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RUST-EXPANSION-CRACK SERVICE LIFE PREDICTION OF EXISTING REINFORCED CONCRETE BRIDGE/VIADUCT USING TIME-DEPENDENT RELIABILITY ANALYSIS

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RUST-EXPANSION-CRACK SERVICE LIFE PREDICTION OF EXISTING REINFORCED CONCRETE BRIDGE/VIADUCT USING TIME-DEPENDENT RELIABILITY ANALYSIS

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Key words: bridge, chloride, corrosion, reinforced concrete, rust-expansion-crack, service life, viaduct.

ABSTRACT

It is necessary to develop a calculation method to help in the making of feasible, reliable, and serviceable predictions for the service lives of bridge or viaduct structures. This article presents the basis for doing rust-expansion-crack service life predictions for existing reinforced concrete (RC) bridges and viaducts in chloride-laden environments based on time-dependant reliability modeling due to the corrosion of steel in concrete. The corrosion process has three stages, the initiation (diffusion or carbonation) time ($t_i = t_c$), the depassivation time (t_p), and the propagation (corrosion) time (t_{corr}). The rust-expansion-crack service lives (t_{cr}) of existing RC bridges or viaducts can be expressed in terms of $t_{cr} = t_c + t_p$. Many mathematical models could be applied to calculate each value of t_c and t_p. The values of t_{cr} may be directly predicted from the relationship between reliability index and time. The existing Wann-fwu bridge and Chorng-ching viaduct in Taipei were provided as illustrative examples for the modeling approach and rust-expansion-crack service life prediction. The results of t_{cr} predicted from the relationship between reliability index and time were in good agreement with the results of t_{cr} calculated from the sum of t_c and t_p. The results of present study were offered as a decision making for repair, strengthening, and demolition of existing RC bridges or viaducts.

I. INTRODUCTION

Reinforced concrete (RC) is used for an increasing number of dams, buildings, airports, coastal embankments, road and railway bridges, harbors and wharfs, marine and ocean structures. Corrosion of reinforcing bars in RC structures is the major cause of structural deterioration of structural members. Nevertheless, if these structures are exposed long-term to a bad environment (chloride ions, carbon dioxide, sulfate dioxide, etc.), then their service lives will be reduced. The safety and health monitoring of RC structures in active service should be laid stress on this issue. The durability problem of RC structures, especially the infrastructure, must be investigated. To make the structure have longer service life, we can choose a better design method in the design stage as stated in the article done by Paik and Thayamballi [15]; or we can place some sensors to detect the health of the structure [9]; or we can adopt reliability theory to have a better management of structures [12]; we can retrofit the structures to enhance its capability [19]. A worthwhile topic for study would be to set up a complete evaluation method for effectively calculating the service lives of RC structures and for accurately offering determination for repair, strengthening, or demolition.

A number of researchers have begun to develope the evaluation methods or non-destructive techniques and to study the safety and durability problems of RC structures. Crumpton and Bukovatz [3] used the copper-copper sulfate half-cell potential detection method to estimate the Kansas bridge deck corrosion due to deicing salts. Stratfull *et al*. [18] employed half-cell potential associated with inspection techniques to evaluate the corrosion behavior of the steel in the bridge decks. They pointed out that the corrosive half cell potentials on a bridge deck exceed about 10 percent or when corrosion-caused delamination exceed about 1 percent of the deck area, a chloride analysis generally would not be required because the chloride content is already too great. Gjorv and Kashino [6] made a detailed investigation of durability of a 60-year-old RC pier in Oslo harbor during demolition. The overall structural quality of the concrete was very good. However, poor frost resistance had damaged parts of the structure. In order to select a rehabilitation alternative for a fifty-year-old RC bridge, Shroff [16] used field inspection and petrographic examination to determine the strength and quality of the existing RC

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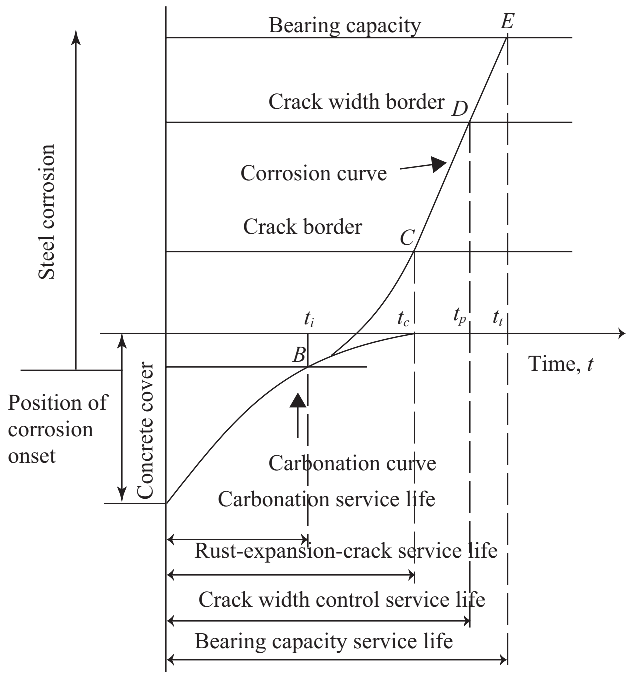

Fig. 1. Schematic diagram of steel corrosion and service life of reinforced concrete structure.

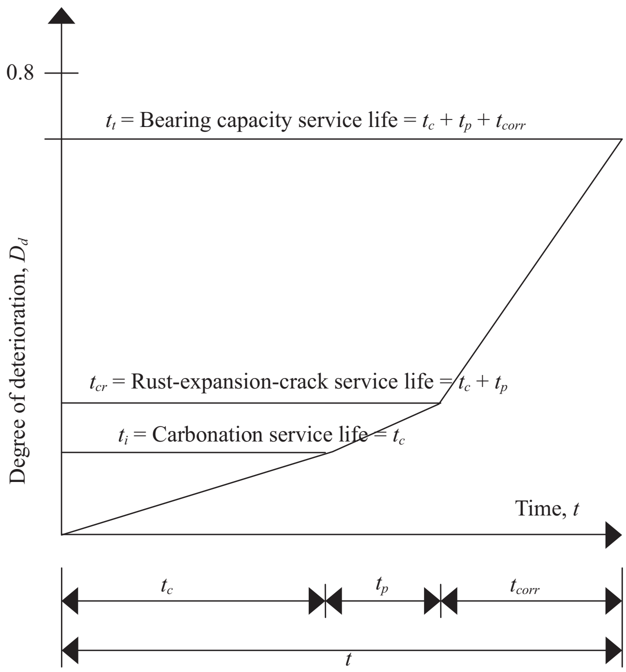

Fig. 2. Schematic diagram of service life of existing reinforced concrete bridge.

