Probability-Based Maintenance Planning for RC Structures Attacked by Chloride

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Abstract

There are many uncertainties relating to the deterioration of reinforced concrete (RC) structure, and as a result, the actual degree of deterioration of structures is not uniform. Currently, various safety factors are considered to cover those uncertainties. This paper proposes a new method for maintenance planning of RC structures degraded by chloride attack based on probability theory. Prediction models of deterioration caused by chloride attack both before and after repair are discussed. The effects of crack and macrocell corrosion were considered to accelerate the deterioration in the prediction model. Surface coating, patching repair, cathodic protection, and combinations thereof were considered as repairing options. Moreover, the effects of partial and full repair were also considered in the deterioration of the repair system. The actual variation of structural properties and the environmental conditions obtained from the inspection program were used directly as input for prediction. By using Monte Carlo simulation, variations of deterioration degree can be predicted. Based on the prediction result, repair and failure costs are determined and used to design the maintenance planning program. Finally, applications of maintenance planning of actual case studies are given as an example.

1. Introduction

Reinforced concrete (RC) structures deteriorate over time when they are subjected to an aggressive environment. To maintain structural safety and serviceability, almost all structures need an appropriate maintenance program to be applied during their service life. In 2002, Japan allocated approximately 13.5 trillion yen, which is 21.5% of the total construction budget, for the maintenance of the existing infrastructure. The ratio of maintenance budget to the overall construction budget is expected to continuously increase in the future because of the increasing number of aging structures.

Corrosion of reinforcing steel due to chloride attack is one of the main mechanisms degrading RC structures. The expansion of corrosion product causes cracking and spalling of covering concrete. Therefore, maintenance is required to prevent corrosion and to repair any damage that has been observed. In general, the mechanisms of chloride induced corrosion can be considered as consisting of 4 stages (JSCE 2005a). The initiation stage is dominated by the diffusion of chloride ions from the environment toward the surface of the reinforcing steel. The propagation stage starts when the chloride content at the steel surface exceeds the threshold level at which corrosion will initiate. Internal pressure is gradually generated by the corrosion product and finally corrosion cracks are generated after internal pressure exceeds the tensile strength of concrete. The third stage is the acceleration stage, during which the presence of corrosion cracks accelerates the deterioration rate. The final stage is the deterioration stage, during which the load bearing capacity of the structure is significantly affected. Various prediction models have been proposed for each stage of deterioration.

JSCE (2005b) regulates durability design to ensure the durability of structures during their design service life based on performance based design. In reality, there are many uncertainties, such as material properties, structural dimensions, environmental conditions, and the prediction model. As a result, the degree of deterioration of an actual RC structure is not uniform (Komure et al. 2002). Traditionally, safety factors are incorporated in design to cover these uncertainties, leading to high cost and overdesign of the structure.

During the service life of a structure, maintenance is required to ensure that acceptable structural performance is maintained. Various bridge management systems (BMS) have been developed in many countries (Hawk and Small 1998 and Thompson et al. 1998). BMS includes periodic assessment of structural conditions, updating deterioration prediction, and making maintenance decisions. Most of the current BMS are based on a deterministic prediction model that takes into consideration safety factors or a Markov process that uses a database of the deterioration progress. As a result, prediction of variation in the degree of deterioration of a structure as observed in the actual structure accurately based on fixed parameter values in a deterministic model is not
possible. In this study, a method to predict variation in the degree of deterioration of an RC structure degraded by salt attack both before and after repair is proposed based on a deterministic model combined with consideration of actual variations in structural properties and environmental conditions based on actual inspection results. Monte Carlo simulation is used to consider the uncertainties. Finally, the predicted degree of deterioration is used to calculate repair and user costs for maintenance planning.

2. Deterioration prediction model

As explained, different deterioration prediction models have to be considered for different stages of deterioration, and re-deterioration of repairs that have been made. This section describes the deterioration prediction models used in this study.

2.1 Chloride diffusion prediction model

The penetration of chloride ions into concrete can be considered as a diffusion process and therefore can be described by Fick’s second law of diffusion. The most famous solution used in the analysis of chloride diffusion in concrete is taking the surface chloride concentration as a boundary condition, as shown in Eq. 1.

\[
C(x,t) = C_0 \left[ 1 - \text{erf} \left( \frac{x}{2\sqrt{D \cdot t}} \right) \right]
\]

where \(C(x,t)\) is the chloride ion concentration at depth \(x\) (cm) after time \(t\) (years) (% by weight of concrete), \(C_0\) is the surface chloride concentration (% by weight of concrete), \(D\) is the apparent diffusion coefficient of chloride ions (cm²/year), and \(\text{erf}\) is the error function. JSCE (2005b) recommends values for the surface chloride concentration and chloride diffusion coefficient. In reality, the value of the apparent diffusion coefficient of chloride ions is time-dependent. Therefore, in this study, the variation in the surface chloride concentration and the apparent diffusion coefficient of chloride ions are obtained directly from the inspection result of the chloride concentration profile. As a result, the time-dependent property of the apparent diffusion coefficient of chloride ions is considered as a long-term average value from the inspection results as well as the effect of convection on the penetration of chloride ions.

After the chloride concentration at the surface of the reinforcing steel reaches the threshold value, corrosion is initiated. The threshold chloride concentration depends on various factors such as cement content, type of cement, and environmental conditions. (Glass and Buenfeld 1997, Taylor et al. 1999, and Whiting et al. 2002). In this study, the threshold chloride concentration is assumed to vary as a lognormal distribution with a mean of 0.05% by weight of concrete and coefficient of variation 10% (Enright and Frangopol 1999).

