Fifth International Conference on Sustainable Construction Materials and Technologies. http://www.claisse.info/Proceedings.htm

Estimation on Deterioration Process Model of Concrete Structure received Chloride induced Damage with considering Repeated Repairing

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ABSTRACT

The deterioration of concrete structures by the corrosion of reinforcement due to chloride induced damage has recently been observed in coastal areas in Japan. This paper focuses on a simulation model of this deterioration process. The target structure is a coastal area drainage flood gate made of concrete. It has undergone 3 repairs since its construction 30 years prior to 2000. The simulation model is based on a probabilistic method and then compared with actual data. Results of the simulation are consistent with the actual deterioration process.

INTRODUCTION

The deterioration of concrete structures in coastal areas, mainly caused by the corrosion of reinforcement due to the action of chloride ions, has recently received a lot attention in Japan. The corrosion of steel material in concrete is commonly caused by an electrochemical reaction due to the migration of airborne chloride ions. Research has been conducted on the migrating characteristics of airborne chlorides into concrete with respect to parameters on chloride ions concentration and the relationship between the concentration and steel corrosion in actual structures. It is very important for the design and the construction work to estimate the amount of deterioration in which a structure will suffer during its service life. Bazant proposed a prediction model of structural service life based on the physical model of steel corrosion in concrete. Brown et al. presented an evaluation of structural integrity and prediction of its service life.

A model of deterioration due to chloride-induced damage based on the stochastic approach is proposed in this paper. Equivalent diffusion and Error of cover thickness were chosen as the stochastic parameters. The target structure is a concrete slab of flood gate which has received chloride induced damage in situ. The results of the proposed model are compared with the survey data.

DETERIORATION MODEL

Deterioration Model based stochastic approach

A probabilistic model was developed to represent the deterioration of concrete structures using a deterioration process model shown in Figure 1. The parameters involved in the model are expected to vary extensively since the phenomenon takes place in a natural environment. The deterioration process consists of three phases: 1) The incubation period t_s until depassivation of the reinforcement occurs after completion of the structure 2) The incubation period t_{cr} commencing from the moment of depassivation and including the development of corrosion at a perceptible rate until the limit state is attained when

cracking appears 3) The accelerative period which occurs after cracking.
In this paper, two parameters, equivalent diffusion and cover thickness is modeled as stochastic parameters. Others are governed by the variation of two stochastic parameters.

Incubation Period

The penetration phenomenon of chloride ions is described by Fick's diffusion equation. Chloride ions density $C^*(x,t)$ at depth from surface concrete x and time t is obtained as shown in Eq.(1) by the calculus of finite differences. The symbol * in the equation indicates stochastic parameters.

$$C(x,t+\Delta t)=C(x,t)+\{C(x+\Delta x,t)-2C(x,t)+C(x-\Delta x,t)\}\frac{D_C{}^*\Delta t}{\Delta x^2} \quad (1)$$

Where, D_c is the Equivalent Diffusion Coefficient (cm^2/sec) and Δt is the time interval (sec).

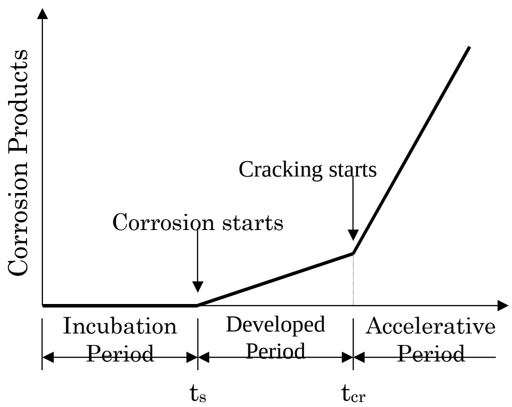

Figure 1. Deterioration process

Boundary condition at surface concrete of Eq.(1) is shown as:

$$C(x=0,t+\Delta t)=C(x=0,t)+W \quad (2)$$

Where, W is the constant value per time of chloride ion density penetrates into concrete (wt%/cm^2/sec).
It can be noted that the chloride ion density at surface concrete increases with elapsed time. D_c^* is obtained by the regression analysis of concrete core sampled in situ. Moreover, from an existing study it was found that the diffusion coefficient decreases exponentially as time increases. In this study, the equivalent diffusion coefficient model with the time depending $D_c^*(t)$ is proposed as shown in Eq.(3):

$$D^*_C(t)=\left(\frac{t_{cr}}{t}\right)^m D^*_s \quad (3)$$

Where, t_{cr} is the Reference year (= 5 elapsed year), D_5^* is the Equivalent diffusion coefficient at 5 elapsed years, t is the elapsed year, m is the coefficient, and m is obtained from an existing study (=0.54).

