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Optimum Thickness of Concrete Cover of R.C. Structures Based on Reliability Theory

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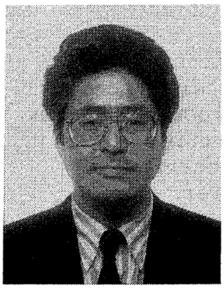
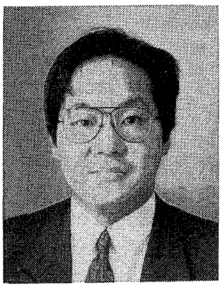
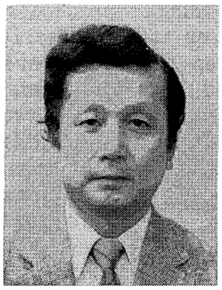


Manabu Matsushima　Tomoaki Tsutsumi　Hiroshi Seki　Kunihito Matsui

If the chloride concentration C_F at the surface of a concrete structure is known, the chloride concentration around reinforcements at year t can be estimated and the initiation time of reinforcement corrosion obtained from the concentration and some associated critical value. The accumulating corrosion products then causes the volume of reinforcement to expand, leading to cracking of the concrete surface. Provided that the structure reaches its critical state when surface cracks appear, the time t_L is obtained from when the cracks appear.

Taking the parameters associated with deterioration to be random variables, the occurrence probability of deterioration is evaluated. Then the optimum concrete cover thickness is obtained based on the concept of least expected cost.

keyword: *concrete cover, deterioration model, least expected cost, reliability theory*

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1. INTRODUCTION

Numerous reports on the deterioration of concrete structures have been published in recent years. Many of them are concerned with the corrosion of reinforcement due to chlorides, which results from inadequate removal of salt from sea sand or the presence of salts from sea water in a marine environment.

Various characteristics of concrete structures gradually fade over the years. The rate of loss depends on factors associated with design, execution, maintenance, and so forth. In taking measures to avert early deterioration, the need for proper durability design has been often stated. However, it has not been realized as yet.

Bazant [1],[2] developed a physical model for the corrosion of steels in concrete and proposed a method of estimating a service life of concrete structures. Browne et al[3],[4] presented a method of evaluating the integrity of structures and estimating their service lives based on a consideration of chloride penetration into the concrete cover as obtained from nondestructive testing data. However, certain problems arise in applying these methods existing structures, preventing a reliable predictions of service life. Although Reference[5] also proposes an evaluation method for durability based on a physical model which considers the initiation of reinforcement corrosion as the critical state, it lacks predictive accuracy. The reason for this may lie in errors due to uncertainties in the given data and/or in the modelling of chloride penetration.

Concrete cover is an important factor in ensuring the durability of concrete structures in the face of chlorides attack. The design thickness of concrete cover specified in the JSCE Standard Design Code for RC structures [6],[7] varies depending on the type of member and the environment surrounding them. However, the specification is not necessarily defined according to service life requirements and corrosion deterioration.

Provided that the chloride concentration C_F is known at the concrete surface, the chloride concentration around the reinforcement at year t can be estimated, and the time at which the estimated concentration exceeds some critical concentration is the corrosion initiation time t_{cr}. After corrosion starts, corrosion products are produced around the reinforcement increasing its volume, and resulting in cracking at the concrete surface. When cracking appears at the concrete surface, it is assumed that the structure has reached its critical state. The initiation time of corrosion cracking is denoted by t_L. Since the parameters associated with deterioration, in general, tend to vary widely, it is unreasonable to treat them deterministically. Therefore, considering them as random variables, the probability of corrosion cracking within the service life T_d is obtained and, based on the concept of the least expected cost, the optimum design cover thickness is found[8].

2. CHLORIDE-INDUCED DAMAGE MODEL

2.1 Deterioration Process

The model of deterioration due to chloride damage is schematically presented in Fig.1. Steel reinforcement is normally considered to be protected by a layer of concrete. Due to the permeability of the concrete, chloride ions penetrate through it and accumulate around the reinforcement surface. Then the passive film preventing corrosion breaks down. Corrosion of the reinforcement therefore starts, causing a volumetric expansion due to corrosion products which result in cracking of the concrete. The deterioration process is regarded as consisting of two phases; one is the period t_{cr} until depassivation of the reinforcement occurs after completion of the structure, and the other is the period t_r commencing from the moment of depassivation