2.2 Corrosion induced cracking

After the initiation of corrosion, corrosion product is formed and the internal pressure gradually increases according to the rate of corrosion. As the internal pressure reaches the tensile strength of concrete, cracks are generated. Li et al. (2006) proposed a model to predict the width of corrosion cracks, as shown in Eq. 2, based on a mechanical model and the thick wall cylinder model proposed by Bazant (1979).

\[
w_c = \frac{4\pi d_s(t)}{(1 - v_s) + (1 + v_s)} + \frac{2\pi d_s(t)^2}{E_{ef}} \]

where \(w_c\) is the corrosion crack width (mm), \(v_s\) is the Poisson’s ratio of concrete, \(a\) is the stiffness reduction factor, which depends on the average tangential strain over the cracked concrete surface and can be determined from Li et al. (2006), \(f_t\) is the tensile strength of concrete (MPa), which can be estimated from the rebound hammer test result, \(E_{ef}\) is effective modulus of concrete, equals to \(E/(1+\nu_e)\) (MPa), \(E\) is the elastic modulus of concrete (MPa), \(\nu_e\) is the concrete creep coefficient, \(a\) is equal to \((D+2d_b)/2\), \(b\) is equal to \(x + (D+2d_b)/2\), and \(D\) is the steel diameter (mm), \(d_b\) is the thickness of the pore band of the steel and concrete interface (mm), which is the interfacial layer between the steel surface and the concrete. The pore band has to be completely filled before pressure can be generated from the rust. Its thickness is assumed as Liu and Weyers (1998), and \(d_s(t)\) can be determined from Eq. 3.

\[
d_s(t) = \frac{W_{rust}(t)}{\pi(D + 2d_b)} = \frac{1}{\rho_{rust}} \frac{1}{\rho_{rust}} \frac{\alpha_{rust}}{\rho_u}
\]

where \(\rho_{rust}\) is the density of corrosion product (kg/m³), \(\rho_u\) is the density of steel (kg/m³), and \(\alpha_{rust}\) is a coefficient related to the types of rust products and has a value in the range of 0.523 to 0.622, based on Liu and Weyers (1998). Also, Bhargava et al. (2005) reported values of \(\alpha_{rust}\) of 0.777, 0.724, 0.699, 0.622, and 0.523 for FeO, Fe₂O₃, Fe₃O₄, Fe(OH)₂, and Fe(OH)₃, respectively. In this study, \(\alpha_{rust}\) is set to be constant and equal to 0.57 as the average value of different corrosion products. \(W_{rust}(t)\) is a mass of rust product (mg/mm) and can be determined from Eq. 4 (Liu and Weyers 1998).

\[
W_{rust}(t) = \int_{t_i}^{t_f} \frac{0.105d_{corr}(t)}{\alpha_{rust}} dt
\]

where \(i_{corr}\) is the annual mean corrosion rate including microcell and macrocell corrosion (μA/cm²). When corrosion is initiated, the corrosion rate should be measured in order to predict the corrosion cracking time during the deterioration stage or loss of cross section area of the reinforcing steel during the deterioration and acceleration stages by using linear polarization resistance. For more accurate prediction of structure deterioration,
the corrosion rate should be obtained from a series of measurements or through continuous monitoring as the corrosion rate is affected by the conditions of the concrete structure at the time of measurement, such as moisture and temperature.

Actual variation in structural performance such as concrete compressive strength, steel covering depth, steel diameter, as well as the corrosion rate obtained from inspection, can be directly considered in the prediction.

The presence of corrosion cracks also accelerates the ingress of chloride ions as discussed by Maeda et al. (2002), Kato et al. (2005) and Islam (2006). By subtracting chloride diffusion in non-cracked concrete from cracked concrete, Maeda et al. (2002) determined the chloride diffusion coefficient along cracks. Eq. 5 is proposed based on this study. Therefore, chloride diffusion in cracked concrete can be predicted as shown in Eq. 6. The initial chloride content can also be considered if chloride is present. However, cracks can be also initiated by other means such as shrinkage, loading, or temperature changes. Therefore, the prediction of various deterioration mechanisms is very important and should be conducted and combined with the current study in the future.

\[
D_{cr} = \begin{cases} 
72.4 \cdot w_c & ; w_c < 0.05 \text{ mm} \\
3 \times 10^4 \cdot w_c^{0.086} & ; 0.05 \text{ mm} \leq w_c \leq 0.15 \text{ mm} \\
3150 & ; w_c > 0.15 \text{ mm}
\end{cases}
\]  

(5)

where \(D_{cr}\) is the chloride diffusion coefficient along the crack (cm²/year). The effect of variation in predicted corrosion crack width can be seen in the variation of chloride diffusion along the crack.

\[
C(x,t) = C_0 \left[ 1 - \text{erf} \left( \frac{x}{2 \sqrt{D_s + D_{cr}} \cdot t} \right) \right]
\]  

(6)

2.3 Re-deterioration of repair system

A number of systems are available to repair an RC structure degraded by chloride induced corrosion. Based on the prediction of the deterioration of a concrete structure, different maintenance systems can be planned to ensure lifetime performance. Each repair system is associated with different costs and performance, and can be applied at different stages of the deterioration process with different effects.

Issues relating to the durability of concrete repair systems are discussed widely by Emberson and Mays (1990), Emmons et al. (1993), Morgan (1996), and Cusson and Mailvaganam (1996) as many repair systems show signs of re-deterioration or failure of the system in a short period after repair has been made. In the case of a structure degraded by chloride induced corrosion, surface coating, patching repair, and cathodic protection are the systems that are normally applied. Their deterioration prediction models are considered in this study.

2.3.1 Concrete surface coating

The main benefit of surface coating is that it suppresses the penetration of harmful ions. There are two major approaches to predict chloride diffusion of concrete with surface coating (JSCE 2005c). Surface coating can be considered as an artificial concrete covering depth. Due to the very low chloride diffusion coefficient of coating material, even 5mm of coating thickness is comparable to 20mm of normal concrete covering (JSCE 2005c). Another approach is to predict chloride diffusion through two layers of materials, i.e. coating material and concrete, as shown in Eq. 7 (JSCE 2005c). In this study, prediction of chloride diffusion of coated concrete leads to this equation.