Corrosion Starts

The beginning time t_s* of steel corrosion is accepted as the time that the chloride ion density C^* (X_t^*,t) close to the steel reaches the threshold chloride ion density C_{cr}^*.

Therefore, the beginning time of steel corrosion is described in Eq.(4):

$$t^*_s = t \quad \text{when} \quad C^*\left(X_t^*, t\right) \geq C^*_{cr} \tag{4}$$

The beginning time t_s^* of steel corrosion is described as the stochastic parameter. The threshold chloride ion density C_{cr}^* is determined by the quality of concrete and environmental conditions. C_{cr}^* is described by parameter W/C* indicating the quality of concrete as shown in Figure 2. The relationship between W/C* and C_{cr}^* is obtained in Eq.(5) based on existing experiments. The symbol ○ indicates the experimental results. Figure 3. shows the relationship between the equivalent diffusion coefficient D_c and the water-cement ratio W/C*. Since JSCE standards reflect safety as shown in Figure ? the proposed model in this paper used Eq.(6):

$$C_{cr}^* = 3.7 \times \left(1.0 - 9.87\left(W/C^* - 0.45\right)^2\right) \tag{5}$$

$$W/C^* = 0.5 \times \left(\frac{D_C^*}{1.0 \times 10^{-8}}\right)^{0.15} \tag{6}$$

Where, W/C* is the water-cement ratio, D_c^* is the equivalent diffusion coefficient and C_{cr}^* is the threshold chloride ion density. Therefore, C_{cr}^* can be obtained using the combination of the relationship of water-cement ratio and equivalent diffusion coefficient.

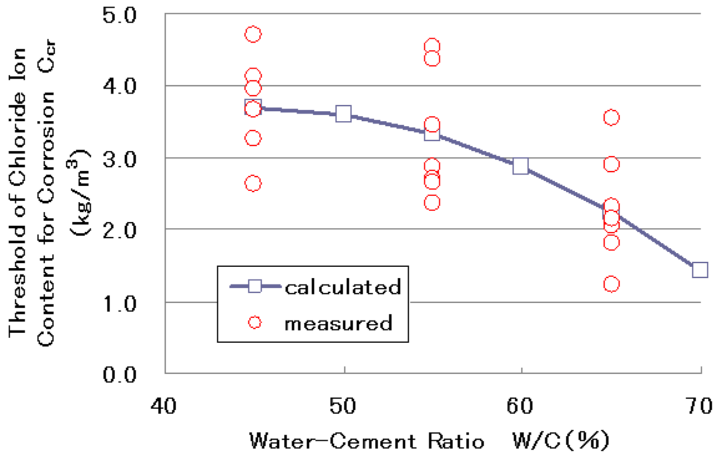

Figure 2. Threshold of Chloride Ion and Water-Cement Ratio (○ indicates experimental data.)

Progressive Period

The progressive period t_{cr}^* is the process by which corrosion cracking occurs due to the pressure from increasing corrosion over time. The estimated equation of corrosion rate can be obtained from the regression analysis developed by using experimental exposure data.

The parameters of the regression equation are the mean temperature per year in concrete, the chloride ion density and the quality of concrete. The regression equation of corrosion rate is described in Eq.(7):

$$R\left(T, C, D_C\right) = C_D\left(D_C\right) R\left(T, C\right) \tag{7}$$

Where, $R(T,C,D_c)$ is the corrosion rate of reinforcement ($mg/cm^2/year$) and $C_D(D_c)$ is the influence coefficient of concrete quality.

