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A Performance-Based Evaluation for a Reinforced Concrete Member Subjected to Chloride Ingress

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Summary

A framework of a performance-based evaluation of a reinforced concrete member subjected to chloride ingress is summarized. Chloride profile in cover and the propagation of chloride-induced corrosion of a rebar embedded in the concrete are quantified. The amount of corrosion products which would increase with time is used to calculate the time for a longitudinal crack due to the corrosion to occur and its width. The width of the longitudinal crack is then used as an index for objective judgement in evaluating the performance of the concrete member.

1. Degradation Process

A steel reinforcement in concrete will rust after chloride ions penetrating in the cover reach its surface and accumulate at a threshold concentration. The cross sectional area of the corroding rebar is reduced with the progress of corrosion resulting in the decreased flexural and/or shear strength of the member. Therefore, the corrosion of a steel reinforcement can affect adversely the safety and serviceability of concrete structures and hence shorten the service life.

The performance of a concrete structure should be examined in a time-dependent phenomena where respect mechanisms involved may need to be quantified on a rational basis⁽¹⁾. Fig.1 shows a progressive loss of the performance of a concrete member subjected to chloride ingress. The degree of the corrosion of a steel-reinforcement is responsible for its performance. Each periods in Fig.1 are normally explained as follows:

- (I)Initiation period(Δt_1)
- (II)Propagation period(Δt_2)
- (III)Acceleration period(Δt_3)

Penetration of chloride ions in concrete and the accumulation on the surface of a rebar to a threshold concentration
 Progression of corrosion with oxygen and water supply
 Advance of corrosion due to increased corrosion rates after the formation of longitudinal cracks and loss of structural performance due to decreased cross section of rebar

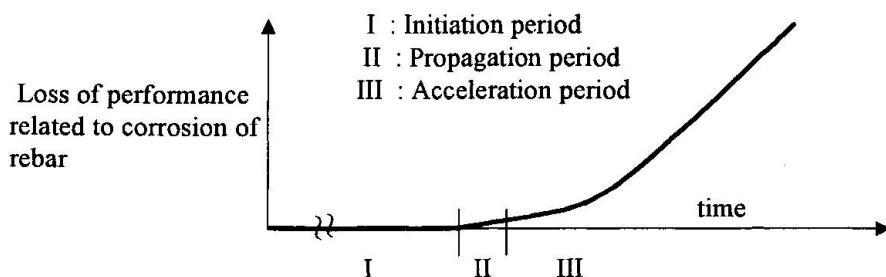


Fig.1 Time-dependent change in the performance of concrete structures with chloride ingress



2. Methodology of the Quantification of Chloride-Induced Corrosion

Several methodologies have been proposed to predict each period in a given stage. Particularly, a number of mathematical models have been used to predict an initiation period. Mathematical models to calculate the period of an initiation stage on the basis of the Fick's first and second laws were introduced in a state-of-the-art report published from the JCI research committee in 1996⁽²⁾. It may be appropriate at present to assume that the movement of chloride ions through concrete follows the diffusion theory.

Propagation and acceleration periods may be calculated with the corrosion rate of an corroding reinforcement in cover. The corrosion rate is normally calculated on the basis of the electro-chemical theory. A state-of-the-art report on related subjects about the corrosion of rebar has been published from a research committee in the JCI in 1996⁽³⁾ and in the JSCE in 1997⁽⁴⁾.

It must be acknowledged that the rate of chloride penetration through concrete and the corrosion rate of corroding reinforcements are calculated with a large number of assumptions and simplified models for the time being. Future works are definitely needed to verify assumptions and hence modify analytical models used to increase the precision of the prediction of the performance of a concrete structure subjected to chloride ingress.