bridge. Woodward [22] described an investigation into the collapse of a single-span, segmental post-tensioned concrete bridge. It was built in 1953 and collapsed at approximately 7:00 A.M. on December 4, 1985. Chlorides were the major cause of corrosion. The bridge collapsed because of corrosion of the tendons where they crossed the joints. Failure occurred when the sectional area of the steel had been reduced to the point where it could no longer carry the imposed load. Novokshchenov [14] surveyed three prestressed concrete bridges. He found that corrosion of the steel and corrosion-related deterioration of concrete occurred due to chloride content. The principal source of corrosion-inducing agents was chloride-laden water coming from bridge deck. Vaysburd [21] pointed out that approximately 564,000 bridge are standing in the United States. At least 105,500 of them need repair or replacement. He also believed that both thickness and quality of the concrete in the protective layer are important for protection of steel reinforcement against corrosion and durability of a RC bridge. Tabsh and Nowak [20] thought that reliability is a convenient measure of bridge performance. There is a need for an efficient methodology to be employed in the development of bridge evaluation and design criteria. Stewart and Rosowsky [17] developed a time-dependent reliability analysis for evaluating RC bridge deck and the consequent loss of structural and serviceability performance due to chloride-induced corrosion. Englund *et al*. [4] developed a probabilistic approach to evaluate the repair and maintenance strategies for concrete coastal bridges. Liang *et al*. [11] used a service life model which consists initiation (diffusion) time included depassivation time and propagation (corrosion) time to predict the service life of Chung-shan bridge in Taipei.

Although these studies have provided much valuable information on the corrosion and durability evaluation of existing RC bridges, there are still many evaluation methodologies that have not yet been explored. This paper describes a time-dependent reliability analysis that predicted the service lives of existing RC bridges or viaducts. The results of this study may be provided as a decision making for the bridge system management of existing RC bridges or viaducts.

II. SERVICE LIFE MODEL OF RC STRUCTURE OR VIADUCTS

In order to predict the service lives of existing RC structures, the prediction model should be first established. Huey [8] provided the service life model of RC structures as shown in Fig. 1. In the Fig. 1, t_i is the time of CO_2 penetrating from concrete and neutralizing concrete such that steel embedded in concrete begins to corrode, i.e., $t_i = t_c$ = carbonation service life. t_{cr} is the time that the surface of concrete has occurred stain due to the corrosion of steel in concrete. t_w is the time that the concrete surface has happened cracking. t_t is the time of load-carrying capacity service life. Fig. 2 indicates the deterioration process of RC structures subjected to corrosion media ingress. The corrosion process in Fig. 2 can be divided into three stages, initiation time ($t_i = t_c$), depassivation time (t_p), and corrosion (or propagation) time (t_{corr}). The initiation time is defined as the time for CO_2 to penetrate from the concrete surface onto the surface of the passive film. The depassivation time is defined as the time that the depassivation normally provided to the steel by the alkaline hydrated cement matrix is locally destroyed, leading to uniform corrosion. The corrosion time extends from the time when corrosion products form to the stage where they generate sufficient stress to disrupt the concrete cover by cracking or spalling, or when the local corrosion attack onto the reinforcement becomes sufficiently severe to impair the load-carrying capacity. The degree of deterioration, D_d, in Fig. 2 can be defined as

$$D_d = 1 - \frac{x}{10} \tag{1}$$

where x is the integrity of the RC structure. The x value ranges from zero to ten. For instance, if RC structure is free of corrosion damage then the value of x is ten. Thus, the degree of deterioration is zero.

Based on the service life models of Figs. 1 and 2, we may make the following relationships

$$t_{cr} = t_c + t_p \tag{2}$$

and

$$t = t_t = t_c + t_p + t_{corr} \tag{3}$$

From Eq. (3) we know that the service lives of existing RC structures can be calculated employing the t_c, t_p, and t_{corr} values.

At present, it is needed to point out that Fang [5] developed a time-dependent reliability analysis to predict the values of t_{cr} of the existing RC bridges or viaducts. How to predict the values of t_c of the existing RC bridges or viaducts is investigated in this paper whereas how to predict the values of t_t is a future work.

III. ANALYTICAL THEORY OF RUST-EXPANSION-CRACK SERVICE LIFE

The criterion of rust-expansion-crack service life of existing RC bridges or viaducts can be expressed as

$$\Omega_{cr}(t) = \{\delta_{cr} - \delta_{el}(t) \geq 0\} \tag{4}$$

where δ_{cr} is the steel corrosion depth(mm) when concrete cover occurs crack due to rust expansion of steel in concrete, and is a random variant, $\delta_{el}(t)$ is the steel rust volume before rust-expansion-crack and is a random process, and $\Omega_{cr}(t)$ is the criterion of rust-expansion-crack and is a random process.

The prediction model of steel corrosion depth ($\delta_{el}(t)$, mm) before cracking of the concrete cover is [13]

$$\delta_{el}(t) = \lambda_{el}(t - t_i) \tag{5}$$

$$\lambda_{el} = 46\, k_{cr}\, k_{ce}\, e^{0.04T}(\text{RH-0.45})^{\frac{2}{3}} c^{1.36} f_{cu}^{-1.83} \tag{6}$$

where λ_{el} is the rate of corrosion of steel (mm/yr) before rust-expansion-crack, t is the time (yr) of steel corrosion, t_i is the onset time (yr) of steel corrosion, k_{cr} is the correction factor of steel location, k_{cr} = 1.6 for steel at corner, k_{cr} = 1.0 for steel at medium, k_{ce} is the correction factor of environmental condition, k_{ce} = 3.0~4.0 outdoor and k_{ce} = 1.0~1.5 indoor in moist region, k_{ce} = 2.5~3.5 outdoor and k_{ce} = 1.0 indoor in dry region, T is the temperature, RH is the relative humidity, c is the concrete cover, f_{cu} is the cube compressive strength of concrete and e = 2.71828…

The steel corrosion depth, $\delta_{el}(t)$, before rust-expansion-crack is a random process obeyed logarithmic normal distribution. Its one-dimensional probability density function is

$$f_1(x,t) = \frac{1}{\sqrt{2\pi}\, x \sigma_1(t)} \exp\{-\frac{1}{2}[\frac{\ln x - \mu_1(t)}{\sigma_1(t)}]^2\} \tag{7}$$

where $\mu_1(t)$ and $\sigma_1(t)$ are the mean and standard deviation functions of $\ln \delta_{el}(t)$ at time t, respectively. They can be calculated by using the following formulas

$$\mu_1(t) = \ln \mu_{\delta_{el}}(t) - \frac{\ln[\delta^2_{\delta_{el}}(t) + 1]}{2} \tag{8}$$

$$\sigma_1(t) = [\ln \delta^2_{\delta_{el}}(t) + 1]^{\frac{1}{2}} \tag{9}$$

where $\delta_{\delta_{el}}(t) = \dfrac{\sigma_{\delta_{el}}(t)}{\mu_{\delta_{el}}(t)}$, in which $\mu_{\delta_{el}}(t)$, $\sigma_{\delta_{el}}(t)$ and $\delta_{\delta_{el}}(t)$ are the mean, standard deviation, and variance functions of $\delta_{el}(t)$, respectively.