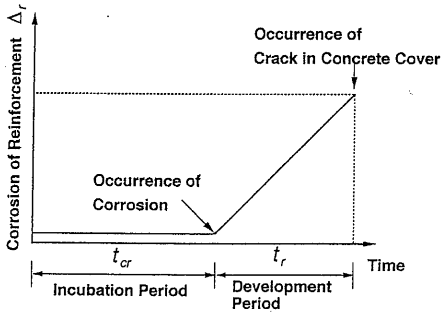
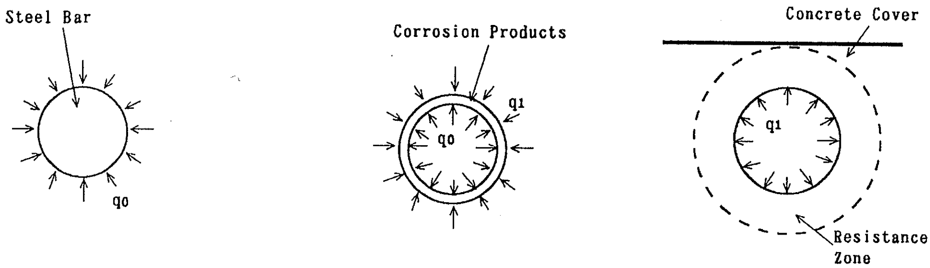


Fig.1 Conceptual Idea for Deterioration



(a) Pressure occurring Steel Bar
(b) Pressure occurring Corrosion Products
(c) Pressure occurring Concrete Cover

Fig.2 Assumption of Cracking Equation

and including the development of corrosion at a perceptible rate until the limit state is attained when cracking appears in the concrete. The former is called the incubation period and the latter the development period. Hence, the total passage of time t_L is defined as,

$$t_L = t_r + t_{cr} \tag{1}$$

When the time t_L is greater than a design service life, the structure reaches the critical state which is defined as the state of deterioration in this study

2.2 Incubation Period

It has been noted that chloride ions penetrate deepest into concrete, although sea water contains various ions aside from chlorides[9]. Deterioration of RC members in a saline environment is considered to begin with the breakdown of the passive film covering the reinforcement by the chloride ions. Chloride ions which penetrate from the concrete surface accumulate within the concrete in various forms, and some reach the reinforcement surface [10],[11]. Chloride transport within the protective layer of concrete involves a complicated mechanism and is not clearly identified so far. Thus, disregarding all the complexities, the chloride transport process is assumed to be represented by a one-dimensional diffusion equation,

$$\frac{\partial C}{\partial t} = D_c \cdot \frac{\partial^2 C}{\partial X^2} \tag{2}$$

Fick[12],[13] gives the solution of Eq.(2) as,

$$C(X_t, t) = C_0\left(1.0 - \mathrm{erf}\left(\frac{X_t}{2\sqrt{D_c \cdot t}}\right)\right) \tag{3}$$

where $C(X_t,t)$ denotes the chloride concentration at depth X_t, D_c is an equivalent diffusion coefficient, $\mathrm{erf}(\bullet)$ is the error function, C_0 is the chloride ion density at the concrete surface, and X_t is the thickness of the covering concrete. The chloride concentration at the surface of the reinforcement can be computed from Eq.(3).
Assuming that corrosion of the reinforcement begins when the chloride concentration exceeds some critical value following breakdown of the passive film, the following performance equation may be established:

$$J_{cr}(X_t, t) = C_{cr} - C(X_t,t) - C_I \tag{4}$$

in which C_{cr} refers to the critical chloride concentration and C_I is the initial chloride ion density inside the concrete. It implies that corrosion commences at the moment when the performance function $J_{cr}(t)$ becomes zero. The period from the beginning of construction to the time $J_{cr}(t)$=0 is denoted by t_{cr}.
The following parameters are chosen as random variables during the incubation period:

(1) Thickness of the concrete cover (X_t)

(2) Equivalent diffusion coefficient (D_c)

(3) Critical chloride ion density (C_{cr})

Hence the distribution of t_{cr} depends on these three parameters.

The chloride ion density at the concrete surface is greatly affected by the environment and can be expected to vary more widely than the other factors. However, it is treated as deterministic in this paper to avoid the complexities of environmental factors possess.