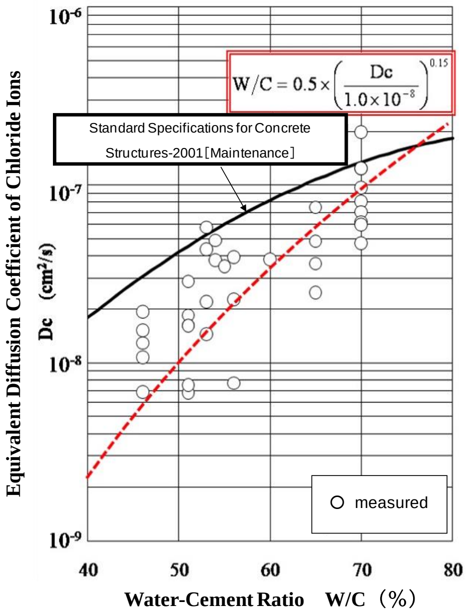

Figure 3. Equivalent Diffusion Coefficients and Water-Cement Ratio (○ indicates experimental data.)

$$R(T,C) = 0.27R_0(T,C)^{1.38} \tag{8}$$

$$R_0(T,C) = R_0 \cdot C_T(T)C_C(C) \tag{9}$$

Where, R_0 is the reference corrosion rate ($21.33mg/cm^2/year$), $C_T(T)$ is the influence coefficient of concrete temperature, $C_c(C)$ is the influence coefficient of chloride ion density at thickness of cover, C is the chloride ion density close to reinforcement (kg/m^3) and T is the Mean temperature per year in concrete.
Eq.(10) to (12) indicates the function of influence parameters for obtaining corrosion rate of reinforcement.

$$C_T(T) = \exp\{-2.593 \times (1000/K) + 8.695\} \tag{10}$$

$$C_C^{*}(C) = 1.93\sqrt{1 - \frac{(C^{*} - 12.0)^2}{(12.0 - C_{cr}^{*})^2}} \tag{11}$$

$$C_D^{*}(D_C) = 0.1129\left(D_C^{*} \cdot \frac{0.419}{t^{-0.54}}\right) \tag{12}$$

Where, K is the Absolute temperature per year (=T+273.15) and t is the elapsed year at investigated year.

OCURANCE OF CRACKING DUE TO CORROSION PRESSURE

Cracking modes at the surface of concrete are governed by cover thickness, diameter of reinforcement and the interval between reinforcements. The prediction of cracking modes is an important part of structure maintenance. The prediction model of when cracking occurs or when the corrosion pressure exceeds tension strength of concrete is also developed in this paper. Cracking occurs in three distinct patterns: cracking along the reinforcement, the spalling of cover concrete and the horizontal cracking between reinforcements. In addition, cracking along the reinforcement and horizontal cracking between reinforcements often develops to the spalling of cover concrete. The boundary between the cracking along the reinforcement, the spalling of cover concrete and the horizontal cracking between reinforcements is shown in Figure 5. Where thin cover concrete and wide intervals between reinforcements were used, spalling occured. Where thick cover concrete and narrow intervals between reinforcements were used, horizontal cracking between reinforcements occurred. Those cracking modes are described by the none dimensional parameters C/φ and C/L. C indicates the thickness of cover concrete, Φ indicates the diameter of reinforcement and L indicates the interval between reinforcements. Authors obtain those boundaries using the nondimensional parameters C/φ and C/L from existing exposure experiments.