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

(7)

where \(C_0(t)\) is the thickness of coating material (cm) and \(D_s\) is the chloride diffusion coefficient of the coating material (cm²/year).

A surface coating material is also gradually degraded by environmental attacks such as UV radiation. Uomoto et al. (2006) described the deterioration of many surface coating systems. Cracks, swelling and color changes in the surface coating material were observed following exposure of the specimens to a marine environment or urban outdoor environment. Due to the complex system and material of the surface coating, a satisfactory deterministic deterioration prediction model has not been determined yet. The review of a large body of literature by JSCE 2005c shows the effective protective period of surface coating to vary in the range of 8 to 26 years. This study evaluates the service life of surface coating based on the recommendation of JSCE 2005c that the effective thickness of surface coating decrease to 20% within 20 years after in-service and be ignored as insignificant after 20 years of service as shown in Eq. 8 (JSCE 2005c).

\[
C_s(t) = \begin{cases} 
C_{is} \left(2 - e^{\lambda \cdot t} \right) & ; t \leq 20 \\
0 & ; t > 20
\end{cases}
\]  

(8)

where \(C_{is}\) is the initial thickness of surface coating (cm) and \(\lambda\) is a constant equal to 0.029 according to the recommendation of JSCE (2005c).

The coating material can deteriorate owing to various reasons, both internal and external. Concrete cracks and ultraviolet are examples of internal and external causes of deterioration, respectively. Ultraviolet mainly decreases the thickness of the coating material. Cracks can be form due to cracking at the concrete surface, which may be caused by loading, corrosion, temperature changes, etc. Crack generation in the case of deterioration of the coating material is due to repeated loading such as cyclic temperature change or cyclic crack opening/closing due to service load. This differs from failure at ultimate strength and therefore is considered as fa-
tigue failure. Kato et al. (2005) reported resistance of the surface coating material against cracking. Many surface coating materials with different thicknesses were tested by fatigue loading to determine the crack width at the concrete surface that will generate cracks in the surface coating. Kato et al. concluded that crack formation in surface coating does not depend on the type of coating material but on the thickness of the coating material. The relation between coating thickness and maximum resistible crack width of surface coating is shown in Fig. 1 and Eq. 9.

$$W_{cslim} = 0.8683 \cdot C_t(t) + 1.7962$$  \hspace{1cm} (9)$$

where $W_{cslim}$ is the limit crack width at which the surface coating will be cracked (mm), as shown in Fig. 1, and $C_t(t)$ is the effective thickness of the coating material after $t$ years (mm). The variation of $W_{cslim}$ depends on the variation in the thickness of the coating material. In this study, the thickness of the coating material is assumed to be constant. Similarly, after a crack is formed in the surface coating, chloride diffusion along the crack also has to be considered, as shown in Eq. (6).

### 2.3.2 Patching repair

Patching repair is a method used to replace localized areas of concrete that shows signs of deterioration such as rust liquid, cracking, spalling, or delamination. The repair process removes loose concrete that has cracks, spalling, or delamination, cleans the surface of the steel reinforcement, and then replaces the defective concrete with patching materials. The patching materials normally used are Portland cement concrete, quick-set hydraulic mortar and concrete, and polymer mortar and concrete. Depending on the chloride diffusion coefficient of the patching material, chloride diffusion can be predicted with an equation similar to Eq. 6.

Many patch repairs and their surrounding areas exhibit new corrosion damage after a few months to a year (Qian et al., 2006; Uomoto et al., 2006). In patch repair systems, the patched area and the substrate areas provide the embedded steel bars with dissimilar electrochemical environments. Imbalance of electrochemical potential is caused by many factors including physical properties (density, porosity, and permeability), and chemical composition (chloride content and oxygen content). Nanayakkara and Kato (2007) also revealed that corrosion of reinforcing steel is mainly found in patch repaired area. This may be due to physical incompatibility such as permeability between the patching material and concrete substrate. Pruckner and Gjorv (2002) determined that the macrocell corrosion current can be simply determined by Ohm’s law as shown in Eq. 10.

$$I_{mac} = \frac{\Delta U}{R_g + R_{st} + R_A + R_C}$$  \hspace{1cm} (10)$$

where $I_{mac}$ is the macrocell corrosion current (A/cm²), $\Delta U$ is the corrosion cell voltage (V), $R_g$ is the concrete resistance ($\Omega$/cm²), $R_{st}$ is the reinforcing steel resistance ($\Omega$/cm²), $R_A$ is the anodic reaction resistance ($\Omega$/cm²), and $R_C$ is the cathodic reaction resistance ($\Omega$/cm²). Feliu and Gonzalez (1989) reported that the polarization resistance of steel in concrete without chloride (cathodic reaction) and with chloride (anodic reaction) is in the range of 10²-10⁶ $\Omega$/cm² and 10⁻²-10⁻⁴ $\Omega$/cm², respectively. Anodic reaction resistance and cathodic reaction resistance depend on the condition of the steel surface. For example, if the steel surface is no depassivated, the anodic resistance is infinite. If no oxygen is available, the cathodic resistance is infinite. In the deterioration stage, corrosion is ongoing. Therefore, the resistance of the anodic area (anodic resistance) is significantly lower than the resistance of the cathodic area (cathodic resistance) and concrete. Thus, anodic resistance can be neglected from Eq. 10 as well as the resistance of reinforcing steel, which is very low by nature. Concrete resistance can be measured directly as explained by Feliu and Gonzalez (1989). In this study, cathodic resistance is assumed to be 100,000 $\Omega$/cm² and concrete resistance to be 600 $\Omega$/cm² based on experimental results obtained by Feliu and Gonzalez (1989).