2.3 Development Period

Reinforcement corrosion proceeds with the diffusion of oxygen, and the expansive pressure due to corrosion products causes the concrete to crack. Figures.2(a),(b), and (c) illustrate the mechanical model. The corrosion products induce pressure not only on the uncorroded reinforcement but on the concrete cover, too. This pressure q1 on the concrete is assumed to be the causes of cracking in the concrete.
A thick cylindrical model is used to simulate the cracking of concrete due to expansive pressure. The average tensile stress over the thickness of the cover is used to judge the occurrence of cracking. The average stress may be written as

$$f_t = \frac{1.0}{\alpha_0(1.0-k)} \cdot q_1 \tag{5}$$

where $k = (2D + \phi)/\phi$, D is the thickness of concrete cover, ϕ is the diameter of the reinforcement, and α_0 is a modification factor.
Assuming the concrete cover behaves like a thick cylindrical shell subjected to a uniform inner pressure q_1, the radial displacement of the concrete is expressed as,

$$u_c = \frac{(1+\upsilon_c)\left\{(1-2\upsilon_c)+k_0^2\right\}\phi}{2E_c(k_0^2-1)} q_1 \tag{6}$$

Provided that the corrosion products accumulate uniformly around the reinforcement surface, which has a reduced diameter from its original ϕ, the reduced diameter and the outer diameter of the corrosion products may be written as,

$$\phi_1^2 = \phi^2 - \frac{4A_w}{\pi} \ , \ \ \phi_2^2 = \phi^2 + \frac{4(n-1)\cdot A_w}{\pi} \tag{7}$$

in which n is the volumetric expansion ratio, thought to be from 2.0 to 3.0 depending on the kind of corrosion. From Ref. [16], $n = 2.5$ is adapted in this study. A_w is the reduced cross sectional area of the reinforcement, and may be expressed as,

$$A_w = \frac{\Delta_r}{100} \times A_r \times (t - t_{cr}) \tag{8}$$

where, $t - t_{cr} > 0$.

Δ_r is the rate of corrosion (%/yr), assumed to be proportional to the cross-sectional area.

The corrosion products form a cylinder with inner diameter ϕ_1 and outer diameter ϕ_2. If inner pressure q_0 and outer pressure q_1 are applied to the cylinder, the tangential displacements u_{r0} and u_{r1} at the inner and outer surfaces may be written as,

$$u_{r0} = \frac{(1+\upsilon_r)\phi_1}{2E_r(K_1^2 - 1)}\left\{(1-2\upsilon_r)(q_0 - q_1 \cdot K_1^2) + (q_0 - q_1)K_1^2\right\} \tag{9}$$

$$u_{r1} = \frac{(1+\upsilon_r)\phi_1}{2E_r(K_1^2 - 1)}\left\{(1-2\upsilon_r)(q_0 - q_1 \cdot K_1^2) + (q_0 - q_1)\right\} \tag{10}$$

in which $K_1 = \phi_2 / \phi_1$, υ_r is Poisson's ratio, and E_r is the modules of elasticity of the corrosion products. According to Ref. [17], E_r = 2.0x10^3 kg/cm^2 and υ_r = 1/6 are adopted.

The uncorroded portion of the reinforcement, with diameter ϕ_1, is subjected to extend pressure q_0. Thus, the tangential displacement at the outer surface is found from,

$$u_s = -\frac{(1-\upsilon_r)\phi_1}{2E_s} \cdot q_0 \tag{11}$$

where υ_r is Poisson's ratio (=1/6) and E_s (=2.1x10^{6}kgf/cm^2) is the modules of elasticity of the reinforcement.

From the equilibrium of forces at the interface between the reinforcement and the corrosion products, and that between the corrosion products and the concrete, Eqs.(12) are obtained.

$$u_{rs} = u_s \ , \quad u_c = \frac{\phi_2}{2} - \frac{\phi}{2} - u_{r1} \tag{12}$$

Here, q_0 and q_1 can be estimated from the quantity of corrosion products. When the tangential tensile stress due to the inner pressure caused by corrosion products exceeds the tensile strength of the concrete, cracking of the concrete occurs. Introducing a performance function as cracking of the concrete cover appears when $J_r(t)$ attains zero.

In this study the following parameters are considered as random variables;

(1) Corrosion rate of reinforcement (Δ_r)

(2) Modification factor (α_0)

(3) Tensile strength of concrete (σ_{tu})

Using the time t_L from Eq.(13) and a given design service life T_d, the following function is defined:

$$J_r(t) = \sigma_{tu} - f_t \tag{13}$$

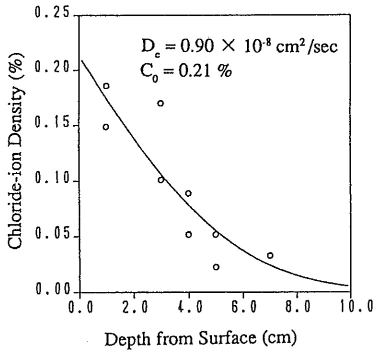


Fig.3 Actual Values for Chloride-Ion Density

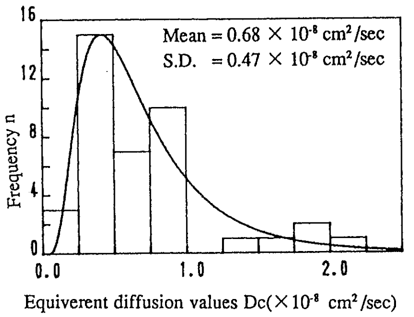


Fig.4 Distribution of Equivalent Diffusion Coefficient

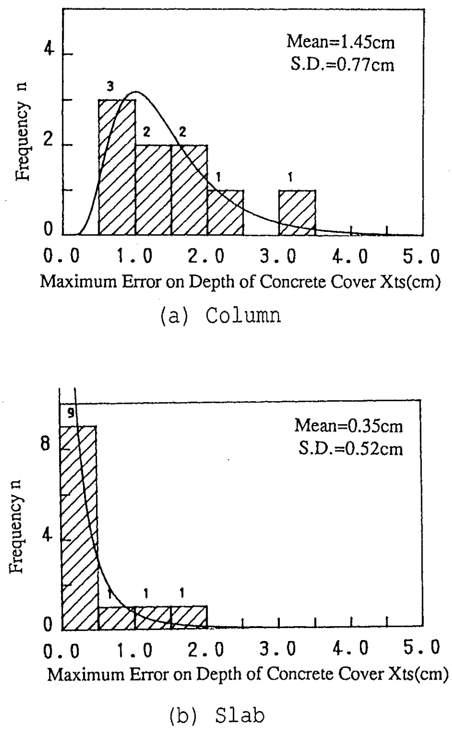


Fig.5 Maximum lack on Concrete Cover Thickness

When $J_L(t_L) < 0.0$, a structure reaches the limit state before the design service life is attained. The probability of $J_L(t_L) < 0.0$ is called the probability of deterioration in this study.

$$J_L(t_L) = T_d - t_L < 0.0 \tag{14}$$

3. PARAMETERS USED IN ANALYSIS

3.1 Diffusion Equation

A real problem may not be representable by a one-dimensional model such as Fick's diffusion but may require a complicated three-dimensional model. In order to determine whether one-dimensional model is able to represent the real phenomenon, the chloride ion density measured in some landing piers is compared with the results obtained using this simple model in Fig.3. Since the figure indicates a relatively good agreement, the simple model is used for analysis in this study. The diffusion coefficient so obtained contains not only the effects of reducing the three-dimensional problem to one dimension, but also simplification errors introduced by reducing the complicated transport process of chloride ions. This is the reason for calling the coefficient the 'equivalent diffusion coefficient'.

In order to determine the equivalent coefficient, an investigation is carried out on various landing piers around Tokyo Bay to find the chloride ion density. Then the equivalent diffusion coefficient is computed using the diffusion model. The frequency distribution of the coefficient is plotted in Fig.4., which may be expressed by a log-normal distribution with mean 0.68×10^{-8} cm^2/sec and standard deviation 0.47×10^{-8} cm^2/

sec. Thus the coefficient of variation is obtained as δ=0.69, which implies that the variation is large. It should be noted, however, that all sampling data are taken from the so called 'splash zone' where concrete structures are more likely to suffer deterioration.

3.2 Thickness of Cover Concrete

The use of dense, homogeneous concrete in a thick layer as the cover concrete is the basis for ensuring durability and reduces the rate of reinforcement corrosion. Cover concrete thickness varies depending on how carefully the construction work is managed. According to reference [18], the number of spacers used is clearly related to problems with cover thickness; the necessary thickness is ensured only if two or more spacers are used per square meter. Based on this report, a histogram of maximum deficiency in thickness is presented in Fig.5(a) for a column and Fig.5(b) for a slab. The actual thickness may be defined as,

$$X_t = X_{td} - X_{ts} \tag{15}$$

in which X_{td} is the design thickness and X_{ts} is the maximum deficiency in the thickness of the cover concrete. From Fig.5, the deficiency may be represented by a log-normal distribution with mean 1.45cm and standard deviation 0.77cm for columns and with mean 0.35cm and standard deviation 0.52cm for slabs.

3.3 Initial Chloride Ion Density

The permissible level of chlorides in a concrete mix and in the finished concrete has been a matter of discussion for years. Since chlorides can be present in a natural form in all constituents of the concrete mix, it is unrealistic to exclude their presence altogether.