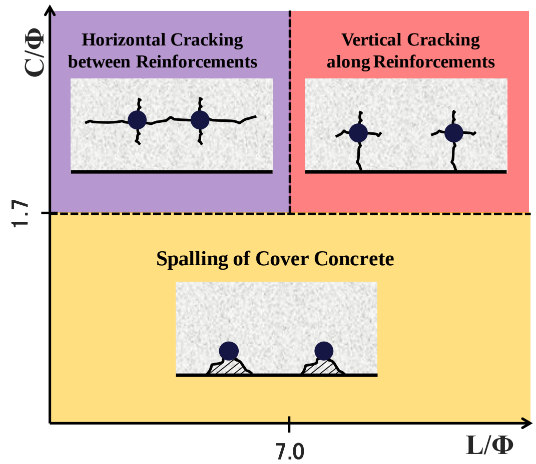

Figure 4. Boundaries of Cracking Modes

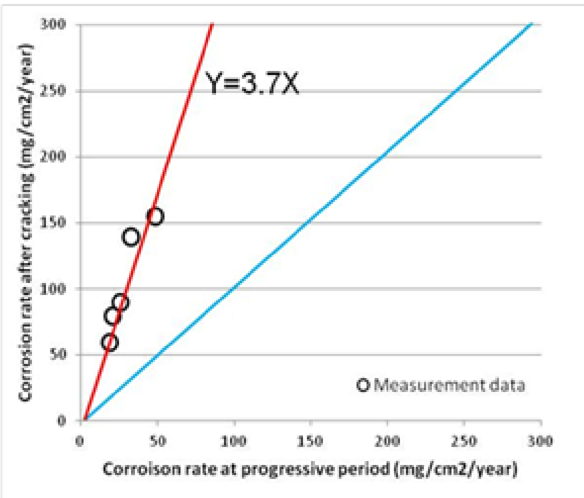

Figure 5. Comparisons with Measured and Predicted Values of Corrosion Products

Acceleration Period

After the occurrence of cracking, the parameter used in computations depends only on the corrosion rate because steel bars corrode more rapidly due to the easy penetration of water and oxygen through cracking.

Figure 5. shows the relationship between the estimated corrosion rates during the progressive period and the measurement corrosion rates after cracking, namely accelerative period. The symbol ○ indicates the survey data of corrosion speed obtained in situ using a non-destructive measurement method. According to Figure 5, the corrosion rate of steel bar during the accelerative period was 3.7 times faster.

$$R'(T,C,D_C) = 3.9 \cdot R(T,C,D_C) \tag{13}$$

Repeated repairing

Figure 6. shows the flow of chloride ions after and before repairing the surface concrete. Chloride ions penetrates into repaired concrete not only from outside but also from inside concrete after repairing.

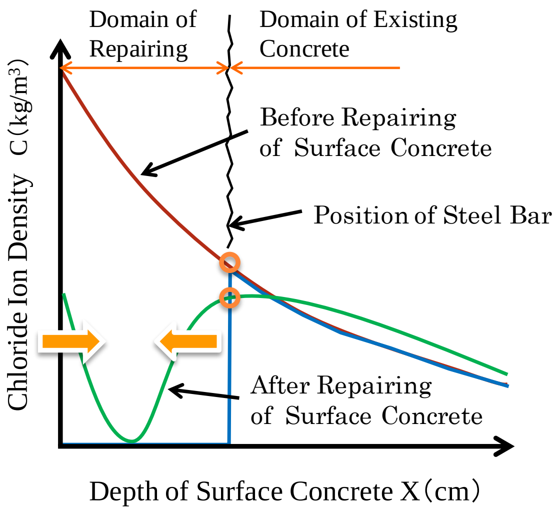

Figure 6. Behavior of Chloride Ions after/before Repairing

Simulation Model

Corrosion of reinforcement in RC structures doesn't develop in the same way because the corrosion configuration is described as a macro-cell phenomenon. The intervals between reinforcements and the thickness of cover concrete, as well as the diffusion coefficient to corrosion rates can all vary by the placement of structures. The parameters involved in this model are also expected to vary extensively since the phenomenon takes place in the natural environment. Therefore, the model of deterioration due to chloride-induced damage based on a stochastic approach is proposed in this paper. Equivalent diffusion and cover thickness were chosen as the stochastic parameters.

SIMULATION IN SITU

Target Structure

The target structure is a concrete slab located at a sea side flood gate in South Japan. 30 years have passed since its construction prior to 2000. Upon visual inspection in situ, the upper slab was deteriorated and cracking at the surface. Spalling of cover concrete had also occurred. All of the steel bars in the slab were corroded and we can easily see the spalling of cover concrete and rust stains on the surface of the upper slab. It seems that half of the upper slab was deteriorated by chloride-induced damage from visual inspection. Figure 7. shows the diagram of the repairing process. Repairs were carried out three times since its construction more than 30 years ago which each repair needing less time. We can assume the repairing rate to be the ratio of repairing area / all of slab area in the Figure. However, the performance of coating material used for repairs in those days was under developed and therefore neglected in our study.