2.1 Initiation periods, Δt_1

A diffusion model may be used to calculate an initiation period. A governing equation for chloride ions diffusion in concrete may be given by the Fick's first law. On the other hands, for calculating the profile of chloride content in concrete the Fick's second law is normally used for the mass conservation. In addition to diffusion process, the transport of chloride ions may occur in flowing water. Especially, when wetting condition arrives at concrete surface after drying cycle, a large amount of moisture is absorbed into pores due to the capillary suction. Therefore, chloride transport through concrete may follow a global equation given below⁽⁵⁾:

$$\frac{\partial C_t}{\partial t} = -\operatorname{div}[D_a \cdot \nabla(C_f)] + C_f J_w \quad (1)$$

C_t : total amount of chloride content in concrete (kg/m^3), C_f : the amount of free chloride ions in pores (kg/m^3), D_a : nominal diffusion coefficient (cm^2/s), J_w : moisture flux ($1/\text{s}$)

In addition to quality of concrete which is controlled by mix proportions, materials and constructions, environmental conditions have significant effects on the transport of chloride ions through concrete. The total content of chloride ions is a product of a free chloride concentration in pore solution added to the amount of chlorides bound in cement hydrates. Then, the binding isotherm of chloride ions in concrete is important, which is primarily controlled by binder types, its amount and concrete age. The diffusion coefficient of chloride ions may be controlled by a microstructure, moisture profile and temperature in concrete. Furthermore, the boundary condition for the differential equation shown in Eq.(1) must be properly determined. For a diffusion equation the change of surface concentration of chloride ions (C_s , $x=0$) with time, t is normally used as the boundary condition, i.e. $C = C_s$, $x=0$, $t \geq 0$ for one dimensional analysis. Therefore, it is largely controlled by such environmental conditions as the amount of airborne salt, rainfall, relative humidity, the amount of deicing agent if used during winter season, temperature and so on. For concrete submerged in sea water, the surface concentration of chloride ions may be constant while it is fluctuated with time for concrete above ground due to rain, carbonation and the periodic change of the amount of airborne salt. Thus the determination of the boundary conditions is complex. The survey of concrete structures above ground in chloride-laden atmospheres shows that the concentration of chloride ions near the surface area, about 0.5 cm deep from the surface, is changed with time⁽⁴⁾ and may be modeled as follows:

$$C_s = C_0 \{1 - \exp(-\beta t)\} \quad (2)$$

C_0 and β are constants and should be determined by an unique characteristic in service



environments and quality of surface concrete. Therefore, Eq.(2) may be used as the boundary conditions for concrete above ground.

For a reinforced concrete subjected to flexure, flexural cracks are allowed to occur within a limited width. However, the transport of chloride ions through a crack in concrete is difficult to be quantified. For the time being, the average transport of chloride ions in a cracked concrete is modeled with an increased diffusion coefficient of chloride ions or an increased void ratio.

2.2 Calculation of Propagation Period, Δt_2 and Acceleration Period, Δt_3

Propagation and acceleration periods may be calculated with the amount of corrosion product and corrosion rate. This is because the development of a longitudinal crack is a time-dependent process and related closely to an increased amount of corrosion product with time.

When an expansive stress induced by corrosion reaches the tensile force of cover concrete, longitudinal cracks will occur. Then, a propagation period may be calculated as follows:

$$\Delta t_2 = \frac{W_{cr.}}{VW_{corr.}} \quad (3)$$

Where $W_{cr.}$ is the amount of corrosion product at the occurrence of longitudinal cracks in mg/cm^2 and $VW_{corr.}$ is the corrosion rate in $\text{mg/cm}^2/\text{year}$.

Corrosion current is necessary to calculate the corrosion rate and may be obtained from a current circuit model. Corrosion current density is generally expressed as follows:

$$I_{corr.} = \frac{\Delta E}{r_A + r_C + \frac{r_{Con.}}{k}} \quad (4)$$

Where $I_{corr.}$: corrosion current density in mA/cm^2 , ΔE : potential difference between anode and cathode in mV, r_A and r_C : anode and cathode polarization resistance, respectively in $\Omega \cdot \text{cm}^2$, $r_{Con.}$: concrete resistivity in $\Omega \cdot \text{cm}$, k : cell constant in cm^{-1} .