According to error propagation formula, both the mean and standard deviation functions of $\delta_{el}(t)$ and λ_{el} can be respectively represented as

$$\mu_{\delta_{el}}(t) = \mu_{k_{mel}} \mu_{\lambda_{el}}(t - t_i) \tag{10}$$

$$\mu_{\lambda_{el}} = 46 k_{cr} k_{ce} e^{0.04T}(RH - 0.45)^{\frac{2}{3}} \mu_c^{-1.36} \mu_{f_{cu}}^{-1.83} \tag{11}$$

$$\sigma_{\delta_{el}}(t) = [(\frac{\partial \delta_{el}}{\partial k_{mel}}\Big|_{\mu})^2 \sigma^2_{k_{mel}} + (\frac{\partial \delta_{el}}{\partial \lambda_{el}}\Big|_{\mu})^2 \sigma^2_{\lambda_{el}}]^{\frac{1}{2}} \tag{12}$$

$$\sigma_{\lambda_{el}}(t) = [(\frac{\partial \lambda_{el}}{\partial c}\Big|_{\mu})^2 \sigma^2_c + (\frac{\partial \lambda_{el}}{\partial f_{cu}}\Big|_{\mu})^2 \sigma_{f_{cu}}]^{\frac{1}{2}} \tag{13}$$

where k_{mel} is the uncertainty coefficient due to the calculation model of rust-expansion crack and is a random variant, $\mu_{k_{mel}}$ and $\sigma_{k_{mel}}$ are respectively the mean and standard deviation functions of k_{mel}, $\mu_{\delta_{el}}(t)$ and $\sigma_{\delta_{el}}(t)$ are respectively the mean and standard deviation functions of $\delta_{el}(t)$, $\mu_{\lambda_{el}}$ and $\sigma_{\lambda_{el}}$ are respectively the mean standard deviation of λ_{el}, μ_c and σ_c are respectively the mean and standard deviation of c, $\mu_{f_{cu}}$ and $\sigma_{f_{cu}}$ are respectively the mean and standard deviation of f_{cu}, and $|\mu$ means that the partial derivative takes the value at the mean.

The steel corrosion depth (δ_{cr}, *mm*) at rust-expansion crack can be calculated by the following formulas [13]

$$\delta_{cr} = k_{mcr}k_{crs}(0.008\frac{c}{d} + 0.00055 f_{cu} + 0.022) \tag{14}$$

(deformed bar)

$$\delta_{cr} = k_{mcr}(0.026\frac{c}{d} + 0.0025 f_{cu} + 0.068) \tag{15}$$

(stirrup and mesh distributing bar)

where k_{mcr} is the uncertainty coefficient, k_{crs} is the influence coefficient of steel location, k_{crs} = 1.0 for steel at corner and k_{crs} = 1.35 for steel at noncorner, and d is the diameter (*mm*) of steel.

Assume that the steel corrosion depth at cracking of the concrete corner, δ_{cr}, is obeyed logarithmic normal distribution. Its one-dimensional probability density function can be written in terms of

$$f_2(x) = \frac{1}{\sqrt{2\pi}x\sigma_2}\exp[-\frac{1}{2}(\frac{\ln x - \mu_2}{\sigma_2^2})^2] \tag{16}$$

where μ_2 and σ_2 are respectively the mean and standard deviation of $\ln\delta_{cr}$. They can be calculated by the following formulas

$$\mu_2 = \ln\mu_{\delta_{cr}} - \frac{\ln[\delta_{\delta_{cr}}^2 + 1]}{2} \tag{17}$$

$$\sigma_2 = \{\ln[\sigma_{\delta_{cr}}^2 + 1]\}^{\frac{1}{2}} \tag{18}$$

where $\delta_{\delta_{cr}} = \frac{\sigma_{\delta_{cr}}}{\mu_{\delta_{cr}}}$, in which $\mu_{\delta_{cr}}$, $\sigma_{\delta_{cr}}$ and $\delta_{\delta_{cr}}$ are the mean, standard deviation, and variance of δ_{cr}, respectively. The mean $\mu_{\delta_{cr}}$, and standard deviation, $\sigma_{\delta_{cr}}$, of δ_{cr} can be calculated by the following formulas [13]

$$\mu_{\delta_{cr}} = \mu_{k_{mcr}}k_{crs}(0.008\frac{\mu_c}{\mu_d} + 0.00055\mu_{f_{cu}} + 0.022) \tag{19}$$

(deformed bar)

$$\mu_{\delta_{cr}} = \mu_{k_{mcr}}(0.026\frac{\mu_c}{\mu_d} + 0.0025\mu_{f_{cu}} + 0.068) \tag{20}$$

(stirrup and mesh distributing bar)

$$\sigma_{\delta_{cr}} = [(\frac{\partial\delta_{cr}}{\partial k_{mcr}}\Big|_{\mu})^2\sigma_{k_{mcr}}{}^2 + (\frac{\partial\delta_{cr}}{\partial c}\Big|_{\mu})^2\sigma_c{}^2 + (\frac{\partial\delta_{cr}}{\partial d}\Big|_{\mu})^2\sigma_d{}^2 + (\frac{\partial\delta_{cr}}{\partial f_{cu}}\Big|_{\mu})^2\sigma_{f_{cu}}{}^2]^{\frac{1}{2}} \tag{21}$$

where $\mu_{k_{mcr}}$ and $\sigma_{k_{mcr}}$ are respectively the mean and standard deviation of k_{mcr}, μ_d and σ_d are respectively the mean and standard deviation of d, and $|\mu$ means that the partial derivative takes the value at the mean.