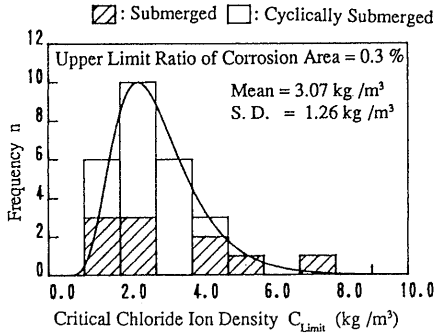
The initial density of chloride ions largely depends on the aggregates used. Usually aggregates contain no chlorides, but there are some exceptions; e.g., sand dredged from the seabed or taken from beaches.

This parameter is not treated as a random variable; rather the value 0.30 kg/m^3 specified in the design code is employed in this study.

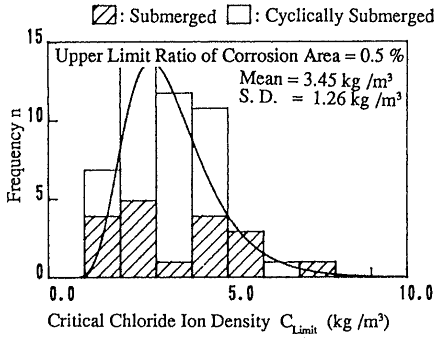
3.4 Critical Density of Chloride Ions

Tsutsumi et al. conducted a series of experiments to find the critical density of chloride ions at which the protective film on the reinforcement surface breaks down. They studied specimens with three different water-cement (W/C) ratios: 40%, 55%, and 70%. One group was submerged for the duration of the experiment and another underwent a repetition of submerging and drying. From the experiments, the relationship between the ratio of corroded area (= corroded area/total surface area of reinforcement) and chloride ion density around the reinforcement was found. The real need, though, is the chloride ion density at the moment of depassivation, yet it is impossible to measure it.

It is assumed in this paper that corrosion commences when the ratio of corroded area attains a given value. The problem becomes one of finding this limiting ratio. Figures. 6(a) and (b) show the distribution of chloride ion density in the neighborhood of the reinforcement when the ratio is 0.3% or less and when it is 0.5% or less respectively. The figures show the distributions not only for submerged specimens but for specimens undergoing the alternate cycles of submerging and drying. Both distributions are very similar. To examine the effect of the limiting ratio, mean, standard deviation, and minimum chloride density corresponding to the ratios 0.1% to 0.5% with an increment of 0.1% are plotted in Fig.7; the number by the mean indicates the number of samples. The mean shows a slight increase as the ratio increases, but the standard deviation and the minimum remains unchanged except for the ratio 0.1%, for which only



(a) Ratio of Corroded Area
(Less Than 0.3%)



(b) Ratio of Corroded Area
(Less than 0.5%)

Fig.6 Distribution of Critical Chloride-Ion Density

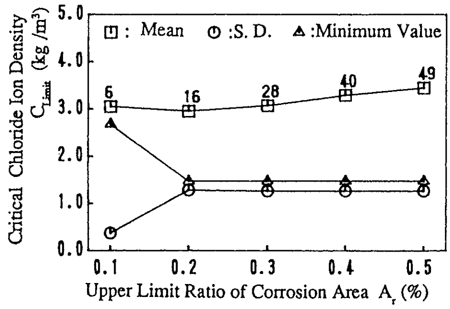


Fig.7 Relationship between Critical Chloride-Ion density and Ratio of Corroded Area

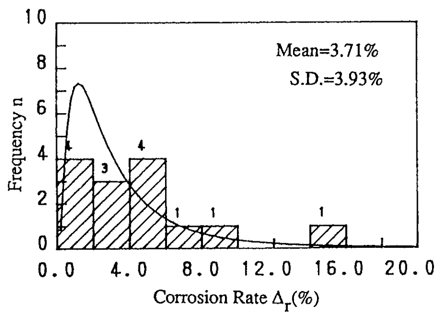


Fig.8 Distribution of Corrosion ratio of reinforcement (t=33 year)

a small number of samples is available. From this figure, the density corresponding to 0.3% is chosen as the critical chloride ion density. The distribution of density is modeled by a log-normal distribution with mean 3.07kg/m^3 and standard deviation 1.26kg/m^3.

3.5 Corrosion Rate of Reinforcement

The corrosion rate of the reinforcement is estimated from the reduction cross-sectional area of reinforcement with age of the concrete piers investigated in Tokyo Bay. Figure 8 is a histogram of corrosion rate obtained from a pier 33 years old. The samples were taken from an area where the main reinforcement had a protective layer thickness of 7cm~13cm. The reduction ratio F_r(%) due to corrosion is defined as

$$F_r = \frac{A_F}{A_s} \times 100 \tag{16}$$

in which A_F is the corroded cross-sectional area of reinforcement and A_s is the original cross- sectional area.