Normally chloride-induced corrosion is in the process of macrocell corrosion where the anode area is created by chloride accumulation while the cathode area is created far from the anode and is supported by the oxygen. The Faraday's law can be used to calculate corrosion current with the amount of oxygen reduced at the cathode, where the corrosion area must be specified. The resistivity of concrete is also necessary for corrosion current, which is converted to a corrosion rate. The amount of corrosion product increases with the continuous flow of corrosion current. In concrete structures which suffer from the chloride attack in a marine atmosphere, the relative humidity in the cover is enough high to motive the corrosion process that is controlled by the diffusion rate of oxygen. Therefore, the diffusion of oxygen in concrete and its amount available for the reduction at the cathode is important to determine the corrosion rate⁽¹⁾. Corrosion rate increases to the maximum when the relative humidity in cover concrete ranges from 60 to 80 %⁽⁴⁾. In concrete structures submerged in sea water, little oxygen can transmit to the surface of rebar and hence the corrosion rate is low. In addition, corrosion rate may be negligibly low for dry concrete although oxygen can diffuse easily. The resistivity of the concrete becomes higher resulting in the reduction of the corrosion current. Therefore, the magnitude of concrete resistivity controls the corrosion process in the dry concrete. The ratio of anode to cathode area are thought to decrease with time and hence the corrosion rate is a time-dependent process⁽¹⁾.

The amount of corrosion product, W_{cr} at the occurrence of longitudinal cracks can be calculated with the applied linear elastic analysis for a thick-wall cylinder⁽¹⁾. Therefore, physical characteristics of a corrosion product and the arrangement of reinforcements in concrete play an significant role as well as the tensile strength of concrete. In addition, drying and shrinkage and dead and live loads applied are responsible partially to the formation of longitudinal cracks. Few studies of these effects on crack formation have been carried out. Furthermore, cover concrete



may be spalled off with a given amount of corrosion product. However, the spalling of cover concrete may be related to the arrangement of reinforcements and their diameter⁽⁴⁾.

Although a corrosion rate after a longitudinal crack occurs is difficult to be quantified because of insufficient knowledge of its mechanism and the lack of field data, acceleration period will be calculated as follows:

$$\Delta t_3 = \frac{W_{corr.}}{VW_{cr.}} \quad (5)$$

Where $W_{corr.}$ is the amount of corrosion product necessary for a given crack width in mg/cm^2 and $VW_{cr.}$ is the corrosion rate after the occurrence of longitudinal cracks in $\text{mg/cm}^2/\text{year}$.

Laboratory studies show that with a given diameter of rebar and cover depth, a crack width appears to increase linearly with the amount of corrosion product. However, there are few studies on the corrosion rate after the occurrence of longitudinal cracks. Therefore, the result of the corrosion rate obtained from experiments where bare bars have been exposed directly to marine environments may be used.

3 Sample calculation

A performance-based evaluation of a reinforced concrete beam exposed to a chloride-laden atmosphere is conducted. The required performance is assumed in this particular example that the width of a longitudinal crack due to corrosion of rebar is less than 0.8mm for considering its serviceability. Construction works are assumed to be carried out properly according to a construction manual so that unfavorable effects on the chloride penetration can never exist.

3.1 Quantified Environments

Periodic change in temperature and air borne salt are shown in Fig.2 and Fig.3. The amount of airborne salt increases in winter season since relatively high ocean wave occurs due to the wind heavily blowing. Other meteorological data is provided in Table 1.

Table 1 Meteorological data

Temp.	RH in average.	Airborne salt	Rainfall
Max. 25 °C	65 %	Max. 0.62 mg-Cl/cm ² /day	1200mm/ year
Min. 2 °C	80% (microclimate effects)	Min. 0.62x10 ⁻² mg-Cl/cm ² /day	

If a river flows underneath the bridge wet conditions may be maintained in the bottom part of the beam. Also, its face is hardly subjected directly to the sunshine and heavy rain- shower. These microclimates are important to the transport of chloride ions and are taken into account to calculate RH in the concrete cover and is assumed to change with time (see Fig.4).