The limit state equation of rust-expansion-crack of concrete cover is

$$Z(t) = \delta_{cr} - \delta_{el}(t) \tag{22}$$

The probability of rust-expansion-crack of concrete cover is

$$P_{f_{cr}}(t) = P\{Z(t) = \delta_{cr} - \delta_{el}(t) < 0\} \tag{23}$$

If the degree of durability of rust-expansion-crack of RC structure can be defined as the probability of concrete cover without rust-expansion-crack, then

$$P_{D_{cr}} = P\{Z(t) = \delta_{cr} - \delta_{el}(t) \geq 0\} = P\{\delta_{cr} > \delta_{el}(t)\} \tag{24}$$

Eq. (24) can be rewritten as

$$P_{D_{cr}} = P\{\frac{\delta_{cr}}{\delta_{el}(t)} \geq 1\} \tag{25}$$

Furthermore, Eq. (25) can be changed as

$$P_{D_{cr}} = P\{\ln\delta_{cr} - \ln\delta_{el}(t) \geq 0\} \tag{26}$$

Let

$$Z'(t) = \ln\delta_{cr} - \ln\delta_{el}(t) \tag{27}$$

Then $Z'(t)$ obeys standard normal distribution. Its mean and standard deviation can be expressed as

$$\mu_{Z'}(t) = \mu_2 - \mu_1(t) \tag{28}$$

$$\sigma_{Z'}(t) = [\sigma_2^2 + \sigma_1^2(t)]^{\frac{1}{2}} \tag{29}$$

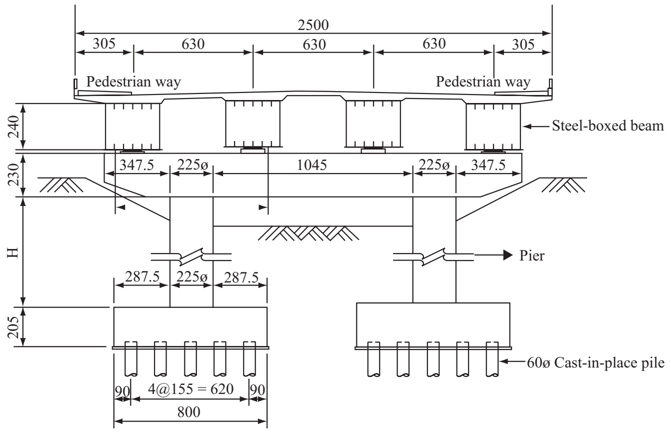

Fig. 3. Schematic diagram of cross-section of Wann-fwu bridge.

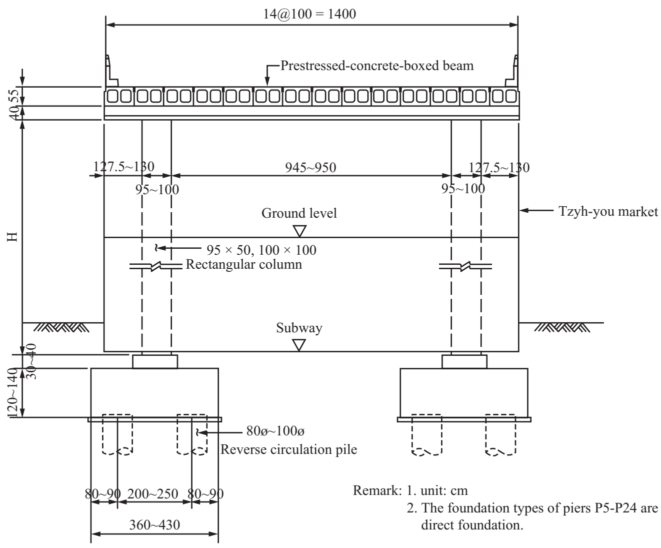

Fig. 4. Schematic diagram of cross-section of Chorng-ching viaduct.

Define the reliability index of rust-expansion-crack as

$$\beta_{cr}(t) = \frac{\mu_{Z'}(t)}{\sigma_{Z'}(t)} \tag{30}$$

The corresponding probability (or degree of durability) of rust-expansion-crack is

$$P_{D_{cr}} = \Phi(-\beta_{cr}(t)) \tag{31}$$

where Φ is the standard normal distribution function.

The probability of concrete cover occurring rust-expansion-crack is

$$P_{f_{cr}} = 1 - P_{D_{cr}} = 1 - \Phi(-\beta_{cr}(t)) \tag{32}$$

IV. ILLUSTRATIVE EXAMPLES

In order to examine the serviceability of the theory of rust-expansion-crack mentioned early, the existing Wann-fwu bridge and Chorng-ching viaduct in Taipei are employed to evaluate the rust-expansion-crack service life. The partially corresponding cross-sections of both the bridge and viaduct are portrayed in Figs. 3 and 4, respectively. Tables 1 and 2 are the compressive and design strengthes of the existing Wann-fwu bridge and Chorng-ching viaduct, respectively. Tables 3 and 4 show the testing data included concrete cover, steel diameter, corrosion current density, and chloride content of the existing Wann-fwu bridge and Chorng-ching viaduct, respectively. Table 5 denotes the CNS 3090 specification [2]. To use the theory stated above to estimate the service lives of the existing Wann-fwu bridge and Chorng-ching viaduct in Taipei many parameters should be well known. However, besides many parameters were provided in the Tables 1-4, other parameters were needed as follows : $\mu_{k_{mel}} = 0.996$, $k_{cr} = 1.6$, $k_{ce} = 3.5$, $k_{crs} = 1.35$ [13], $T = 21$°C, and RH = 70% (The annual average values of both T and RH of Taipei city in Taiwan from 2001 to 2010 were taken.). Moreover, using the conversion formulas $f'_{c(cylinder)} = 0.85 f'_{c(cu)} = 1.10 f'_{c(prism)}$ [7], the cylindrical compressive strength listed in Tables 1 and 2 were needed to convert into cubic compressive strength, f_{cu}. Substituting the well known parameters and the average values of compressive strengths and concrete covers listed in Tables 1-4 into Eqs. (11) and (13), the mean and standard deviation of corrosion rate of steel before the rust-expansion-crack of concrete cover were obtained. Putting the values of mean and standard deviation of corrosion rate of steel into Eqs. (10) and (12), the mean and standard deviation of corrosion thickness before the rust-expansion-cracks of concrete cover were attained. The coefficients of variation of corrosion thickness before the rust-expansion-crack of concrete cover were also calculated. Substituting the corresponding values into Eqs. (8) and (9), the logarithmic mean and standard deviation of corrosion thickness were obtained.

Inverting the values of concrete cover, steel diameter, and compressive strength listed in Tables 1-4 into Eqs. (19) and (21), the mean and standard deviation of corrosion thickness during rust-expansion-crack were obtained. The coefficients of variation of corrosion thickness during rust-expansion-crack were also calculated. Putting the corresponding values into Eqs. (17) and (18), the logarithmic mean and standard deviation of corrosion thickness during rust-expansion-crack were obtained. Finally, substituting the logarithmic mean and standard deviation of corrosion thickness before and during rust-expansion-crack of concrete cover into Eqs. (28) and (29), the mean and standard deviation obeyed standard normal distribution were obtained. Furthermore, from Eq. (30), the reliability index of rust-expansion-crack of existing bridge or viaduct is obtained while, from Eq. (31), the corresponding probability or the degree of durability of rust-expansion-crack is also obtained.