The mean and standard deviation of corrosion rate are also given in Fig.8, along with the log-normal distribution of the same age.

In order to find the corrosion rate of the reinforcement, samples from piers of approximately same age are treated as a set. The mean ratio of cross-sectional area reduction is plotted in Fig.9, in which the number indicates the number of samples in the set. For the purpose of examining the difference in thickness of cover concrete,

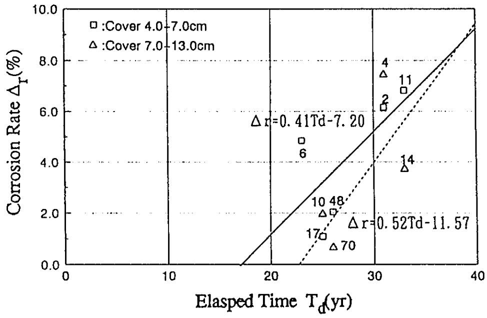


Fig.9 Average Corrosion Rate of Reinforcement

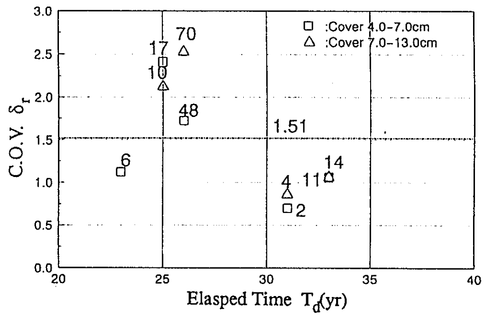


Fig.10 Variation of Corrosion Rate of Reinforcement

the samples are roughly classified into two groups: one from 4cm to 7cm and the other from 7cm to 13cm.

As Fig.9 shows, the ratio of cross-sectional area reduction increases as time passes. The slope represents the corrosion rate. A regression line of the slope is also shown in the figure. From the regression analysis, the corrosion rate for the thickness of 4 ~ 7 cm is found to be 0.41%/yr and that for the thickness of 7 ~ 13 cm is 0.51%/yr. Thus, the corrosion rate is almost the same regardless of the thickness of the cover concrete. Although it has been accepted that the thickness of the cover affects the corrosion rate, no great distinction can be made due to differences in thickness in our sampled data. Our theoretical work also supports this observation. Hence, by taking the average of the two corrosion rates, a corrosion rate of 0.45% is adopted for the analysis.

The coefficient of variation (COV) within a group and the age it represents are plotted in Fig.10. The figure shows that COV varies from 0.7 to 2.52. A mean COV of δ_r = 1.51 will be used in analysis.

3.6 Modification Factor

Harada et al demonstrated that the theory of a thick cylindrical shell is applicable to the collapse of concrete due to an expansive agent. They observed that cracks propagate instantaneously from the inside to the surface of the concrete when $K_0 (= (2X_t + \phi) / \phi)$ is less than 5.0. From their results, the modification factor based on the average stress criterion takes a value from 0.5 to 0.8. Therefore, a rectangular distribution between 0.5 and 0.8 is assumed in this paper.

3.7 Tensile Strength of Concrete

The concrete used in marine structures is of compressive strength σ_{ck} from 350 to 450 kg/cm^2 with a cement content 350kg/m^3 and a water-cement ratio W/C=45%. The COV of compressive strength is thought to be about 10%. Assuming a compressive strength of 400kg/cm^2, the mean tensile strength and its corresponding standard deviation are 31.5kg/cm^2 and 3.15kg/cm^2.

4. OPTIMUM THICKNESS OF COVER CONCRETE

The distribution of deterioration is schematically shown in Fig.11 for the random variables described above.

The probability of deterioration is computed using a Monte Carlo simulation with 10,000 random numbers for each random variable. By considering two cases of chloride

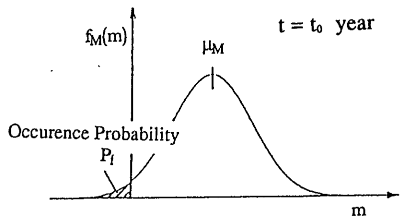
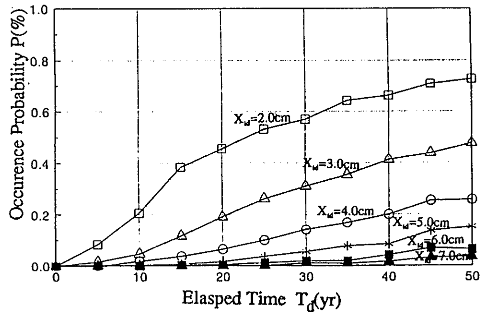