Deterioration Process in Computation

The numerical method proposed in this paper was carried out using a target slab of concrete from a flood gate. Table 1. shows the values used in computation to predict the deterioration of the slab using the Monte Carlo simulation with 3000 random numbers. The main bar and reinforcement bar used was D16. The interval between reinforcements in design was 150mm. Cover thickness of the reinforcement bar in design was 40mm. Standard deviation of cover thickness was 10mm from in situ investigation. The equivalent diffusion coefficient was obtained from results of testing the concrete core sampled at site in 2000. The mean value with 2.38×10^{-8} cm^2/sec and the standard deviation with 9.50×10^{-9} cm^2/sec were obtained from the survey data. The shape of distribution can be approximated as log-normal distribution. Two models were chosen as the model of equivalent diffusion coefficient as shown in Figure 8. One is the flat model against the elapsed year (Case1). Another is the decreasing model with elapsed year (Case2).

Simulation Method of Diffusion Coefficient

Figure 9. shows the results of computation in comparison with the inspection data. Red round marks indicate the results of repairing rate computed by Case 2. Green round marks indicate the results of repairing rate computed by Case 1. Blue marks indicate the results obtained in situ. The inspection data can meet among both results in computation. The results in computation were found to coincide with the results of inspection date by considering uncertainty of actual environments.

Photo 1. Target Structure

Table 1. Conditions in Computation

Item	Values	
	Mean	S.D.
Constant value per time of chloride ion density penetrates into concrete W (wt%/cm²/sec)	2.0×10^{-9}	
Initial Chloride Ion Density C'(kg/m³)	0.019	
Equivalent Diffusion Coefficient at 5 year Dc_5 (cm²/sec)	2.38×10^{-8}	9.51×10^{-9}
Equivalent Diffusion Coefficient oi repairing material at 5 year Dc_5 (cm²/sec)	2.38×10^{-9}	9.51×10^{-10}
Thickness of Cover X_t (cm)	4.0	1.0
Distance between each reinforcement X_d (cm)	15.0	1.0
Compressive concrete strength σ_{cr} (N/mm²)	39.7	
Mean Value of Temperature per year T_0 (°)	18.8	