Based on the analytical results of the service lives of rust-expansion-crack, the durability degree and reliability

Table 1. Compressive and design strengths of Wann-fwu bridge.

Test Point No.	Testing Points	Compressive strength (kgf/cm^2)	Design strength (kgf/cm^2)
A	A1-P1 Slab (left)	237	240
B	P1-P2 Slab (left)	484	240
C	P2-P3 Slab (left)	353	240
D	P3-P4 Slab (left)	310	240
E	P4-A2 Slab (left)	354	240
F*	A1-P1 Slab (right)	380	240
G*	P1-P2 Slab (right)	584	240
H	P2-P3 Slab (right)	660	240
I	P3-P4 Slab (right)	313	240
J	P4-A2 Slab (right)	310	240
K	S1S1 (side)	246	240
L	P1 Right Capbeam (rear)	560	280
M	P1 Left Capbeam (rear)	374	280
N	P2 Right Capbeam (rear)	351	280
O	P2 Left Capbeam (rear)	307	280
P	P2 Middle Capbeam (rear)	305	280
Q	P3 Right Pier	411	280
R*	P4 Capbeam	352	280
S	P4 Left Pier	374	280
T	Retaining Wall (guide passage) (1)	450	280
U	Retaining Wall (guide passage) (2)	324	280
	Average	382.81	259.05

* The carbonation depth of cored sample has surpassed concrete cover. Remark: A: Abutment; G: Girder; P: Pier; S: Slab (bridge deck).

Table 2. Compressive and design strengths of Chorng-ching viaduct.

Test Point No.	Testing Points	Compressive strength (kgf/cm^2)	Design strength (kgf/cm^2)
A	A1 Abutment	248	210
B	P23 Left Pier	287	210
C	P3 Right Pier	196	210
D	P24 Right Pier	205	210
E	Retaining wall (terminal guide passage) (right)	241	210
F	P24 Left Pier	143	210
G	Retaining wall (terminal guide passage) (left)	225	210
H	Retaining wall (guide passage) (right)	321	210
I	Retaining wall (guide passage) (left)	291	210
J	P23 Right pier	287	210
K*	G6S4 (Girder)	361	350
L	G3S4 (Girder)	415	350
M	G10S4 (Girder)	269	350
N	G14S24 (Side)	622	350
O	G1S4 (Side)	415	350
P	G7S4 (Girder)	472	350
Q	G11S4 (2) (Girder)	562	350
R	G2S4 (Girder)	414	350
S	G14S23 (Side)	474	350
T	G1S23 (Side)	514	350
	Average	350.4	280

* The carbonation depth of cored sample has surpassed concrete cover. Remark: A: Abutment; G: Girder; P: Pier; S: Slab (bridge deck).

Table 3. Testing data of Wann-fwu bridge.

Test point No.	Test points	Concrete cover (mm)	Steel diameter (mm)	Corrosion current density ($\mu A/cm^2$)	Chloride content (kg/m^3)
A	A1-P1 Slab (left)	40	18.62	0.45	0.43
B	P1-P2 Slab (left)	40	19.64	0.16	0.38
C	P2-P3 Slab (left)	40	18.95	0.18	0.48
D	P3-P4 Slab (left)	40	19.78	0.52	0.35
E	P4-A2 Slab (left)	40	18.47	0.36	0.35
F*	A1-P1 Slab (right)	40	19.89	0.23	0.32
G*	P1-P2 Slab (right)	40	18.02	0.17	0.79
H	P2-P3 Slab (right)	40	19.63	0.31	0.68
I	P3-P4 Slab (right)	40	19.04	0.23	0.32
J	P4-A2 Slab (right)	40	18.75	0.36	0.31
K	S1S1(Side)	25	19.67	0.48	0.43
L	P1 Right cap beam (rear)	50	19.61	0.15	0.52
M	P1 Left cap beam (rear)	50	19.7	0.11	0.36
N	P2 Right cap beam (rear)	50	18.55	0.17	0.8
O	P2 Left cap beam (rear)	50	19.9	0.21	0.31
P	P2 Middle cap beam (rear)	50	18.31	0.18	0.43
Q	P3 Right pier	50	18.14	0.14	0.51
R*	P4 Cap beam	50	19.68	0.27	0.42
S	P4 Left pier	50	18.64	0.19	0.55
T	Retaining wall (guide passage) (1)	50	19.71	0.68	0.42
U	Retaining wall (guide passage) (2)	50	19.92	0.79	0.52
	Average	44.04	19.17	0.31	0.461

* The carbonation depth of cored sample has surpassed concrete cover. Remark: A: Abutment; G: Girder; P: Pier; S: Slab (bridge deck).

Table 4. Testing data of Chorng-ching viaduct.

Test point No.	Test points	Concrete cover (mm)	Steel diameter (mm)	Corrosion current density ($\mu A/cm^2$)	Chloride content (kg/m^3)
A	A1 Abutment	50	19.58	0.401	0.10
B	P23 Left pier	50	18.58	0.396	0.11
C	P3 Right pier	50	18.95	0.328	0.04
D	P24 Right pier	50	19.26	0.354	0.07
E	Retaining wall (terminal guide passage) (right)	50	18.1	0.319	0.14
F	P24 Left pier	50	19.88	0.325	0.10
G	Retaining wall (terminal guide passage) (left)	50	18.08	0.261	0.09
H	Retaining wall (guide passage) (right)	50	19.52	0.286	0.80
I	Retaining wall (guide passage) (left)	50	19.04	0.317	0.10
J	P23 Right pier	50	19.77	0.295	0.09
K*	G6S4 (Girder)	25	18.86	0.362	0.31
L	G3S4 (Girder)	25	19.45	0.266	0.13
M	G10S4 (Girder)	25	19.22	0.282	0.24
N	G14S24 (Side)	50	18.55	0.294	0.20
O	G1S4 (Side)	50	19.9	0.263	0.03
P	G7S4 (Girder)	25	18.31	0.314	0.15
Q	G11S4(2) (Girder)	25	19.84	0.267	0.23
R	G2S4 (Girder)	25	19.9	0.298	0.14
S	G14S23 (Side)	50	18.64	0.276	0.20
T	G1S23 (Side)	50	18.66	0.321	0.39
	Average	42.5	19.10	0.311	0.183

* The carbonation depth of cored sample has surpassed concrete cover. Remark: A: Abutment; G: Girder; P: Pier; S: Slab (bridge deck).

Table 5. CNS 3090 specification of maximum concentration of chloride ions in concrete.