Fig.11 Relationship between Occurrence Probability and Performance Function



(a) C_F=5.0 kg/cm^2

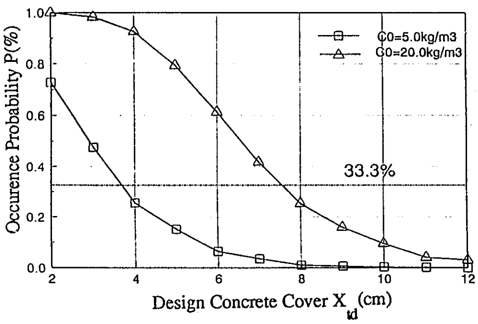
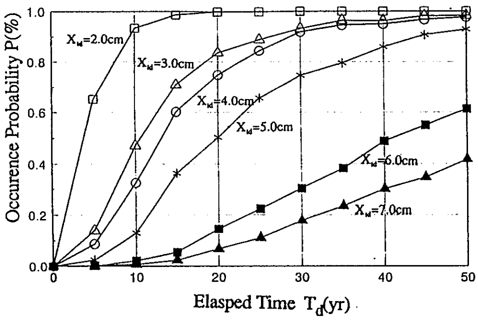


Fig.13 Design Thickness of Concrete Cover v.s. Occurrence Probability in Service Life 50 year



(b) C_F=20.0 kg/cm^2

Fig.12 Relationship between Elapsed Time and Occurrence Probability

ion density at the concrete surface C_F=5.0kg/m^3 and 20kg/m^3 and six different thicknesses of cover concrete, 2.0, 3.0, 4.0, 5.0, 6.0, and 7.0 cm, the probability is computed at different ages. The results are presented in Figs.12(a) and (b). These figures show that the probability of deterioration increases with ageing and that this trend is more conspicuous when the thickness is less.

The optimum thickness is determined based on the concept of least expected cost. Let X_t be the selected thickness of the cover concrete. Then let the expected cost of loss be $C_{f1}(X_{td})$ in the case that deterioration is incorrectly judged not to occur when in fact it does. Also let the expected cost be $C_{f2}(X_{td})$ when deterioration is judged to occur when it does not.

When $C_{f1}(X_{td})$ coincides with $C_{f2}(X_{td})$, X_{td} is the optimum thickness of the cover concrete X_{topt}. The mathematical formulation to find the optimum may be stated as follows:

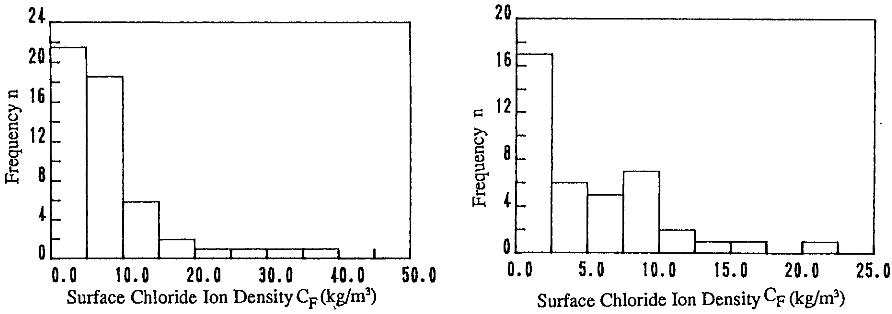
$$X_{topt} = X_{td} \text{ when } C_{f1}(X_{td}) = C_{f2}(X_{td}) \tag{17}$$

where

$$C_{f1}(X_{td}) = P_f(X_{td}) \times L_2$$

$$C_{f2}(X_{td}) = (1 - P_f(X_{td})) \times L_1$$

$P_f(X_{td})$ is the probability of deterioration when the cover thickness is X_{td}. L_1 is the cost of incorrectly predicting deterioration when it does not in fact occur and L_2 is the cost of not predicting deterioration when it does occurs. L_1 is a wrong judgement