Corrosion cell voltage ($\Delta U$) is caused by the electrochemical potential difference between the existing concrete and repair patch (Gu et al., 1997) due to different chloride concentrations and permeability. The electrochemical potential at the reinforcing steel surface can be measured relatively by the half-cell potential method. Suzuki et al. (2007) measured the half-cell potential of similarly treated ordinary Portland cement concrete specimens cast with different chloride content. Figure 2 shows the half-cell potential results. An empirical equation is proposed to estimate the half-cell potential at different chloride concentrations in concrete, as shown in Eq. 11.

$$U_c = -953.1 \cdot C(x,t) - 7.9921$$  \hspace{1cm} (11)$$

where $U_c$ is the half-cell potential of reinforcing steel in OPC concrete (mV) and C(x,t) is the chloride ion concentration in concrete at the steel surface at time t (% by weight of concrete).
The half-cell potential of reinforcing steel when using polymer modified mortar as the patching material is different from that in OPC concrete. Specimens made from patching material as described by Nanayakkara (2006) were measured for their half-cell potential, and the results are shown in Fig. 3. Similarly, the equation to estimate the half-cell potential at different chloride concentrations is shown in Eq. 12. The half-cell potential depends largely on the type of material. The specific equation of each material should be obtained before considering macrocell corrosion and the applied method proposed in this study.

\[
U_p = -313.84 \cdot C(x,t) - 277.65
\]  

where \(U_p\) is the half-cell potential of reinforcing steel in patching material (mV), and \(C(x,t)\) is the chloride ion concentration in concrete at the steel surface at time \(t\) (% by weight of concrete). The comparison of the calculated macrocell corrosion current based on Eq. 10 and the measured corrosion current conducted by Nanayakkara (2006) is shown in Fig. 4 at different chloride concentrations between the existing concrete and the repaired section. Although a trend of increase in the macrocell corrosion current with increasing chloride concentration can be seen in both the measured and calculated results, differences between the measured and calculated results can be observed. The corrosion current is affected by many factors such as temperature, the moisture condition of specimens, etc. For more accurate prediction of the macrocell corrosion, the corrosion rate should be obtained from a series of measurements or through continuous monitoring as the corrosion rate is affected by the conditions of the concrete structure at the time of measurement, such as moisture and temperature. The macrocell corrosion calculation principle is applicable but Eqs. 11 and 12 should be modified in the future when more data on the long-term monitoring of macrocell corrosion becomes available.

### 2.3.3 Cathodic protection

Cathodic protection is one of the most common and effective methods for corrosion control of steel reinforced concrete. Cathodic protection controls the corrosion of steel in concrete by applying an external source of direct current to the surfaces of the embedded steel. Depending on the type of anode system, the service life of the systems is between 5 and 35 years, as shown in Table 1 (SHRP 1993a).

![Fig. 2 Half-cell potential at different chloride concentrations in OPC concrete specimens.](image)

![Fig. 3 Half-cell potential at different chloride concentrations in patching repair material.](image)

![Fig. 4 Comparison between experimental measured macrocell corrosion current and calculation result.](image)

<table>
<thead>
<tr>
<th>Anode system</th>
<th>Structure protected</th>
<th>Estimated service life, years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coke-asphalt overlay</td>
<td>Decks</td>
<td>20</td>
</tr>
<tr>
<td>Slotted conductive polymer grout</td>
<td>Decks</td>
<td>15</td>
</tr>
<tr>
<td>Mounded conductive polymer with concrete overlay</td>
<td>Decks</td>
<td>20</td>
</tr>
<tr>
<td>Titanium mesh with concrete overlay</td>
<td>Decks</td>
<td>35</td>
</tr>
<tr>
<td>Titanium mesh with shotcrete</td>
<td>Substructures</td>
<td>35</td>
</tr>
<tr>
<td>Conductive paint</td>
<td>Substructures</td>
<td>5</td>
</tr>
<tr>
<td>Sprayed zinc</td>
<td>Substructures</td>
<td>15</td>
</tr>
</tbody>
</table>

Table 1 Estimated service life of cathodic protection system.
During the effective period of the cathodic protection system, the corrosion current of the reinforcing steel is neglected even though the chloride concentration is higher than the threshold concentration. When the cathodic protection system is ineffective, the corrosion current is similar to no protection RC structure, as shown in Eq. (13).

\[
\begin{align*}
    i_{corr}(t) &= 0 ; t \leq t_{eff} \\
    &\text{if } t > t_{eff} \\
\end{align*}
\]

(13)

where \(i_{corr}(t)\) is the corrosion current of reinforcing steel at time \(t\) (\(\mu\)A/cm\(^2\)) and \(t_{eff}\) is the effective service life of the cathodic protection system (year).

3. Probability-based deterioration prediction model

Due to the high level of uncertainty that exists in reality, it is difficult to predict the degree of deterioration of RC structures with certainty. Moreover, in practice, limited data and incomplete knowledge about the parameters used in the prediction model, and the varying nature of environmental conditions introduce a degree of uncertainty in prediction results. Instead of considering safety factors, a probability-based model is used in this study to directly consider uncertainties. Monte Carlo simulation is one of the methods used to deal with probability-based simulation and it is used in this study.

Monte Carlo simulation (Fishman 1995) uses generated random numbers and probability statistics sampling of uncertainty variables to provide approximate solutions to a variety of mathematical problems. The values of parameters are randomly generated based on the determined probability density function of each parameter and used in the prediction model. Therefore, variations in deterioration degree can be predicted. Table 2 lists a sample of the various parameter values used in prediction from the literature (Novak et al. 1994, Liu and Weyers 1998, Enright and Frangopol 1999, and JSCE 2005b). In reality, actual variation of parameters as shown in Table 2 can be obtained directly by inspecting the target structure before conducting maintenance planning. Due to the variation of parameters used in prediction models, variation of prediction results can be seen in Figs. 5 and 6 for the chloride content at the steel surface and the corrosion crack width at the concrete surface, respectively. Because the distribution function of input parameters differ (log-normal, normal, or constant) as shown in Table 2, and a random numbers is generated independently for each distribution function of input parameters, the prediction results shown in Figs. 5 and 6 show unique distributions. The distribution of prediction results can be used to further estimate the amount of repair and the repair cost.