Structure type	Maximum concentration of chloride ions in concrete C (kg/m^3)
Prestressed concrete	0.15
Reinforced concrete (consider durability based on environment)	0.30
Reinforced concrete (general)	0.60

Note: If the maximum concentration of chloride ions in concrete is greater than 0.3 kg/m^3~0.6 kg/m^3, the steel in concrete should be performed corrosion protection.

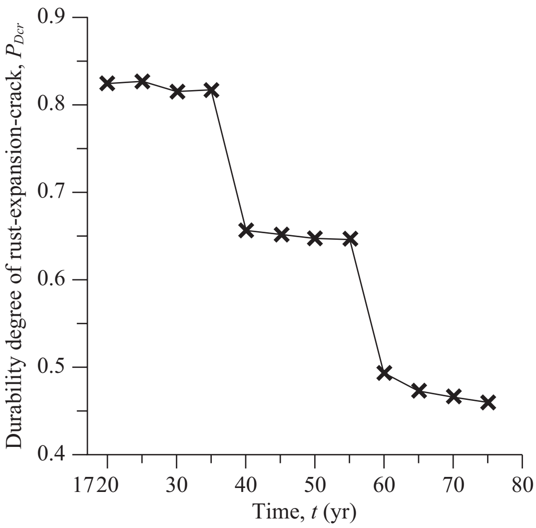

Fig. 5. Relationship between rust-expansion-crack durability degree and time for the Wann-fwu bridge.

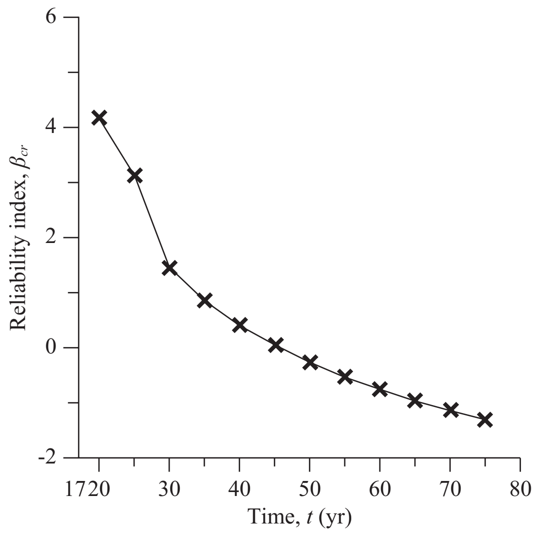

Fig. 6. Relationship between rust-expansion-crack reliability index and time for the Wann-fwu bridge.

index versus time for the existing Wann-fwu bridge and Chorng-ching viaduct were shown in Figs. 5-8, respectively. Table 6 [13] indicates the allowable probability of concrete cover owing to rust-expansion-crack. According to Table 6

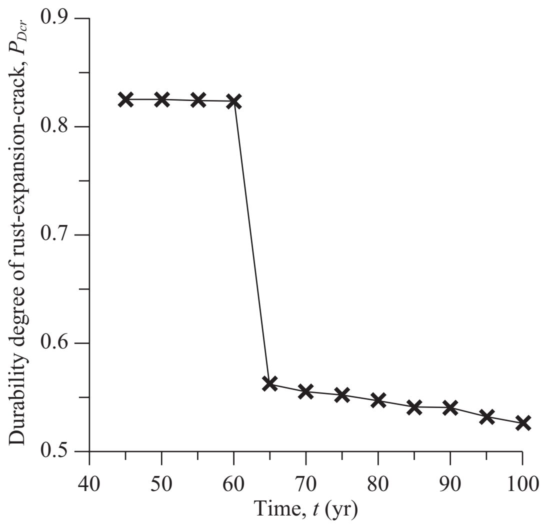

Fig. 7. Relationship between rust-expansion-crack durability degree and time for the Chorng-ching viaduct.

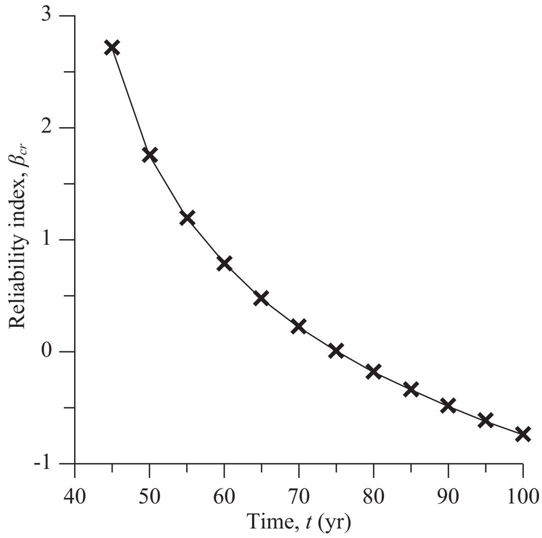

Fig. 8. Relationship between rust-expansion-crack reliability index and time for the Chorng-ching viaduct.

and Figs. 6 and 8 the service lives of rust-expansion-crack (t_{cr}) of existing Wann-fwu bridge and Chorng-ching viaduct were 38 and 65 and 33 and 57 years at reliability indexes β_{cr} = 0.5 and 1.0, respectively.

Table 6. Allowable probability of concrete cover due to rust-expansion-crack.

Classification		p_{fcr} (%)	β_{cr}
Presressed concrete structure		5	1.5
Concrete structure	Important structure	15	1.0
	General structure	30	0.5

Table 7. Prediction method for t_p for existing RC viaduct and bridge.

Prediction method	Formula	Remark	Reference
Bazant (Parabolic)	$t_p = \frac{1}{12D_c}\left(\frac{L}{1-\sqrt{\frac{C^*}{C_0}}}\right)^2$	D_C = Diffusion coefficiant of Cl^- (m^2/year) L = Concrete cover (m) C^* = Threshold value of Cl^- concentration (kg/m^3) C_0 = Cl^- concentration on the concrte surface (kg/m^3)	Bazant (1979)
Liang (Straight)	$t_p = \frac{1}{4D_c}\left(\frac{L}{1-\sqrt{\frac{C^*}{C_0}}}\right)^2$	D_C = Diffusion coefficiant of Cl^- (m^2/year) L = Concrete cover (m) C^* = Threshold value of Cl^- concentration (kg/m^3) C_0 = Cl^- concentration on the concrte surface (kg/m^3)	Liang *et al.* (2009)
Liang (Parabolic + Straight)	$t_p = \frac{7}{384D_c}[\frac{L}{1-\sqrt{\frac{C^*}{C_0}}}]^2 + \frac{1}{64D_c}[\frac{L}{1-\frac{C^*}{C_0}}]^2$	D_C = Diffusion coefficiant of Cl^- (m^2/year) L = Concrete cover (m) C^* = Threshold value of Cl^- concentration (kg/m^3) C_0 = Cl^- concentration on th econcrte surface (kg/m^3)	Proposed method

V. DISCUSSION

Eq. (2) has been shown that the service life of rust-expansion-crack of RC structure, t_{cr}, is the sum of carbonation service life, t_c and depassivation time, t_p.