3.2 Calculation of Chloride Penetration

Initiation period is calculated with Eq.(1) where $C_f J_w$ is neglected since the transport of chloride ions dragging into flowing water due to absorption is limited near surface area in this particular example. For the boundary condition Eq.(2) is used where constants, C_s and β , are properly determined from environments, especially the amount of airborne salt and the annual rainfall. Then the boundary condition is assumed in Fig.5. Chloride diffusion coefficient changes with increased hydration periods the periodic change of temperature and relative humidity in cover. Also, the porosity of concrete surface may be different from the porosity of the inner concrete due partially to a curing method. The binding isotherm of chloride ions can be referred from the past study of similar materials⁽⁶⁾. The flexural cracks will be present on the tension region of the beam. Its effect is taken into account by increasing the average porosity of the cover.

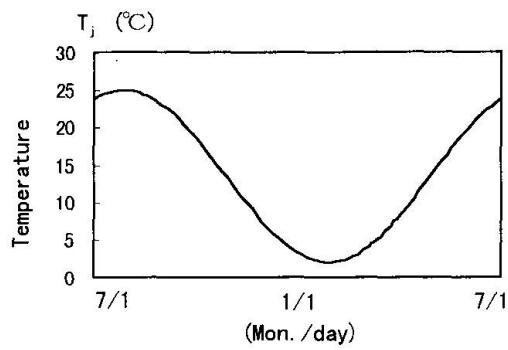


Fig. 2 Seasonal change of temperature

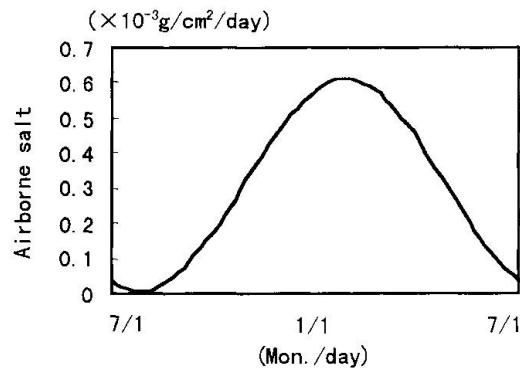


Fig. 3 Seasonal change of airborne salt

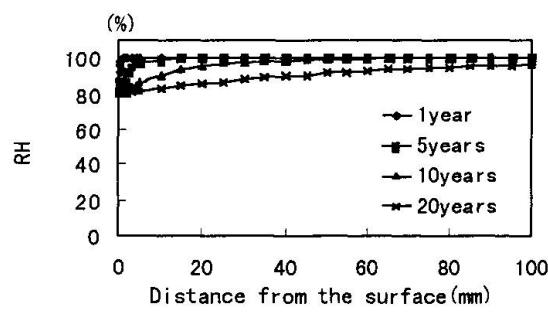


Fig. 4 Decrease of relative humidity in cover

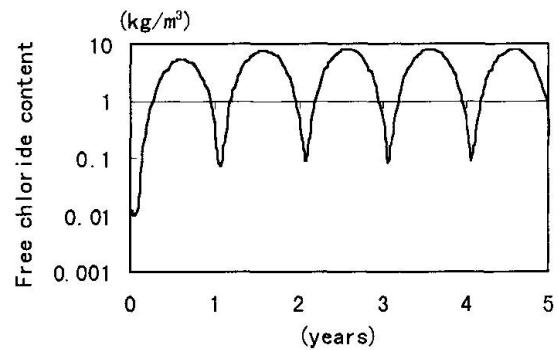
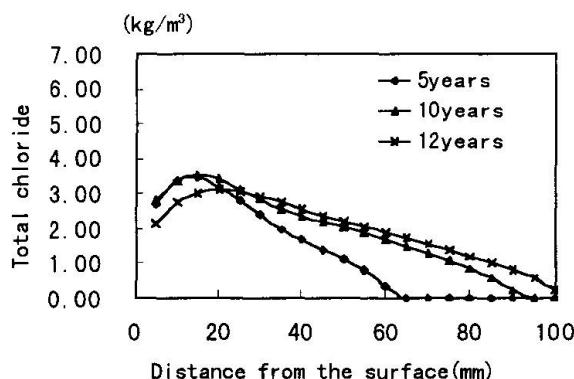
Fig. 5 Boundary conditions ($x=0, t>0$)

Fig. 6 Chloride profile in cover

The profiles of chloride concentration in concrete cover after 5, 10 and 12 years are shown in Fig. 6. A threshold content at rebar surface for chloride ions to initiate corrosion is 1.2 kg/m^3 on the basis of total amount of chloride content. Also, the cover depth is equal to 84 mm. Therefore the chloride content at an depth of 84 mm reaches 1.2 kg/m^3 at an period of 12.4 years. It means that the steel embedded in the cover starts to rust at an service period of 12.4 years.