3.1 Probability of Damage

The probability of damage is defined as the probability that the limit state of a system will be violated after time period \(t\). In order to determine the probability of damage, a limit state function has to be defined. Generally, a limit state function can be defined in terms of system resistance against applied load over time in different ways. In this study, the corrosion crack width is considered as a limit state. Therefore, the probability of damage is the probability that the corrosion crack width will be larger than the limit crack width. The crack width is limited as the deterioration of RC structure accelerates in the presence of cracks. JSCE (2005b) regulates the allowable crack width as shown in Table 3. The limit state function is shown in Eq. 14.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Type of distribution</th>
<th>Mean</th>
<th>Coefficient of variation</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>(x)</td>
<td>Log Normal</td>
<td>38.1 mm</td>
<td>0.05</td>
<td>Enright and Frangopol (1999)</td>
</tr>
<tr>
<td>(D_{cl})</td>
<td>Log Normal</td>
<td>1.29 cm(^2)/year</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>(C_{lim})</td>
<td>Log Normal</td>
<td>0.05 % by weight of concrete</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>(C_s)</td>
<td>Log Normal</td>
<td>0.20 % by weight of concrete</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>(D)</td>
<td>Normal</td>
<td>1.6 cm</td>
<td>0.015</td>
<td>Nowak et al. (1994)</td>
</tr>
<tr>
<td>(f'_c)</td>
<td>Normal</td>
<td>35.1 MPa</td>
<td>0.18</td>
<td></td>
</tr>
<tr>
<td>(i_{cor})</td>
<td>Constant</td>
<td>2 (\mu)A/cm(^2)</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>(d_0)</td>
<td>Constant</td>
<td>12.5 (\mu)m</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>(\alpha_{rust})</td>
<td>Constant</td>
<td>0.57</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>(\rho_{rust})</td>
<td>Constant</td>
<td>3600 kg/m(^3)</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>(\rho_{fr})</td>
<td>Constant</td>
<td>7850 kg/m(^3)</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>(E_{fr})</td>
<td>Constant</td>
<td>30.1 GPa</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>(\phi_{cr})</td>
<td>Constant</td>
<td>1.1</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>(\nu)</td>
<td>Constant</td>
<td>0.20</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Table 2 Random variables of parameters.
where \( w_{\text{clim}} \) is the allowable crack width (mm) as shown in Table 3, and \( w_c(t) \) is the corrosion crack width at time \( t \) (years) (mm).

Individual random variables required in prediction by Monte Carlo simulation are generated. The value of the limit state function at time \( t \), \( Z(t) \), is evaluated for \( N \) times. The whole life performance of a deteriorating structure can be characterized by finding its probability of damage over the lifetime interval \((0, T]\) as shown in Eq. 15.

\[
P_f(t) = \frac{N_f(t)}{N}
\]

where \( P_f(t) \) is the probability of damage at time \( t \), \( N_f(t) \) is the number of evaluations where \( Z(t) < 0 \), and \( N \) is the total number of evaluations.

The effect of cracking due to drying shrinkage, thermal cracking since the beginning of the service life of the structure, or corrosion cracking after the structure entered service can be considered in the prediction of the probability of damage. After cracking is initiated due to corrosion, the rate of chloride diffusion is accelerated. Therefore, the probability of damage increases faster after initiation of corrosion cracking after year 7 compared to when cracking is not considered, as shown in Fig. 7. Moreover, the effect of macrocell corrosion also increases the probability of damage. As shown in Fig. 7, considering initial crack does not significantly affect the result of probability of damage.

### 3.2 Effect of repair on probability of damage

After repair has been conducted, the performance of the repaired section normally recovers to the non-deteriorated condition, and the durability of the repaired section is normally improved, whereas non-repaired sections continue to deteriorate. There is also a difference in the deterioration rate of non-repaired and repaired sections. In this study, differences in the deterioration rates of repaired and non-repaired sections are also considered.

In this section, the parameters listed in Table 2 are used for the calculation of structural performance, and the chloride diffusion coefficient of the patching material is 0.1 cm²/year. Figure 8 shows a comparison of the probability of damage between fully recovered and partially recovered repaired sections. The target structure was divided into 1000 sections and the degree of deterioration was determined for each divided section. The number of sections that fail at the limit state and need repair can be calculated based on the variation in inspection results as discussed in section 3.1. The macrocell corrosion current is affected by the number of re-

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**Table 3 Permissible crack width.**

<table>
<thead>
<tr>
<th>Type of steel</th>
<th>Environmental condition</th>
<th>Normal</th>
<th>Corrosive</th>
<th>Severely corrosive</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformed and Plain bar</td>
<td>0.005C</td>
<td>0.004C</td>
<td>0.0035C</td>
<td></td>
</tr>
<tr>
<td>Prestressed steel</td>
<td>0.004C</td>
<td>-------</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( C = \) covering depth (mm)

\[
Z_i(t) = w_{\text{clim}} - w_c(t)
\]

\[
P_f(t) = \frac{N_f(t)}{N}
\]
paired sections as the result of differences in chloride content between repaired and non-repaired sections. In the case of 100% recovery, this means all divided sections of a structure are assumed to have been recovered to the initial condition (0% chloride content) after repair at year 40 has been conducted. While in the case of partial recovery, this means that only the divided sections that need repair at year 40, which account for 40% of the structure as shown in Fig. 8, have recovered to the initial condition (0% chloride content). The chloride content of the remaining 60% of structures that have not been repaired remains undiminished and continues increasing. As shown, the probability of damage of partially recovered cases re-deteriorates faster than that of 100% recovered. The effect of each repairing method considered in this study on the probability of damage is discussed in the next section.