The carbonation service lives of existing Wann-fwu bridge and Chorng-ching viaduct can be cited from Fang [5]. The corresponding values were 15 and 40 years. As to how to predict the values of t_p, the Bazant formula [1] and two approaches are listed in Table 7. It is worthwhile to point out that Bazant [1] used the concept of parabolic curve of chloride profile to predict the value of t_p due to the diffusion equation which is a kind of parabolic type of partial differential equation. Fang [5] and Liang *et al*. [10] applied the concept of declined straight line of chloride profile due to that chloride concentration is decreased as declined straight line when the depth of chloride penetration is increased. In present study, among the concepts of Bazant [1], Fang [5] and Liang *et al.* [10] are used to establish a model for predicting the value of t_p. The formula of this model is derived as shown in Appendix. Inverting $D_c = 77\ mm^2/yr$, $c^* = 8\ kg/m^3$, $C_0 = 25\ kg/m^3$ [11], and average L listed in Tables 3 and 4 into Table 7, the values of t_p for existing Wann-fwu bridge and Chorng-ching viaduct are listed in Tables 8 and 9, respectively.

Compared Tables 3 and 4 with Tables 8 and 9 we find that the value of t_p of bridge/viaduct member is longer when its concrete cover is larger. Based on Tables 8 and 9 the value of t_p obtained by the concepts of parabolic curve and straight line is less than 2~3 times that of t_p calculated by the Bazant formula. The value of t_p obtained by the concept of straight line is larger than 3 times that of the Bazant formula.

Aside from the concepts of parabolic curve or straight line or mixed type, the value of C_0 is an important parameter which is influenced on the predicted value of t_p. If taking the average of this three methods, then the values of t_p are 16.29 and 15.93 years for the existing Wann-fwu bridge and Chorng-ching viaduct, respectively.

Now consider the relationship between rust-expansion-crack durability degree and time as shown in Figs. 5 and 7. It is very obvious that these curves are discontinuous due to the corrosion of steel in concrete subjected to the chloride ingress which is a sort of pitting corrosion. Except for the influence factor of pitting corrosion, we need more to illustrate it. In the case of Fig. 5, the concrete covers of 40 *mm* and 50 *mm* were deteriorated during 35~40 and 55~60 years, respectively. In such measure as Fig. 7, the durability degree of rust-expansion-crack of Chorng-ching viaduct is of larger decrease during 60~65 years.

Owing to bridge or viaduct belonged to infrastructure and according to Table 6, we should choose $\beta_{cr}(t) = 1$ when the relationship between rust-expansion-crack reliability index and time is used to predict the value of t_{cr}. Based on Figs. 6 and 8, if subtracted the carbonation service life, t_c, then we obtain that the values of t_p are 21 and 25 and 16 and 17 years at β_{cr} = 0.5 and 1.0 for the existing Wann-fwu bridge and Chorng-ching viaduct, respectively. It is found that the values

Table 8. Calculated t_p of Wann-fwu bridge.

Test point No.	Concrete cover (*mm*)	t_p (yrs)		
		Bazant (1979) (p)	Liang *et al*. (2009) (s)	Proposed method (p+s)
A	40	9.18	27.54	2.71
B	40	9.18	27.54	2.71
C	40	9.18	27.54	2.71
D	40	9.18	27.54	2.71
E	40	9.18	27.54	2.71
F	40	9.18	27.54	2.71
G	40	9.18	27.54	2.71
H	40	9.18	27.54	2.71
I	40	9.18	27.54	2.71
J	40	9.18	27.54	2.71
K	25	3.59	10.76	1.06
L	50	14.34	43.03	4.24
M	50	14.34	43.03	4.24
N	50	14.34	43.03	4.24
O	50	14.34	43.03	4.24
P	50	14.34	43.03	4.24
Q	50	14.34	43.03	4.24
R	50	14.34	43.03	4.24
S	50	14.34	43.03	4.24
T	50	14.34	43.03	4.24
U	50	14.34	43.03	4.24
Average	44.04	11.37	34.12	3.36

*p: parabolic curve, s: straight line, p+s: parabolic curve + straight line.

Table 9. Calculated t_p of Chorng-ching viaduct.

Test point No.	Concrete cover (*mm*)	t_p (yrs)		
		Bazant (1979) (p)	Liang *et al*. (2009) (s)	Proposed method (p+s)
A	50	14.34	43.03	4.24
B	50	14.34	43.03	4.24
C	50	14.34	43.03	4.24
D	50	14.34	43.03	4.24
E	50	14.34	43.03	4.24
F	50	14.34	43.03	4.24
G	50	14.34	43.03	4.24
H	50	14.34	43.03	4.24
I	50	14.34	43.03	4.24
J	50	14.34	43.03	4.24
K	25	3.59	10.76	1.06
L	25	3.59	10.76	1.06
M	25	3.59	10.76	1.06
N	50	14.34	43.09	4.24
O	50	14.34	43.09	4.24
P	25	3.59	10.76	1.06
Q	25	3.59	10.76	1.06
R	25	3.59	10.76	1.06
S	50	14.34	43.03	4.24
T	50	14.34	43.03	4.24
Average	42.5	11.12	33.36	3.29

*p: parabolic curve, s: straight line, p+s: parabolic curve + straight line.

of t_p predicted from the reliability index versus time at $\beta_{cr} = 1$ are in good agreement with those values of t_p calculated by the Bazant formula. Hence, both bridge and viaduct really belong to an important RC infrastructure (see Table 8).

VI. CONCLUSIONS

The two service life models of RC structures and the analytical theory of rust-expansion-crack service life have been described in this paper. The service life model of existing RC structure consists of three phases, initiation (diffusion or carbonation) time, t_c, described by Fick's second law, depassivation time, t_p, and propagation (corrosion) time, t_{corr}. The rust-expansion-crack service life, $t_{cr} = t_c + t_p$, is primary issue in this paper. The values of t_{cr} predicted from the relationship between reliability index and time at $\beta_{cr} = 1.0$ for the existing Wann-fwn bridge and Chorng-ching viaduct are 33 and 57 years, respectively. The values of t_{cr} estimated from the sum of t_c = 15 and 40 years [5] and t_p = 11.37 and 11.12 years (average calculated from the Bazant formula and listed in Tables 8 and 9) are 26.37 and 51.12 years for the existing Wann-fwn bridge and Chorng-ching viaduct, respectively. It is worthy of notice that the results of t_{cr} predicted from the β_{cr} *vs*. t are coincided with the results of t_{cr} calculated from $t_{cr} = t_c + t_p$. The results of this study may provide a basis for repair, strengthening, and demolition of existing RC bridges or viaducts. The prediction method proposed in this paper can be extended to application for other existing RC bridges or viaducts.