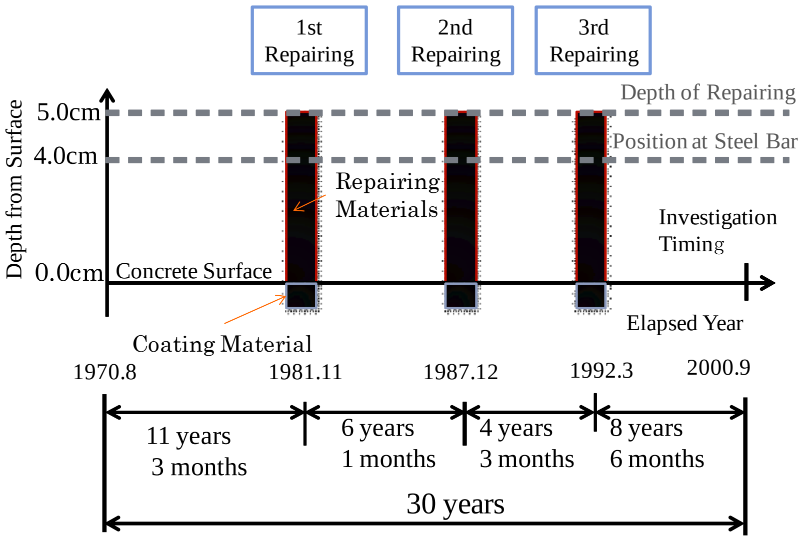

Figure 7. Diagram of Repairing Process

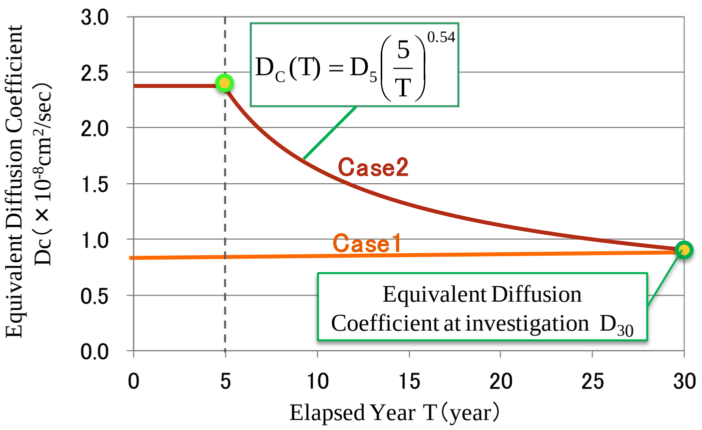

Figure 8. Model of Equivalent Diffusion Coefficient

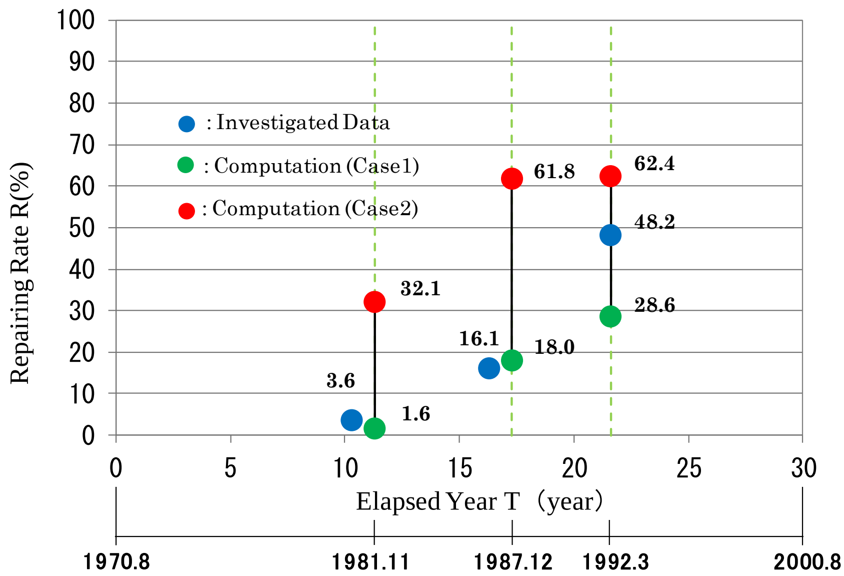

Figure 9. Comparison of Computation and Investigated Data

CONCLUTION

Based on a reliability approach, a deterioration process model of concrete that has received chloride induced damage was proposed and the results in computation and inspection data were compared.
The following is a summary of the findings:

(1) A probabilistic model was developed to represent the deterioration of concrete structures using a deterioration process model that includes an incubation period, a developed period and an accelerative period with consideration of repeated repairing.
(2) The distribution models of two stochastic parameters, equivalent diffusion coefficient and error of cover thickness, were obtained from existing studies. Two models as equivalent diffusion coefficients were proposed in computation.
(3) The numerical method proposed in this paper was carried out using a 30-year-old target slab of concrete. The results of the repairing rate in computation coincided with the results of the inspection date considering the uncertainty of actual environments.

REFFRENCES

Katawaki K., Moriya S. and Minosaku K.1985. Character and method of measurement of saline air coming from the sea, *Journal of Prestressed Concrete*, , Vol.27, No.1, Japan (in Japanese).
West R. E. and Hime, W. G. (1985). Chloride profiles in salty concrete, *Material Performance*.
Seki H. (1981). Deterioration of reinforced concrete wharfs, *Concrete International*, Vol.3, No.3.
Bazant Z. P. (1979). Physical model for steel corrosion in concrete sea structures -Theory-, *Journal of the Structural Division* Vol.105, No.ST6.
Bazant Z. P. (1979). Physical model for steel corrosion in concrete sea structures -Application-, *Journal of the Structural Division* Vol.106, No.ST6.

Brown R. D. (1982). Design prediction of the life for reinforced concrete in marine and other chloride environments, Durability of Building, *Elsevier Scientific Publishing Company*.

Brown R. D., Geoghegam M. P. and Baker A. F. (1983). Analysis of structural condition from durability results, Corrosion of Reinforced in Concrete Construction, *Ellis Horwood Limited*.

Manabu Matsushima, Kousaku Matsuda, Masaru Yokota (2012). Estimation on Deterioration Process of Concrete Members Suffering Chloride Induced Damage Based on a Stochastic Approach, *Concrete in the Low Carbon Era,* pp.1368-1379, Dundee, UK.