3.3 Calculation of Corrosion Rate

To calculate the corrosion rate Eq(3) is used. However, the prediction of the corrosion rate would be more difficult as compared with the chloride analysis at this present. Then, each values necessary for the calculation are determined from experimental data⁽⁴⁾ and summarized in Table 2. The corrosion rate results in $28.2 \text{ mg/cm}^2/\text{year}$. In the calculation the concrete quality and environmental conditions unique to this particular case should be taken into consideration.

A linear elastic analysis⁽³⁾ is conducted to calculate the amount of corrosion at the occurrence of longitudinal cracks. Cover concrete responses elastically against expansion force induced by

*Table 2 Values for the calculation of the corrosion rate and the results*

ΔE :	r_A	r_C	r_{Con}
300 mV	0 (due to effects of flexural cracks)	$80 \text{ k}\Omega \cdot \text{cm}^2$	$13 \text{ k}\Omega \cdot \text{cm}$
I_{corr}	i_{corr} (corrosion current)	VW_{corr}	W_{cr}
$3.1 \mu \text{A}/\text{cm}^2$	$49.7 \mu \text{A}$	$28.2 \text{ mg}/\text{cm}^2/\text{year}$	$15.8 \text{ mg}/\text{cm}^2$

corrosion product. A crack occurs when the tensile stress induced by the expansion reaches the tensile strength of cover concrete. This corrosion amount results in $15.8 \text{ mg}/\text{cm}^2$. Therefore, the propagation period is 0.56 years.

3.4 Calculation of Crack Width with the Amount of Corrosion

The corrosion rate after the occurrence of longitudinal cracks is determined on the basis of experimental data⁽¹⁾ where bare bars have been exposed to a tropical marine atmosphere for 3.5 years. Then, it is assumed that the corrosion rate would increase with square root time as shown in Table 3. In addition, the width of the longitudinal crack would increase linearly with the amount of corrosion product and the equation for this particular example is shown in Table 3 where constants are determined with bar diameter and cover depth.

Table 3 Corrosion rate after longitudinal crack and relationship with the crack width

VW^*_{cr}	W_{corr}	$W_{corr} = a w_{cr} + b$
$86\sqrt{\Delta t_3} \text{ mg}/\text{cm}^2$	$74 \text{ mg}/\text{cm}^2$	$W_{corr} = 80 w_{cr} + 10$

VW^*_{cr} : the corrosion rate after longitudinal crack is modeled in a square root time relation.

3.5 Summary of the Sample Calculation

The results of the initiation, propagation and acceleration periods are summarized in Table 4.

Table 4 Results of the initiation, propagation and acceleration periods

Δt_1	Δt_2	Δt_3	$\Delta t_1 + \Delta t_2 + \Delta t_3$
12.4 years	0.56 years	0.74 years	13.7 years

This shows that after a service period of 13.7 years the width of a corrosion-related longitudinal crack reaches 0.8 mm. It seems that the results of Δt_2 and Δt_3 are too short. Assumptions to calculate these periods might be conservative. Based on the results, if an intended service period for the given required performance, *i.e.* a crack width is below 0.8 mm is longer than 13.7 years then materials properties and cover depth and so on need to be changed to meet the requirement. Also, a repair strategy and a protection methodology such as surface coating must be planned.

4. Conclusion

A performance-based evaluation for a reinforced concrete beam in a given chloride-laden environment was introduced. A sample calculation was conducted where a required performance was examined on a rational basis. The concept of the examination will be included in a future design methodology for concrete structures subjected to chloride ingress. However, uncertainties in each mechanisms involved and analytical methods may need to be justified with some safety factors. Future works will be directed to increase the reliability of the examination.

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