APPENDIX

The time of depassivation, t_p, may be calculated from diffusion of Cl^- ions. Because this diffusion is uncoupled and can probably be thought to be linear, we could solve t_p by the well known solution in terms of the error function. Nevertheless, Bazant [1] used parabolic curve to express the Cl^- profile. In the present study, the Cl^- profile may be divided into parabolic curve during $0 < x < \frac{H}{2}$ and straight line during $\frac{H}{2} < x < H$, where the varying penetration depth $x = H(t)$, as shown in Fig. A-1. The analytical process is described in the following.

1. The AB parabolic curve in Fig. A-1 can be expressed as

$$C = C_0(1 - \frac{x}{H})^2, \quad 0 < x < \frac{H}{2}, \tag{A-1}$$

where C_0 is the Cl^- concentration on the concrete surface. Differentiating with respect to x to Eq. (A-1), we have

$$\frac{\partial C}{\partial x} = C_0 \cdot 2(1 - \frac{x}{H})(-\frac{1}{H}) \tag{A-2}$$

For concrete surface (x = 0), Eq. (A-2) gives

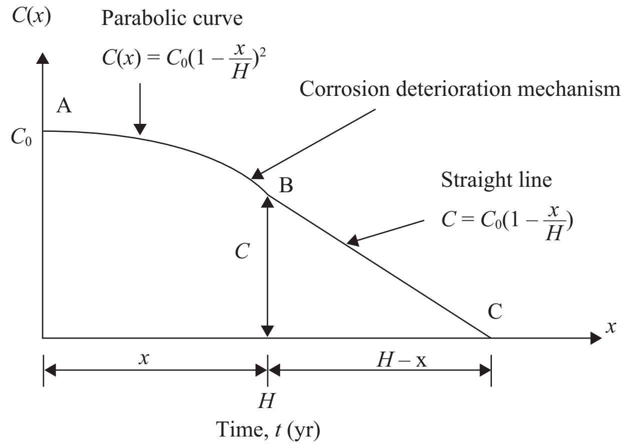

Fig. A-1. Combination of parabolic curve and straight line for the relationship between $C(x)$ and x.

$$-\frac{\partial C}{\partial x} = -\frac{2C_0}{H} \tag{A-3}$$

Eq. (A-3) multiplied by D_c, which is the coefficient of diffusion of Cl^-, and changed as

$$-D_c \frac{\partial C}{\partial x} = \frac{2C_0 D_C}{H} \tag{A-4}$$

The mass of Cl^- ions in concrete is

$$M_C = \int_0^{\frac{H}{2}} C dx \tag{A-5}$$

Substituting Eq. (A-1) into Eq. (A-5) and integrating, we obtain

$$M_C = \frac{7}{24} HC_0 \tag{A-6}$$

Differentiating with respect to time t to Eq. (A-6), we have

$$\frac{dM_C}{dt} = \frac{7}{24} C_0 \frac{dH}{dt} \tag{A-7}$$

The flux of Cl^- into concrete at x = 0 must equal to dM_C/dt, i.e., Eq. (A-4) is equal to Eq. (A-7).

$$\frac{dM_C}{dt} = \frac{2C_0 D_C}{H} \tag{A-8}$$

From Eqs. (A-7) and (A-8), we obtain

$$\frac{2C_0 D_C}{H} = \frac{7}{24} C_0 \frac{dH}{dt} \tag{A-9}$$

After integrating to Eq. (A-9), we have

$$t = \frac{7H^2}{96D_c} \tag{A-10}$$

When the penetration depth $x = \frac{L}{2}$, where L is the concrete cover, Eq. (A-1) can be rewritten as

$$H = \frac{L}{2(1 - \sqrt{\frac{C^*}{C_0}})} \tag{A-11}$$

where C^* is the threshold value of Cl^- ions concentration. The substitution of Eq. (A-11) into Eq. (A-10) yields the time of depasivation t_{p_1}

$$t_{p_1} = \frac{7}{384D_c}[\frac{L}{1 - \sqrt{\frac{C^*}{C_0}}}]^2 \tag{A-12}$$

2. The BC straight line in Fig A-1 can be described as

$$C = C_0(1 - \frac{x}{H}), \quad \frac{H}{2} < x \le H \tag{A-13}$$

Differentiating with respect to x to Eq. (A-13), we have

$$\frac{\partial C}{\partial X} = -\frac{C_0}{H} \tag{A-14}$$

Eq. (A-14) multiplied by $-D_c$ and changed as

$$-D_c \frac{\partial C}{\partial X} = \frac{C_0 D_c}{H} \tag{A-15}$$

The mass of Cl^- ions in concrete is

$$M_c = \int_{\frac{H}{2}}^{H} C dx \tag{A-16}$$

Substituting Eq. (A-13) into Eq. (A-16) and integrating, we obtain

$$M_c = \frac{C_0 H}{8} \tag{A-17}$$

Differentiating with respect to time t to Eq. (A-17), we have

$$\frac{dM_c}{dt} = \frac{C_0}{8}\frac{dH}{dt} \tag{A-18}$$

The flux of Cl^- into concrete at $x = \frac{H}{2}$ must equate to dM_c/dt, i.e., Eq. (A-15) is equal to Eq. (A-18)

$$-\frac{dM_c}{dt} = \frac{C_0 D_c}{H} \tag{A-19}$$

Eq. (A-18) equals Eq. (A-19), i.e.,

$$\frac{C_0}{8}\frac{dH}{dt} = \frac{C_0 D_c}{H} \tag{A-20}$$

After integrating, Eq. (A-20) becomes

$$t = \frac{1}{16D_c} H^2 \tag{A-21}$$

When the penetration depth $x = \frac{L}{2}$, Eq. (A-13) can be rewritten as

$$H = \frac{L}{2\left(1 - \frac{C^*}{C_0}\right)} \tag{A-22}$$

Substituting Eq. (A-22) into Eq. (A-21), we obtain the time of depassivation t_{p_2}

$$t_{p_2} = \frac{1}{64D_c}\left[\frac{L}{1 - \frac{C^*}{C_0}}\right]^2 \tag{A-23}$$

The sum of Eqs. (A-12) and (A-23) is

$$t_p = t_{p_1} + t_{p_2} = \frac{7}{384D_c}\left[\frac{L}{1 - \sqrt{\frac{C*}{C_0}}}\right]^2 + \frac{1}{64D_c}\left[\frac{L}{1 - \frac{C*}{C_0}}\right]^2 \tag{A-24}$$

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