(a) Japan Sea Coast (b) Tokyo Bay

Fig.14 Chloride-Ion density of Concrete Surface

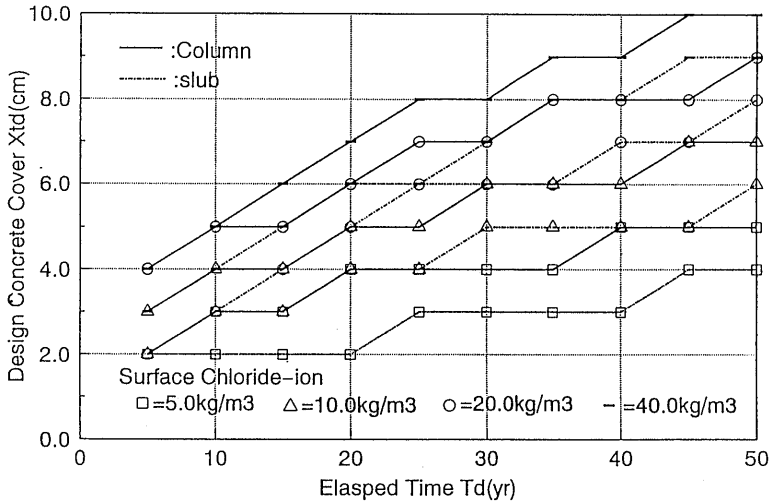


Fig.15 Relationship Between design Concrete Cover and Elapsed Time

on the danger side while L_2 is wrong but on the safe side. In general, if a wrong judgement is made on the safe side, the expected cost of the loss is smaller. Thus, if $L_2 = 1/2 \times L_1$ is selected, $P_f(X_{td})$ = 33.3% will be obtained.

The relationship between design cover thickness and the probability of deterioration with a service life 50 years and C_F = 5.0 and 20.0 kg/m^3 is shown in Fig.13. The probability of deterioration decreases rapidly as the concrete cover increases at the beginning. However, once the concrete cover goes beyond a certain value, the probability decreases slowly. By solving Eq.(17), the optimum concrete cover is found to be X_{topt} = 4cm with C_F = 5.0kg/m^3 and X_{topt} = 8cm with C_F = 20.0 kg/m^3.

5. DESIGN THICKNESS OF CONCRETE COVER

We seek a design thickness of the concrete cover for a given chloride ion density at the concrete surface and service life.
Figures 14(a) and (b) are histograms of chloride ion density measured in piers in Tokyo Bay and along the Japan Sea Coast [25]. These surface chloride ion densities are estimated values of C_F in Fick's expression (Eq.(3)) from sampled chlorides at various depths of concrete. The values of C_F tend to be larger on the Japan Sea coast due to

the severe environment. However, some are as small as those in Tokyo Bay. Chloride concentration seems to vary depending on wind direction and where in a structure the samples are extracted from. The figures show that the maximum density of chloride ions on the Japan Sea coast is about 40.0 kg/m^3, while in Tokyo Bay it is 20.0 kg/m^3. From these results, we selected four different densities: C_F = 5.0, 10.0, 20.0, and 40.0 kg/m^3.

It has been assumed that a structure suffers more severe damage in the splash zone. Hence, this study focuses on chloride concentrations and deteriorations particularly in that zone. Service lives of 5 years to 50 years are considered in 5-year increments. The weighing for loss cost is assumed to be $L_2 = 1/2 \times L_1$, although there is an alternative method of selecting weighing considering the importance of the target structure[26].

Since, from the point of view of engineering practice, it is more suitable to make the design thickness of concrete cover a discrete number, the smallest integer which satisfies Eq.(17) is chosen as the design cover. Figure 15 shows a service life and the corresponding design cover thickness. This figure demonstrates that the concrete cover for columns is 1 cm greater than that for slabs in most cases. It is also clearly seen from the figure that the design thickness of concrete cover increases when the surface chloride concentration is higher.

6. SUMMARY AND CONCLUSIONS

This paper demonstrates a method of finding the design thickness of concrete cover based on the concept of least expected cost, assuming that the surface chloride concentration is given and that a concrete structure reaches its critical state when corrosion expansion causes cracking.

The following is a summary of the findings:

(1) A probabilistic approach is introduced into a deterioration model for the penetration of chloride ions. Assuming that corrosion of the reinforcement commences when the chloride concentration in Fick's equation exceeds a given critical value, a mathematical model is derived, in which the following parameters are regarded as random variables: i) equivalent coefficient of diffusion; ii) thickness of concrete cover; and iii) critical density of chloride ions. A thick cylindrical model is employed to examine the onset of cracking. The mathematical model contains three random variables which are i) the corrosion rate of the reinforcement; ii) a modification factor α_0 used in the model; and iii) the tensile strength of concrete σ_{tu}.

(2) The proposed method of finding the design thickness of concrete cover employs the deterioration model and the concept of least expected cost.
If the service life of pier it is to be 50 years, is found that the design cover thickness should be 4.0cm when C_F = 5.0 kg/m^3 and 8.0 cm when C_F = 20kg/m^3

(3) The design thickness of concrete cover is given in the form of a diagram from which the necessary design cover thickness can be selected once surface chloride density and service life are known.

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