr>
<tr>
<td>C=40mm</td>
<td>16.2</td>
<td>17.4</td>
<td>0.34</td>
</tr>
<tr>
<td>C=50mm</td>
<td>29.9</td>
<td>32.0</td>
<td>0.41</td>
</tr>
<tr>
<td>C=60mm</td>
<td>46.6</td>
<td>50.0</td>
<td>0.48</td>
</tr>
<tr>
<td>C=65mm</td>
<td>56.1</td>
<td>60.3</td>
<td>0.52</td>
</tr>
</tbody>
</table>

(5) Effect of water-cement ratio

Fig. 10 shows the variation of reinforcement corrosion rate with time for different water-cement ratios. Table 14 gives the initial time of corrosion, the cracking time of concrete cover, the time of crack width reaching 1 mm and the relevant corrosion amounts for different water-cement ratios.

Table 14 Controlling times of corrosion for different water-cement ratios

<table>
<thead>
<tr>
<th>Water-cement ratio</th>
<th>Initial time of corrosion (a)</th>
<th>Cracking of concrete cover</th>
<th>Crack width reaching 1 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time (a)</td>
<td>Amount $\rho$ s(%)</td>
<td>Time (a)</td>
</tr>
<tr>
<td>w/c=0.40</td>
<td>46.6</td>
<td>50.0</td>
<td>0.48</td>
</tr>
<tr>
<td>w/c=0.45</td>
<td>24.8</td>
<td>27.6</td>
<td>0.48</td>
</tr>
<tr>
<td>w/c=0.55</td>
<td>11.6</td>
<td>13.3</td>
<td>0.47</td>
</tr>
</tbody>
</table>
(6) Effect of concrete strength and concrete type

Fig. 11 shows the variation of reinforcement corrosion rate with time for different concrete types. Table 15 gives the initial time of corrosion, the cracking time of concrete cover, the time of crack width reaching 1 mm and the relevant corrosion amounts for different concrete types.

Table 15: Controlling times of corrosion for different concrete types

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Initial time of corrosion (a)</th>
<th>Cracking of concrete cover</th>
<th>Crack width reaching 1 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time (a)</td>
<td>Amount $\rho_s$ (%)</td>
<td>Time (a)</td>
</tr>
<tr>
<td>C30</td>
<td>29.9</td>
<td>1.8</td>
<td>7.3</td>
</tr>
<tr>
<td>C40</td>
<td>29.9</td>
<td>2.1</td>
<td>7.7</td>
</tr>
<tr>
<td>C50</td>
<td>29.9</td>
<td>2.2</td>
<td>8.0</td>
</tr>
</tbody>
</table>

6 Conclusions

Various calculation models of reinforcement corrosion process are presented and the results calculated from different models are scattered. Some models, such as Niu Ditao model is established for atmospheric environment, they are not suitable for corrosion calculation in marine environment. Some models are obtained in marine environment, but their basis is experimental data for un-cracked concrete, such as Kim model with corrosion rate decreasing with time, they are not definitely suitable for corrosion calculation of cracked concrete. Some models are based on the experimental data for concrete with loads induced cracks in marine environment, such as Chun Qing Li model with corrosion rate increasing with time,
they are not definitely suitable for un-cracked concrete or concrete with cracks induced by corrosion. In this paper, Kim model is adopted before concrete cracks, in which the corrosion velocity decreases with time, while Chun Qing Li model is adopted after concrete cracks, in which the corrosion velocity increases with time. The combined model is presented to simulate reinforcement corrosion process before and after the concrete cracking in marine environment, but the model is in lack of experimental verification. Further investigations are needed on the reinforcement corrosion model of reinforced concrete structures in marine environment.

Chloride concentration on concrete surface, chloride diffusion coefficient, threshold chloride concentration and water-cement ratio etc. have great effect on the durability of reinforced concrete structures. The values of parameters in the models determined by different literatures are quite scattered because of the different environmental conditions, measurement errors, statistical errors and analytical errors. It is important to choose the rational model and the model parameters according to specific environmental condition.

In the case of small safety storage of structures, if the variation of bearing capacity with time is not considered, the safety of reinforced concrete structures will probably not satisfy the designed safety due to corrosion of reinforcement before concrete cracks, i.e the structure reaches the limit state of bearing capacity before reaching the serviceability limit state. So it is necessary to consider the performance of bearing capacity varying with time in the design of reinforced concrete structures.

7 Acknowledgements

Foundation item: Supported by the Nation Natural Science Foundation of China under Grant 50279027 Biography: Wang Yuanzhan (1958), male, Hebei Province, China, Professor, Ph.D.

8 References


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