3.2.1 Concrete surface coating
Figure 9 shows the effect of the surface coating thickness on the probability of damage due to corrosion crack width. As shown, the probability of damage decreases as the thickness of the coating layer increases during the service life of the surface coating. In the case of a thin surface coating, the probability of damage increases before the end of the service life of the surface coating because the surface coating develops cracks before the end of the service life.

3.2.2 Patching repair
The probability of damage of a patching repair system depends on the performance of patching repair, which is mainly affected by the chloride diffusion coefficient and macrocell corrosion. Figure 10 shows the effect of different diffusion coefficients of chloride ions of the patching material on the probability of damage. As the chloride diffusion coefficient of the patching material decreases, the probability of damage of the corrosion crack width also decreases following repair because the lower chloride diffusion of the repair material can decrease the rate of re-deterioration of the repaired section.

3.2.3 Cathodic protection
As cathodic protection can prevent corrosion of reinforcing steel due to applied current, the system has an effective service life. In this section, the parameters listed in Table 2 are used in calculation of structural performance and 20 years is used as the service life of a cathodic protection system. The chloride diffusion coefficient of the patching material is 0.1 cm²/year and the thickness of the coating material is 2 mm. The structure is assumed to be repaired at year 40 by different repair methods. The effect of the cathodic protection system on the probability of damage due to corrosion crack width is shown in Fig. 11. This figure shows also a comparison between cathodic protection and other repair methods. As shown, following repair of the structure at year 40 using different repair methods, the probability of damage of repair by cathodic protection is the lowest among the various repair methods. After the effective period of cathodic protection, the probability of damage increases rapidly after cathodic protection system failure because there is no longer protection against chloride penetration. Application of cathodic protection with surface coating can minimize the probability of damage even after the failure of the cathodic protection system.
The general goals of engineering design are maximizing the utility of a structure while simultaneously minimizing its life-cycle costs, which actually include the costs for developing, manufacturing, and maintaining the structure. This task is complicated by the inherently non-deterministic nature of the structure itself and the environmental conditions to which it is exposed. Also, many maintenance methods presenting different costs and performance are available. Together with the probability-based deterioration prediction model, the maintenance program has to be economically planned. Life cycle cost evaluation is one of the economical tools that can help decision makers determine the most suitable plan. The repair method and its schedule can be planned based on the method proposed in this study.

4.1 Life cycle cost calculation

Life cycle cost can be classified mainly into agency direct cost and user cost. Agency direct cost includes all costs incurred directly by the agency over the life of the project such as construction cost, inspection cost, and repair cost. User cost consists mainly of cost of traffic delay or detour of users of that facility. Failure cost due to loss of life or property is also considered to be part of user cost. In this study, life cycle maintenance cost is considered to consist mainly of the repair cost, user cost, and failure cost.

4.1.1 Repair cost

In order to determine repair cost, information on the cost of repair and the expected quantity of repair have to be determined for each repair method as shown in Eq. 16.

$$E[C_{ri}] = \sum_{i=1}^{n} \left[C_{ri} + (C_{ri} \cdot p_{ri})\right]$$

where $E[C_{ri}]$ is the expected cost of repair $i$th at time $t$, $C_{ri}$ is the undiscounted fixed repair cost of repair $i$th, $C_{ri}$ is the undiscounted variable repair cost of repair $i$th, and $p_{ri}$ is the quantity of repair $i$th at time $t$.

In this study, surface coating, patching repair, and cathodic protection are considered as the available repair methods. Their costs of repair can be determined from historical cost data. Fixed cost, variable cost, and annual maintenance cost are defined, as follows. Fixed cost is repair cost that does not depend on the quantity of repair. It is the minimum amount of resources that have to be used in order to conduct one repair such as the cost of heavy equipment, office space, etc. Variable cost is repair cost that depends on the quantity of repair such as single-use items, material, and labor. In some repair methods such as cathodic protection, the annual maintenance fee includes electricity including power used for system monitoring is also a factor. Historical data of the Strategic Highway Research Program (SHRP) is used as the repair cost data in this study. SHRP (1993b) reported cost information of surface coating and patching repair. SHRP (1993c) reported cost information of various cathodic protection systems. A sample of the cost data is shown in Table 8. The quantity of repair is determined from the deterioration prediction result based on probability-based model explained in the previous section.

4.1.2 User cost

User cost is made up of added vehicle operating costs and delay costs to highway users resulting from the deteriorated condition of the structure. In this study, time loss due to increased travel time of users along the deteriorated target structure and loss of life and property of users due to structural failure are considered. Note that different user costs should be considered for different types of structures.

User loss time is the difference between the travel time of users along a healthy structure and deteriorated structure, as shown in Eq. 17.

$$t_{loss}(t) = \frac{\text{Dist}}{\alpha \cdot V_i} - \frac{\text{Dist}}{V_i}$$

where $t_{loss}(t)$ is the time loss of user (min), Dist is the travel distance of users along the target structure, $V_i$ is the designed travel speed along the target structure, and $\alpha$ is the reduction ratio of travel speed of users along the deteriorated structure and can be determined from Eq. 18 obtained from the relation between the average speed and the ratio of current volume of users to capacity of the structure (New Jersey Department of Transportation 2001). Note that maintenance planning should use local information, if available, for appropriately determine the maintenance plan. In this study, results from the USA and Japan are used as examples as there is a shortage of user cost information in Thailand. However, this data from the USA and Japan is not actually appropriate for Thailand. Local information on the designed capacity of traffic volume and reduction of traffic volume at each level of structural deterioration should be collected and used in Eq. 18, as well as the local user cost, shown in Table 4, in the future.
\[ \alpha_i = 0.7143 \left[ \frac{Vol(t)}{Vol_i} \right]^2 + 0.238 \left[ \frac{Vol(t)}{Vol_i} \right] + 0.019 \quad (18) \]

where \( Vol_i \) is the designed capacity of traffic volume of the target structure (vehicles/hour) (in this study 2200 vehicles/hour (New Jersey Department of Transportation 2001) is used), and \( Vol(t) \) is the maximum capacity of traffic volume of the deteriorated structure at time \( t \) (vehicles/hour), which can be determined from Eq. 19.

\[ Vol(t) = Vol_i - p_f(t) \cdot (Vol_i - Vol_{min}) \quad (19) \]

where \( p_f(t) \) is the probability of damage due to crack width at time \( t \), and \( Vol_{min} \) is the minimum capacity of traffic volume along the fully deteriorated structure.

The Study Group on Road Investment Evaluation (2000) has determined the time value of road users, as listed in Table 4. Therefore, user cost can be calculated based on the expected volume of users and predicted time loss of users.

### 4.1.3 Net present value

In order to compare life cycle cost along the service life of the structure, it is important to convert the value of money at different times to a common point in time. Present value (PV) uses an appropriate discount rate to determine the value of future costs at present time, as shown in Eq. 20.

\[ PV_{LCC} = \sum_{i=1}^{n} \left[ E(C_{r,i}(t)) + C_f(t) \right] \frac{1}{(1+r)^t} \quad (20) \]

where \( PV_{LCC} \) is the present value of life cycle cost, \( E(C_{r,i}(t)) \) is the expected cost of repair \( i^{th} \) at time \( t \), \( C_f(t) \) is the failure cost at time \( t \), and \( r \) is the discount rate.

### 5. Case studies

In order to demonstrate the application of the maintenance management program proposed in this study, actual RC structures attacked by chloride induced corrosion were selected for application of the methods proposed in the previous section. Two structures, one representative of a bad quality structure and the other of a good quality structure, were selected. In the case of a good quality structure, emphasis is placed on structural durability starting from the design phase, such as increasing covering depth, specifying a selective mineral admixture to improve the durability of the concrete material, and good quality control of construction. The background information of the selected structures is given in Table 5. Figures 12 and 13 show the location and pictures of the structures. An inspection program was conducted to determine variations of structural properties such as covering depth, chloride diffusion coefficient, and compressive strength. Variation of deterioration degree was predicted and maintenance planning was proposed based on the expected life cycle cost. Note that the two structures have different ages in service. Thus, the older structure may show higher compressive strength due to full hydration and the younger structure may show a higher apparent chloride diffusion coefficient due to a lower degree of hydration. Inspection should be periodically conducted during the service life of each structure. Inspection results should be used to predict the structural performance during different periods.

### Table 4 Time value of user.

<table>
<thead>
<tr>
<th>Vehicle class</th>
<th>Time value (yen/vehicle-minute), (US$/vehicle-minute)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Weekday</td>
</tr>
<tr>
<td>Passenger car</td>
<td>56 (0.54)</td>
</tr>
<tr>
<td>Bus</td>
<td>496 (4.75)</td>
</tr>
<tr>
<td>Small truck</td>
<td>90 (0.86)</td>
</tr>
<tr>
<td>Ordinary truck</td>
<td>101 (0.97)</td>
</tr>
</tbody>
</table>

### Table 5 Information of inspected structures.

<table>
<thead>
<tr>
<th>Case</th>
<th>Location</th>
<th>Year of construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bad</td>
<td>Samutprakan</td>
<td>1963</td>
</tr>
<tr>
<td>Good</td>
<td>Chanthaburi</td>
<td>2005</td>
</tr>
</tbody>
</table>
5.1 Inspection program
Sancharoen et al. (2006a) revealed that parameter variables, including steel covering depth, chloride diffusion coefficient, surface chloride content, and concrete compressive strength, significantly affect the performance of RC structures against chloride attack as well as the prediction result of maintenance planning. Therefore, the inspection program conducted in this study was mainly focused on determining the random variable of these parameters, as explained below. The total numbers of samples of each inspected item are listed in Table 6.

The covering depth of reinforcing steel was measured by a rebar detector based on the principle of the electromagnetic method. Only concrete with a smooth surface was inspected. Then all of data were collected together for each structure to finalize their variations. The chloride diffusion coefficient and surface chloride content were calculated from the profile of chloride content. Samples of concrete powder were collected on site using a drilling machine with a 14 mm diameter drilling bit based on the method of JSCE-G573. Sample powder was collected from three adjacent holes with depths of 0-2cm, 2-4cm, 4-6cm, 6-8cm, and 8-10cm from the surface to minimize the effect of aggregate size. Samples were collected not only at the same level from sea level but also at different heights in order to determine the effect of height from the sea level. The total chloride content of the powder was measured based on the method of JCI-SC4. Then the chloride diffusion coefficient and surface chloride content were calculated from the profile of chloride content. The number of samples for chloride analysis was limited owing to difficulty and high resource consumption. The average value was used from the inspection result but variation was assumed to be equal to variation of concrete properties measured by rebound hammer and normal distribution was used as the distribution type, as shown in Table 7.

Concrete compressive strength was measured by rebound hammer. Although there are other NDT methods such as the air permeability test, the rebound hammer is still one of the most convenient methods. Twenty points were tested for one sample set. Only smooth concrete surfaces were tested. As currently in Thailand there is no available formula to relate rebound number to compressive strength, concrete compressive strength can be calculated from the rebound number as shown in Eq. 21 (JSCE 2005d). The calculated compressive strength is in the range of the strength class normally used in Thailand.

\[
f'_c = -18 + (1.27 \times RN)
\]  

where \(f'_c\) is the concrete compressive strength (MPa) and RN is the rebound number.

5.2 Inspection results
By the goodness-of-fit test, the most suitable distribution type and its parameters to inspected data were selected based on the chi-square goodness-of-fit test. The distribution type, its parameters, mean value, and coefficient of variation are reported in this section. Table 7 lists the inspection results of covering depth, surface chloride content, chloride diffusion coefficient, and concrete compressive strength of both structures. As a result, actual variation of inspection results can be considered in the prediction model. Normally, higher variation of the inspection results at the same average value caused the structure to deteriorate faster. The effects of degree
of variation of the inspection result are discussed by Sancharoen et al. (2006b). Other parameters used in prediction are from the literature (Nowak et al. 1994, Liu and Weyers 1998, Enright and Frangopol 1999, and JSCE 2005b), as shown in Table 2.

### Table 7 Inspection result.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Case</th>
<th>Specified value</th>
<th>Inspection result</th>
<th>Mean</th>
<th>COV, %</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Covering depth</td>
<td>Bad</td>
<td>50 mm</td>
<td>24.21 mm</td>
<td>38.69</td>
<td>Gamma</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>75 mm</td>
<td>88.34 mm</td>
<td>22.30</td>
<td>Weibull</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Chloride diffusion coefficient</td>
<td>Bad</td>
<td>-</td>
<td>1.34 cm²/year</td>
<td>18.35</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>-</td>
<td>0.92 cm²/year</td>
<td>19.89</td>
<td>Normal</td>
<td></td>
</tr>
<tr>
<td>Surface chloride content</td>
<td>Bad</td>
<td>-</td>
<td>8.71 kg/m³</td>
<td>-</td>
<td>Uniform</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>-</td>
<td>8.36 kg/m³</td>
<td>-</td>
<td>Uniform</td>
<td></td>
</tr>
<tr>
<td>Compressive strength</td>
<td>Bad</td>
<td>24 MPa</td>
<td>34.30</td>
<td>18.35</td>
<td>Extreme value</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>30 MPa</td>
<td>34.97</td>
<td>19.89</td>
<td>Extreme value</td>
<td></td>
</tr>
</tbody>
</table>

### Table 8 Conclusion of performance and cost of repair option.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Patching</td>
<td>$D_p = 0.175$ cm²/year</td>
<td>1500</td>
<td>200</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>Patching</td>
<td>$D_p = 0.175$ cm²/year</td>
<td>1500</td>
<td>200</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Surface coating</td>
<td>$C_s = 0.5$ mm $D_s = 1.0E-3$ cm²/year</td>
<td>500</td>
<td>200</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>Patching</td>
<td>$D_p = 0.175$ cm²/year</td>
<td>1500</td>
<td>200</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Cathodic protection</td>
<td>Titanium mesh (Service life 30 years)</td>
<td>6500</td>
<td>150</td>
<td>2.5</td>
</tr>
</tbody>
</table>

### 5.3 Maintenance planning

In this section, an example of a maintenance planning program for selected structures is given based on the actual inspection results of each structure. The service life of structures is considered to be 100 years. Therefore, there are 53 and 99 years of service life left for the bad and good structure, respectively. The life cycle costs of the remaining service life of different maintenance interventions are optimized and compared, using the 0% discount rate. The maintenance method with the lowest life cycle cost is considered as the most suitable option. The applicable interventions to maintain the structures considered in this study are shown below as examples.

As patching repair and surface coating are the most frequently applied repairing methods in Thailand, they are considered in this study. Moreover, cathodic protection is also considered for comparison purposes. Combinations of different repairing methods are also considered, such as the combination of patching with surface coating or cathodic protection.

- **Patching**: Corrective maintenance through removal of chloride contaminated concrete and execution of patching repair as shown in Table 8 after the limit corrosion crack width was reached.
- **Patching and surface coating**: Preventive and corrective maintenance by initial coating of overall surface of structure, and removal of chloride contaminated concrete, patching of repaired material and surface coating as shown in Table 8 after limit corrosion crack width was reached.
- **Patching and cathodic protection**: Corrective maintenance by removal of chloride contaminated con-
crete, patching of repaired material and cathodic protection as shown in Table 8 after limit corrosion crack width was reached. As there are various systems of cathodic protection, cathodic protection with a service life of 30 years is selected for application in this section.

Figures 14 and 15 show the probability of damage and life cycle cost in US$ of the bad quality structure following repair by different methods using the 0% discount rate. Only the schedule of repair that results in the lowest life cycle cost is shown in Fig. 15. As shown, the life cycle cost of the repaired case is significantly lower than the no repair case due to failure cost decreases. For the bad quality structure, 2 times patching with cathodic protection repair at year 49 and 79 shows the lowest life cycle cost even though the initial repair cost is higher than the other two options.

For the good quality structure, due to its designed covering depth having been increased to improve resistance against chloride attack, the probability of damage due to corrosion cracking is very low throughout the service life of the structure, as shown in Fig. 16. Therefore, repair is actually not necessary throughout the remaining service life of the structure. As shown in Fig. 17, the lowest life cycle cost of the good quality structure is the case without repair because failure is not significantly in this structure.

6. Conclusion

As an alternative to the current method of maintenance of RC structures based on safety factors, this study proposed a method of maintenance planning based on variations of actual inspection results and probability theory to predict variations of deterioration degree of RC structures and life cycle cost of maintenance. Therefore, not only the average value of inspection results is used in prediction but also their variations. The effects of cracks, macrocell corrosion, partial repair, and performance of different repair methods are considered in deterioration prediction. Therefore, time dependent variations of damage can be determined and repairing cost can be also estimated. The life cycle cost is calculated and maintenance planning can be decided based on optimization of the life cycle cost. The results show that the good quality structure requires lower maintenance cost during its service life compared to the bad quality structure.

In the future, besides salt attack, other deterioration mechanisms should also be considered for deterioration prediction. Moreover, prediction of damage location should also be considered in order to minimize the need for another inspection program for damage when repair has to be conducted. Further, more information on performance and repair cost is required for accurate determination of the repair method.
References


“Effect of variation of parameters used in the deterioration prediction model on the R&M Plan.”


